MONTGOMERY STORMWATER MANAGEMENT MANUAL

City of

MONTGOMERY

Alabama

Manual No. ___

CH2M HILL

PREFACE

This Stormwater Management Manual is intended to document acceptable stormwater design approaches for local agencies, engineers, developers, or others when their activities deal with stormwater management in Montgomery. It is available from the City Engineering Department of Montgomery for a fee sufficient to cover copying costs.

ACKNOWLEDGEMENTS

This manual represents the combined efforts of many individuals who worked directly on the project, responded with helpful comments, or served in a review capacity. All contributions to the manual are gratefully acknowledged.

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Guidance and review from the City of Montgomery contributed substantially to the development of this manual. The following individuals provided help during the preparation and review of the manual:

o	James H. Sigler	Chief Administrator Technical Services
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EXECUTIVE SUMMARY

The Montgomery Stormwater Management Manual was developed to serve as a key element in a comprehensive Stormwater Management Program for the City of Montgomery. This manual is intended to document acceptable stormwater design approaches for local agencies, engineers, developers, or others engaged in stormwater management.

The emphasis of the Montgomery Stormwater Management Program is to promote a basinwide approach to watershed development planning, designing, and construction. Any alteration of existing land use to another form of development which will impact the quantity or quality of stormwater runoff will require implementation of appropriate stormwater management techniques. No stripping of vegetation, earth moving, or landfilling will be permitted until a stormwater management plan has been approved in accordance with the procedures established by the Subdivision Regulations and Zoning Ordinance of the City of Montgomery.

This manual recognizes the fact that stormwater management design approaches must be sufficiently flexible to deal with the myriad of problems associated with urban stormwater. It is therefore intended that this manual be utilized in a manner consistent with promoting design professionalism in the City of Montgomery: approaches not documented in this manual can be utilized subject to approval by the City Engineering Department.

A brief synopsis of each chapter in the manual follows:

Chapter 1--Introduction

Major aspects of stormwater management design are presented on Figure 1-3, showing typical examples of each component considered, along with tools required for design calculations. The major parts of the manual are identified, as follows:

Part I	-	Guidelines for Stormwater Management in Montgomery
Part II	-	Hydrologic Analysis
Part III	-	Stormwater Management System Design

A brief synopsis of each part is included in Chapter 1.

Chapter 2--Stormwater Problems in Montgomery

Drainage characteristics of Montgomery are described, and typical stormwater problems are illustrated with photographs. Four areas of stormwater problems are identified: (1) drainage, (2) flooding, (3) erosion, and (4) drainage system maintenance. Drainage problems include street flooding, standing water, and flow transition sections (e.g., open channel to a culvert). Flooding problems typically occur either as a result of an intense local rainstorm or when the Alabama River crest is reached due to upstream inputs. These two events do not necessarily occur concurrently; one may occur without the other. Erosion and sedimentation is one of the greatest stormwater problems facing Montgomery officials. Uncontrolled erosion results in plugged culverts, possible foundation damage to structures (e.g., highways or buildings) and the potential for increased flooding due to decreased channel capacities. Proper drainage system maintenance requires provisions for adequate easements and access ramp such that work crews can perform maintenance duties. Easement requirements are presented in Chapter 3.

Chapter 3--Overview of Stormwater Management in Montgomery

The following key elements of the Montgomery Stormwater Management Program are considered:

1. Program Philosophy and Policy Statements
2. Subdivision Regulations and Zoning Ordinance
3. Obtaining a Building Permit
4. Obtaining a Certificate of Occupancy

The following general policy statements will apply to the Stormwater Management Program within the City of Montgomery:

1. It is the intent of the Montgomery Stormwater Management Program to develop a master stormwater management plan for each major urban watershed within the City of Montgomery. This master plan shall include appropriate flood elevation profiles, flood insurance rate maps, and details related to existing and proposed components of the stormwater management system.

2. Each individual project shall be evaluated in relation to how it impacts the master stormwater management plan for the major urban watershed or watersheds within which the project site is located.

3. In the absence of such a master plan, a system of uniform requirements shall be applied to each individual project site. In general, these uniform requirements will be based on the criterion that post-development stormwater quantity and quality must not differ significantly from pre-development conditions. A general rule to follow is to correct all possible downstream problems before proceeding with upstream improvements. If downstream flooding problems cannot be corrected, upstream peak flows must somehow be attenuated to relieve downstream problems.

4. New construction may not aggravate upstream or downstream flooding during the 100-year rainfall event.

5. Unwarranted acceleration of erosion due to various land development activities must be controlled.

6. An adverse accumulation of eroded soil particles in the major stormwater management system must be avoided.

7. This Stormwater Management Manual is intended to establish accepted stormwater design procedures for the Montgomery Stormwater Management Program. Procedures not included in the Manual can be accepted subject to approval by the City Engineering Department of Montgomery.

8. Development within a flood plain shall be consistent with the Flood Plain Policy Statement presented in Table 10-1 of this Manual and Flood Plain Resolutions 38-74 and 39-74 presented in Appendix C.

9. All temporary erosion control systems shall conform to the requirements of Section 665 of the Alabama Highway Department Standard Specifications.

A flow chart of the procedures required by the Subdivision Regulations and Zoning Ordinance is presented on Figure 3-1. It is important to note that an applicant wishing to develop a particular area in Montgomery can take two possible pathways as shown on Figure 3-1. If the area to be developed is not subdivided, subdivision plat approval is required. If the area to be developed is already subdivided, development plan approval is required. Details related to each of these procedures are itemized. In addition, diagrams of the pre-construction review process and final inspection request are presented on Figures 3-2 and 3-3, respectively.

Chapter 4--Hydrologic Data

Published hydrologic data for Montgomery presented in Chapter 4 relate to precipitation, soils (both runoff and erodibility data), land use and topography, and streamflow. In addition, guidance for collecting site-specific field data is offered. Rainfall intensity-duration-frequency curves and depth-duration-frequency data for Montgomery are shown on Figure 4-2. A summary of addresses for locating published or readily available hydrologic data in Montgomery is presented in Table 4-1. Major urban watersheds are listed in Table 4-6 and located on Figure 4-6.

Chapter 5--Stormwater Runoff Estimation

Conceptual hydrologic design models and empirical approaches which are recommended as hydrologic design tools in Montgomery are presented in Chapter 5. Recommended guidelines for selecting a design storm return period in Montgomery are presented in Table 5-1. Hydrologic methods considered in Chapter 5 include the Rational Method, Soil Conservation Services (SCS) curve number approaches, unit hydrograph theory, flood frequency regression equations, and historical high water marks. The recommended applicability of four peak flow estimating procedures for Montgomery is presented in Table 5-7.

Unit hydrograph theory is best suited for final design computations when a runoff hydrograph is required, because each increment of excess rainfall is explicity routed through the subject watershed.

Chapter 6--Sediment Yield Estimation

The material presented in Chapter 6 focuses on application of the Universal Soil Loss Equation (USLE) to site conditions. Soil erodibility and rainfall factors for use with the USLE are provided. In all cases, the average annual soil loss from a construction site with appropriate erosion control measures shall be calculated to be less than 15 tons/acre/year. Details related to the requirements for, and performance of, erosion and sediment control systems are presented in Chapter 9.

Chapter 7--Hydraulic Design of Stormwater Conveyance Systems

The procedures identified in Chapter 7 require appropriate hydrologic data, developed according to the procedures presented in Chapter 5, as an input to the hydraulic design process. Four types of stormwater conveyance structures are considered: (1) open channels, (2) culverts, (3) stormwater inlets and gutter flows, and (4) storm sewers.

Open channels are generally designed using Manning's equation. Values of Manning's roughness coefficient required for open channel design in Montgomery are reported in Tables 7-2 and 7-3. Open-channel design charts are provided in this manual for nonvegetative-, vegetative-, and riprap-lined channels. The steepest side slopes allowed for concrete-lined channels are 1.5:1 (horizontal to vertical) and 3:1 for grass-lined channels. To allow for adequate maintenance, the easements identified in Chapter 3 for obtaining a building permit are required for all open channels. In addition, concrete-lined pilot channels or pipe underdrains are required to convey the base flow in grass-lined open channels. A cross slope or superelevation of 1/4-inch per foot is required for all curves in concrete-lined open channels as shown on Figure 7-48.

The hydraulic design of a culvert is a trial and error process. A trial culvert size is assumed and then the capacity is determined using nomographs presented in the manual to determine if the culvert size selected will satisfy the conditions prevailing at the proposed location. It should be noted that if a culvert is found to be operating under inlet control, modifications downstream of the inlet will not increase the capacity of that culvert. Class IV (ASTM 76) reinforced concrete pipe is required under streets in Montgomery.

Hydraulic design procedures are presented for curb-opening, grate, and combination stormwater inlets along with a special form of Manning's equation to analyze gutter flow. The design procedures for curb-opening and grate inlets are further classified as continuous grade or sump condition procedures. Curb-opening inlets are generally preferred to grated inlets because they are usually less susceptible to clogging. In addition, they are less hazardous to cyclists. The maximum distance allowed between stormwater inlets in Montgomery is 500 feet. The maximum allowable gutter spreads and depths of flow are presented in Tables 7-15 and 7-16, respectively. Concrete valley channel section, approved by the City Engineering Department, can be used in lieu of curb and gutters.

The Inlet Hydrograph Method of hydrologic routing is recommended as good engineering practice in Mongomery for storm sewer systems greater than 1,200 feet in length. For systems less than 1,200 feet in length the Rational Method is recommended. The hydraulic analysis of non-pressure pipe systems should be conducted using a special form of Manning's equation. A value of 0.012 (see Table 7-2) for Manning's roughness coefficient is required for concrete pipe in Montgomery. The final step of storm sewer design should be to analyze the system using hydraulic nomographs for culverts (i.e., pressure flow conditions) in order to determine the water surface elevation at critical points. The structural design of a closed conduit system should be conducted using the Concrete Pipe Design Manual, published by the American Concrete Pipe Association (1978). A minimum of Class III (ASTM 76) reinforced concrete is required with Class IV (ASTM 76) required under streets.

Chapter 8--Hydraulic Design of Stormwater Storage Systems

For the purposes of this manual, the primary function of a stormwater storage system is to reduce the peak flow of a hydrograph to a desired value. To determine the peak flow reduction obtained by a stormwater storage system, a reservoir routing procedure is required. Three approaches to reservoir routing are presented in this chapter. The first is applicable only to storage areas less than 20 acres and is called the Modified Rational Method. The second is a graphical procedure developed by the SCS which provides a quick preliminary routing for watersheds greater than 20 acres. The third is a hydrologic routing procedure, called the Storage Indication Method, which is to be used for conducting the final design of any basin with watershed areas greater than 20 acres.

Three types of stormwater storage are considered in Chapter 8: runoff control Best Management Practices (BMP's), detention storage, and retention storage. Runoff control BMP's which are considered include porous pavement, concrete grid pavements, rooftop ponding, parking lot ponding, and percolation storage. Stormwater storage basins can be designed to provide temporary or permanent storage. Temporary storage is commonly called "detention storage," while permanent storage is commonly called "retention storage." A schematic diagram of these two types of storage is presented on Figure 8-4. Special design factors for retention storage are discussed.

Chapter 9--Erosion and Sediment Control Systems

Performance factors for various erosion and sediment control practices are presented in Chapter 9. These performance factors are to be utilized in conjunction with the USLE, presented in Chapter 6 to ensure that the average annual soil loss from a construction site is less than 15 tons/acre/year. The maximum allowable exposure for unprotected soil is 90 days. All temporary erosion control systems shall conform to the requirements of Section 665 of the Alabama Highway Department Standard Specifications.

Chapter 10--Flood-Plain Management

Flood plains in Montgomery have been delineated and are summarized in the Flood Insurance Study for the City of Montgomery. This study was recently completed by the U.S. Army Corps of Engineers and includes a series of flood boundary maps.

City policy related to development within designated flood plains is presented in Table 10-1. In addition, two flood-plain resolutions, passed by the Montgomery Board of Commissioners, are provided in Appendix C.

All encroachments within the 100-year flood plain will require a grading, filling, erosion, and surface drainage plan approved by the City Engineering Department of Montgomery. If flood-plain development is planned for a wetland area in or near navigable waters, it may be necessary to secure permits and approvals from the U.S. Army Corps of Engineers--Mobile District, U.S. Fish and Wildlife Service--Department of the Interior, and Alabama Water Improvement Commission.

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The objective of this Stormwater Management Manual is to provide the technical framework for implementing a comprehensive Stormwater Management Program in the City of Montgomery, Alabama. This framework is provided to document acceptable stormwater design approaches for local agencies, engineers, developers, and others engaged in stormwater management.

Stormwater design approaches must be sufficiently flexible to deal with a myriad of problems associated with urban stormwater. It is, therefore, intended that this manual be utilized in a manner consistent with promoting design professionalism in the City of Montgomery. This manual is intended to establish accepted stormwater design procedures in the City; procedures not included can be accepted subject to approval by the City Engineering Department of Montgomery.

The City of Montgomery is located in the northwest corner of Montgomery County along the Alabama River and is situated between Autauga, Elmore, and Lowndes Counties as shown on Figure 1-1. A detailed map of the City is presented on Figure 1-2 in Appendix D. Problems associated with stormwater management in Montgomery are typical of those found throughout the country in major metropolitan areas. Descriptions of specific problems in Montgomery, including photographs, are presented in Chapter 2.

Stormwater management is concerned with both stormwater quantity and stormwater quality. Although these two categories are unique, they are interdependent. As shown on the conceptual diagram of the major aspects of stormwater management design (Figure 1-3), stormwater quantity management can be further divided into stormwater storage and stormwater conveyance systems. With regard to stormwater quality management, suspended solids is the only parameter considered in detail in this manual. As shown on Figure 1-3, construction erosion control can be further classified as erosion prevention or sediment removal control measures. Typical examples of each of these components of stormwater management systems are shown on Figure 1-3 along with tools required for design calculations.

This manual is presented in three parts as follows:

1. Part I--Guidelines for Stormwater Management in Montgomery

2. Part II--Hydrologic Analysis

3. Part III--Stormwater Management System Design

Part I provides a format for identifying stormwater problems and an overview of stormwater management in Montgomery. This overview identifies the key elements of the Stormwater Management Program in Montgomery and presents operating policies of the City Engineering Department. This information includes procedures for obtaining plat approval before land is subdivided, the pre-construction review process for obtaining a building permit, and the final inspection request for obtaining a certificate of occupancy.

Part II provides published hydrologic data for Montgomery, guidance for collecting site-specific field data, and accepted hydrologic design tools for the Montgomery area. Hydrologic design tools are presented for estimating the stormwater runoff and sediment yield from small urban watersheds.

Part III provides accepted hydraulic design methods for stormwater conveyance and storage systems. Stormwater conveyance structures include open channels, culverts, stormwater inlets and gutter flow, and storm sewers. In general, structural and material specifications for stormwater structures are not included with these hydraulic design methods. Detailed standard plans and specifications for such structures are available from the City Engineering Department of Montgomery. Part III also presents information related to the planning and design of erosion control systems. Specifications of the Alabama Highway Department apply to erosion control systems. The final chapter of Part III deals with flood-plain management. Flood plains in Montgomery have been delineated and are summarized in the _Flood Insurance Study for the City of Montgomery_. Flood-plain policy and two flood plain resolutions for Montgomery are identified.

As noted in the Table of Contents, the tables and figures referred to in the text are located at the end of each chapter. Selected example problems are presented at the end of selected sections of chapters where deemed appropriate. A list of example problems, equations, and symbols is likewise provided at the end of appropriate chapters.

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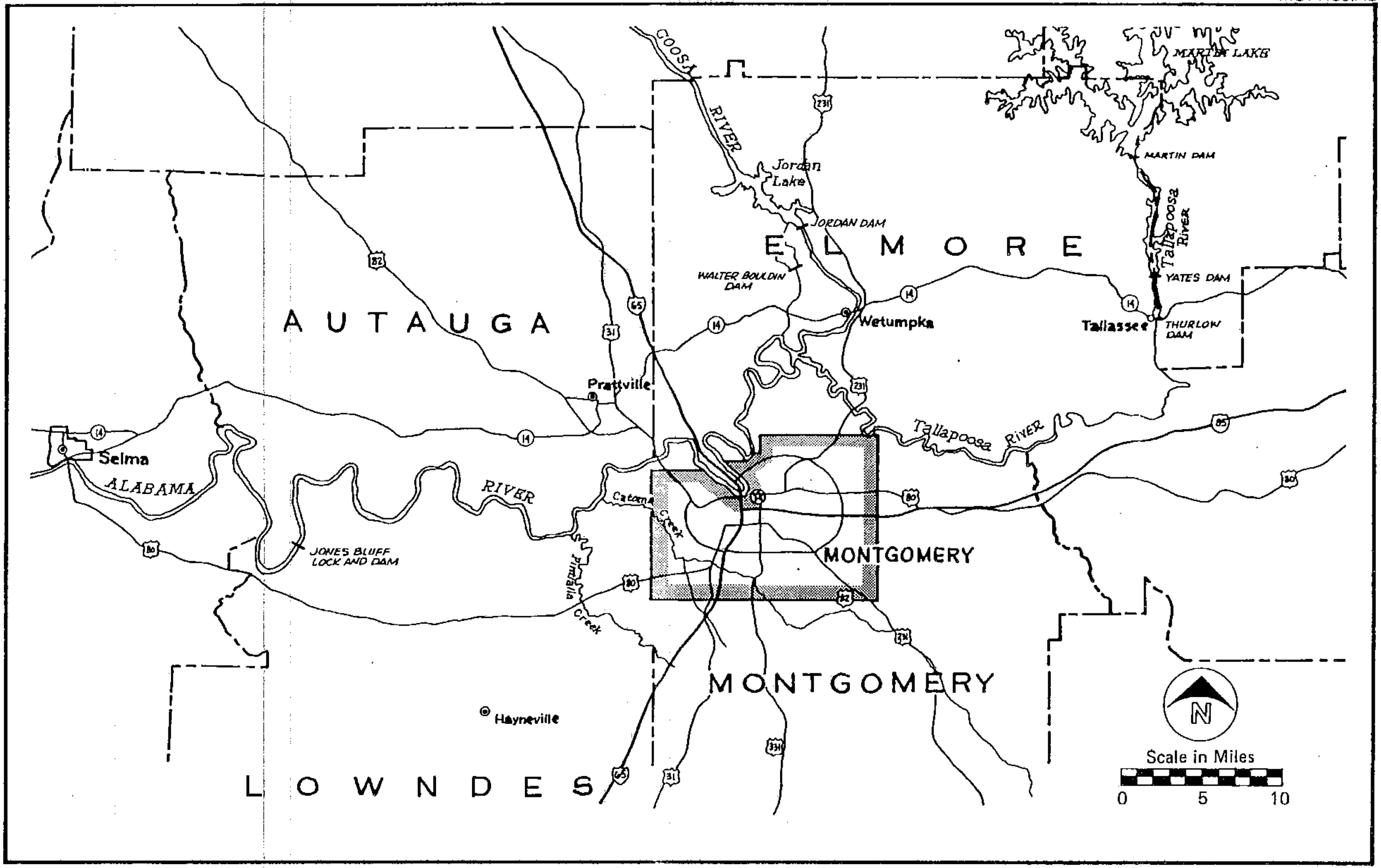

FIGURE 1-1. General location map of Montgomery, Alabama.

Figure 1-2.
Detailed map of the City of Montgomery, Alabama.

(Map inserted in Appendix D.)

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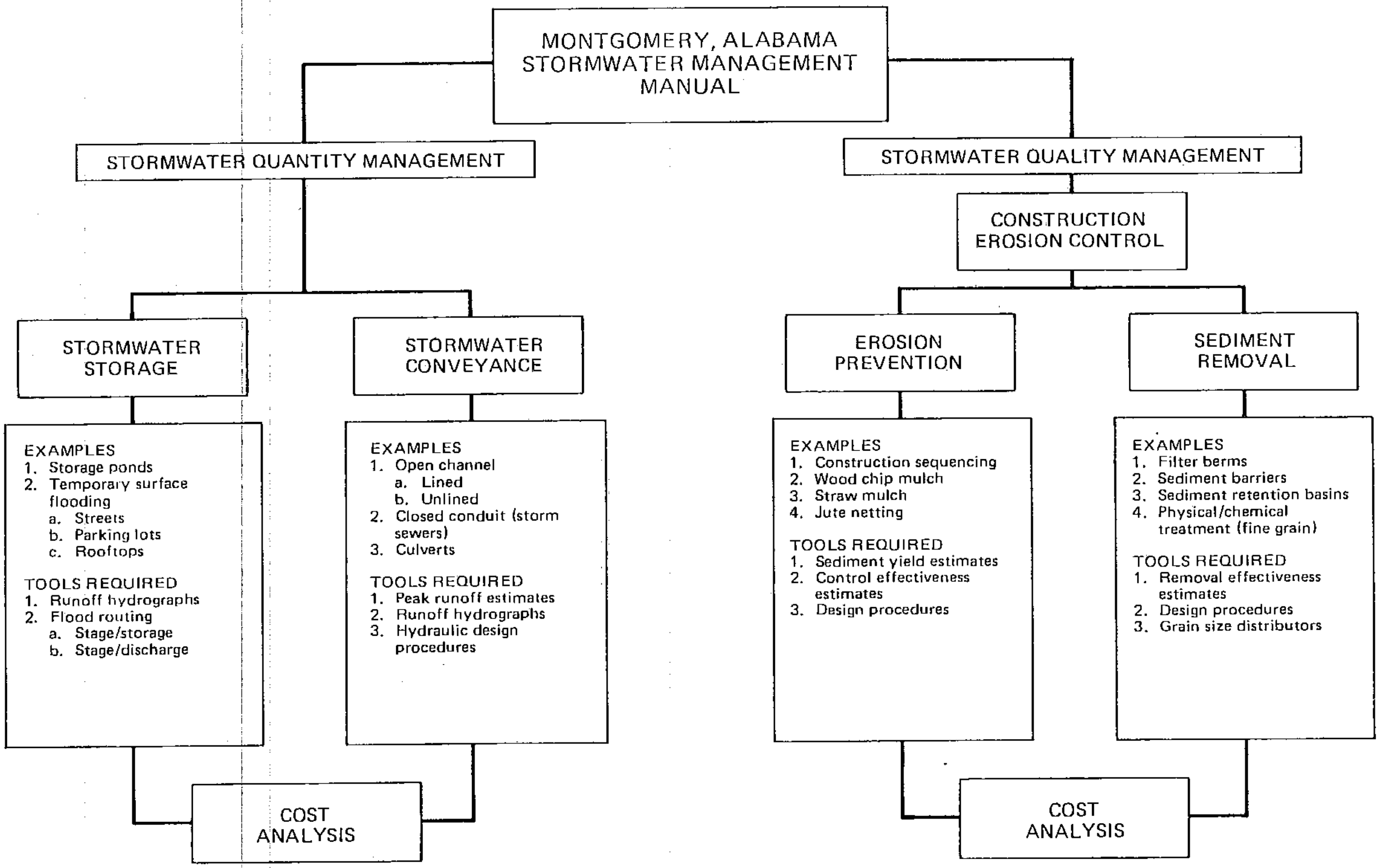

FIGURE 1-3. Major aspects of stormwater management design.

Chapter 2
STORMWATER PROBLEMS IN MONTGOMERY

SECTION 1.0 INTRODUCTION

The stormwater management problems found in Montgomery are typical of those found throughout the country in major metropolitan areas. Accommodating existing and future stormwater flow is a major item of concern to City officials, private developers, engineers, and planners when evaluating future growth potential for the area. This chapter discusses drainage characteristics for Montgomery and illustrates typical stormwater problems found in the area.

SECTION 2.0 INTERIOR DRAINAGE

The topography of Montgomery is flat to gently rolling. The main business district of Montgomery is located on relatively high ground, but is surrounded by low-lying areas and streams with wide flood plains. Interior drainage is accomplished by a network of natural and manmade ditches and sloughs. The natural waterways have little or no vegetative cover, except for weeds. Typical natural waterways in Montgomery are illustrated on Figures 2-1 and 2-2. A natural drainage channel with minor improvements through a subdivision near Lagoon Park is shown on Figure 2-1. A natural drainage downstream of Federal Drive on Three-Mile Branch is shown on Figure 2-1.

Concrete-lined open channels are typically installed in Montgomery when natural waterways are inadequate. Typical concrete-lined open channels in Montgomery are illustrated on Figures 2-3 and 2-4. A concrete-lined open channel in the Carriage Hills Subdivision is shown on Figure 2-3, and Figure 2-4 illustrates the confluence of Sherwood Ditch and Three-Mile Branch.

Major streams and ditches of Montgomery in which some form of excavation, realignment, or concrete lining has been installed or performed are Three-Mile Branch, Sherwood Ditch, Galbraith Mill Creek, Baldwin Slough, Hannon Slough, Audubon Ditch, Genetta Ditch, Cloverland Ditch, Catoma Creek, and West End Ditch. Numerous smaller ditches in developed areas have been improved.

The major urban watersheds of Montgomery are presented in Chapter 4 (see Table 4-6 and Figure 4-6). In general, the stormwater of Montgomery is transported from small ditches and sloughs to creeks and streams which discharge to either the Tallapoosa or Alabama Rivers. The Tallapoosa River defines the Elmore-Montgomery County line, is located northeast of the City of Montgomery, and meanders northwest.

North of Montgomery, the Tallapoosa joins the Coosa to form the Alabama River, which flows generally southwest. Sections of the Alabama River define the Elmore-Montgomery and Autauga-Montgomery County lines. The Alabama River is navigable from Montgomery to the Gulf of Mexico.

Overbank flooding from the Alabama and Tallapoosa Rivers is quite extensive in northern Montgomery areas, inundating the flood plains on many of the tributaries such as Catoma Creek, Oliver Creek, Galbraith Mill Creek, Three-Mile Branch, and West End Ditch. The problem is further compounded by the general flatness of the streambeds in the lower reaches near their confluences with the Alabama and Tallapoosa Rivers. Catoma Creek, with a total drainage area of approximately 368 square miles, drains the entire southern half of Montgomery. Numerous sloughs and ditches discharge into Catoma Creek, which generally flows northwest to the Alabama River. The West End Ditch drains northwest Montgomery and discharges directly into the Alabama River. The remainder of the City is drained northward by three primary waterways: Three-Mile Branch, Galbraith Mill Creek, and Oliver Creek. Three-Mile Branch joins Galbraith Mill Creek prior to final discharge at the Alabama River. Oliver Creek discharges to the Tallapoosa River near the northeast corner of the Montgomery City limits. Other sections of Montgomery which border the Alabama River drain to the river by way of numerous small ditches and creeks.

SECTION 3.0 STORMWATER PROBLEMS

Stormwater problems in Montgomery can be summarized under the following four areas: drainage, flooding, erosion and sedimentation, and drainage system maintenance. This section briefly discusses typical problems experienced in Montgomery.

3.1 Drainage Problems

Nuisance drainage problems, such as that shown on Figure 2-5, generally do not cause widespread damage during rain events. However, they can cause the inconvenience of requiring traffic to be detoured as shown on Figure 2-6. Street and parking lot flooding are generally classified as nuisance drainage problems, as they generally cause a temporary inconvenience to the general public rather than physical damage. This type of drainage problem can be caused by a variety of factors which include inadequate inlet design, inadequate culvert design, and obstructed inlets. Major design and construction is typically required to correct inadequate inlets and culverts. Where this corrective action becomes necessary, the evaluator should carefully consider downstream flow limitations so as not to compound or create downstream drainage and/or flooding problems.

During the fall months, inlets, culverts, and channels become laden with leaves and other vegetative material. This severely restricts the flow capacity of these drainage structures and typically causes nuisance drainage problems. However, these flooding problems are not frequent since long-duration, low-intensity rainfall generally occurs during this time of the year. Obstructed inlets and culverts cause frequent drainage problems in the summer due to the typically short-duration, high-intensity storms which occur from April to September.

Standing water is not only a nuisance drainage problem but can also threaten the health of the general public by providing breeding areas for mosquitoes and other insects. This problem is easily corrected through proper grading techniques. To provide adequate ground slope for correcting standing water problems, fill material is generally required. As discussed in Chapter 10, fill encroachment limits have been established in Montgomery flood plains; such encroachments must be approved by the City of Montgomery and Corps of Engineers.

An easily neglected component of a typical drainage system is the transition from open channel to culvert flow conditions. A properly constructed open channel to culvert transition is shown on Figure 2-7 which illustrates a cast-in-place concrete double box culvert at Eastdale Mall. Note the wingwalls and small overland flow channel at this culvert entrance. This properly constructed transition is contrasted with the culvert shown on Figure 2-8 which illustrates a concrete pipe culvert near Day Street. In this case, no wingwalls or overland flow channel were provided. The result is embankment erosion and standing water at the outlet.

3.2 Flooding Problems

In Montgomery, flooding problems typically occur either as a result of an intense rainstorm or the crest of the Alabama River. These two events normally do not occur concurrently; one may occur without the other. For example, on April 13, 1979, a storm passed through Montgomery which crested in the Alabama River at Montgomery on April 16, 1979. Events in the Alabama River Basin upstream of Montgomery have a dominant effect on the timing and magnitude of flood crests.

During moderate to intense local rainfall, major flooding can occur throughout the City of Montgomery. During the morning of May 9, 1978, an intense thunderstorm moved eastward across Montgomery County over an area where the soil had been saturated due to a light to moderate rainfall on the previous day. As a result of the runoff from this thunderstorm, moderate to severe flooding occurred in low-lying areas along most ditches and streams in Montgomery. This event caused considerable flooding and damage to numerous residential dwellings and commercial establishments. Flooding of commercial/business areas along Day Street due to the May 9, 1978 storm are shown on Figure 2-9, and flooding of residences along Homeview Street south of Day Street is shown on Figure 2-10. This flooding problem was caused by inadequate channel capacity.

Inadequate culvert and channel structures cause widespread flooding in other sections of Montgomery during moderate to intense rainfall. Before replacing inadequate structures, the evaluator must first consider what effect this action may have on the conditions downstream. A general rule to follow in urban stormwater management is that all possible downstream problems should be corrected before proceeding upstream with improvements. If downstream flooding problems cannot be corrected, upstream peak flows must somehow be attenuated to relieve downstream problems. Techniques to accomplish this are discussed in subsequent chapters.

The flood plain for the Alabama River and major tributaries has been delineated and is summarized in the Flood Insurance Study for the City of Montgomery, Alabama. The flood-plain maps available in this report enable the evaluator to determine which, if any, flood hazard zone his site occupies.

3.3 Erosion and Sedimentation Problems

One of the greatest stormwater problems facing Montgomery officials is that of erosion and sedimentation during rainfall events. Deposition of soil and silt downstream reduces the flow capacity of culverts and channels. A typical residential construction site without proper erosion and sediment control provisions is shown on Figure 2-11. Uncontrolled erosion such as this results in plugged culverts as shown on Figure 2-12. Since city work crews must be dispatched on a regular basis to clear the drainage paths of silt and clay, such plugging proves costly to the taxpayer.

Erosion occurs along natural channels which have no significant vegetation or along large unprotected open areas, such as crop lands and construction sites. Methods of estimating sediment yield and practical procedures to reduce soil loss are presented in Chapters 6 and 9, respectively.

3.4 Drainage System Maintenance

In addition to considering the erosion and sedimentation problems associated with a stormwater conveyance system, adequate provisions for periodic maintenance of the system are required. These provisions include easements and access ramps such that work crews can periodically remove vegetation and debris which accumulate in the system. In addition, if adequate low-flow pilot channels or underdrains are installed, the vegetation associated with standing water can be reduced.

Easement requirements for stormwater management structures in Montgomery are presented in Section 4.0 of Chapter 3, and pilot channel considerations for open channels are presented in Subsection 2.2.4 of Chapter 7.

An example of poor channel maintenance conditions downstream of Terminal Road culvert are illustrated on Figure 2-13. Note that no easement is provided along the channel for periodic maintenance. Poor channel conditions such as these can increase flood elevations as shown on Figure 2-14, which illustrates Community Street Bridge during the May 9, 1978 storm.

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FIGURE 2-1. Natural drainage channel through a subdivision near Lagoon Park (note the check dam).

FIGURE 2-2. Natural drainage channel of Three-Mile Branch down-stream of Federal Drive.

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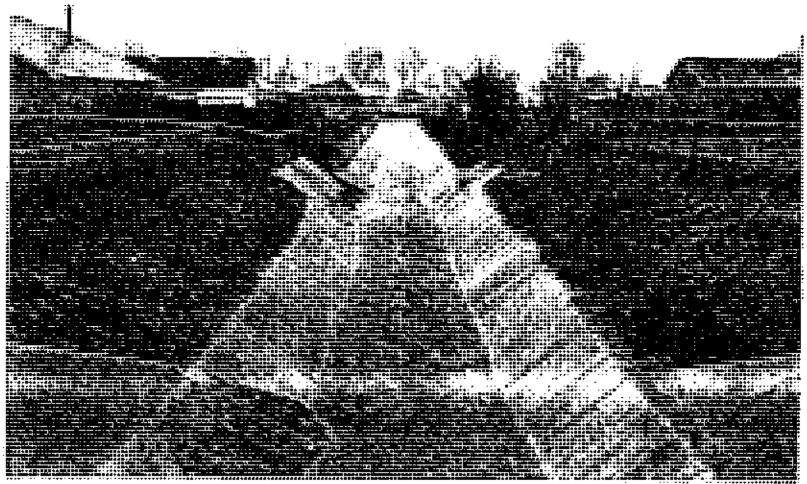

FIGURE 2-3. Concrete-lined open channel in Carriage Hills Subdivision.

FIGURE 2-4. Concrete-lined open channel at the confluence of Sherwood Ditch and Three-Mile Branch.

FIGURE 2-5. Major street flooding at the intersection of Carter Hill Road and Robinson Hill Road which caused traffic to be detoured.

FIGURE 2-6. Traffic detour due to street flooding shown on Figure 2-5.

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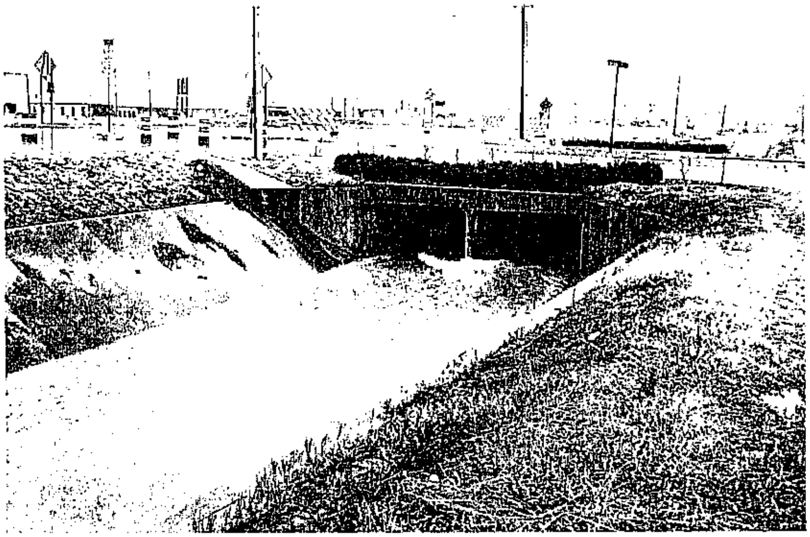

FIGURE 2-7. Properly constructed cast-in-place concrete double box culvert at Eastdale Mall.

FIGURE 2-8. Concrete pipe culvert near Day Street with inadequate erosion protection at the outlet.

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Courtesy of *Montgomery Advertiser—Alabama Journal.*

FIGURE 2-9. Flooding of commercial/business areas along Day Street as a result of the May 9, 1978 storm.

Courtesy of *Montgomery Advertiser—Alabama Journal.*

FIGURE 2-10. Flooding of residences along Homeview Street south of Day Street as a result of the May 9, 1978 storm.

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FIGURE 2-11. A residential construction site without proper erosion and sediment control provisions.

FIGURE 2-12. A culvert plugged with sediment.

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FIGURE 2-13. Poor channel conditions downstream of Terminal Road Culvert cause standing water and dense vegetation.

FIGURE 2-14. Poor channel conditions resulted in flooding of Community Street Bridge during the May 9, 1978 storm.

Chapter 3
OVERVIEW OF STORMWATER MANAGEMENT IN MONTGOMERY

SECTION 1.0 INTRODUCTION

It is the purpose of Chapter 3 to identify and briefly discuss the key elements of the Montgomery Stormwater Management Program which impact the various phases of a typical stormwater management project. The following key elements are included in the Montgomery Stormwater Management Program:

1. Program Philosophy and Policy Statements
2. Subdivision Regulations and Zoning Ordinance
3. Pre-Construction Review and Permitting
4. Construction Inspection and Enforcement

These key elements are tied together with the objective of minimizing or mitigating the adverse effects of development.

SECTION 2.0 PROGRAM PHILOSOPHY AND POLICY STATEMENTS

The emphasis of the Montgomery Stormwater Management Program is to encourage utilization of a basinwide approach to watershed development planning. Any alteration of existing land use to another form of urbanized development which will impact the quantity or quality of stormwater runoff will require implementation of appropriate stormwater management techniques. No stripping of vegetation, earth moving, or landfilling will be permitted until a stormwater management plan has been approved according to the procedures identified in Section 4.0 of this chapter.

Ideally, each individual project should be evaluated using a stormwater management master plan for the particular urban watershed within which the project site is located. It is the intent of the Montgomery Stormwater Management Program to develop a master stormwater management plan for each major urban watershed within the City. In the absence of such a master plan, a system of uniform requirements will be applied to each individual project. In general, these uniform requirements will be based on the philosophy that post-development stormwater quantity and quality must not differ significantly from pre-development conditions. Although this system of uniform requirements for each individual project may be more stringent than necessary, it does allow development to proceed in an orderly manner.

The following general policy statements will apply to the Montgomery Stormwater Management Program:

1. It is the intent of the Montgomery Stormwater Management Program to develop a master stormwater management plan for each major urban watershed within the City of Montgomery. This master plan shall include appropriate flood elevation profiles, flood insurance rate maps, and details related to existing and proposed components of the stormwater management system.

2. Each individual project shall be evaluated in relation to how it impacts the master stormwater management plan for the major urban watershed or watersheds within which the project site is located.

3. In the absence of such a master plan, a system of uniform requirements shall be applied to each individual project site. In general, these uniform requirements will be based on the criterion that post-development stormwater quantity and quality must not differ significantly from pre-development conditions. A general rule to follow is to correct all possible downstream problems before proceding with upstream improvements. If downstream flooding problems cannot be corrected, upstream peak flows must somehow be attenuated to relieve downstream problems.

4. New construction may not aggravate upstream or downstream flooding during the 100-year rainfall event.

5. Unwarranted acceleration of erosion due to various land development activities must be controlled.

6. An adverse accumulation of eroded soil particles in the major stormwater management system must be avoided.

7. This Stormwater Management Manual is intended to establish accepted stormwater design procedures for the Montgomery Stormwater Management Program. Procedures not included in the manual can be accepted subject to approval by the City Engineering Department of Montgomery.

8. Development within a flood plain shall be consistent with the Flood Plain Policy Statement presented in Table 10-1 of this manual and Flood-Plain Resolutions 38-74 and 39-74 presented in Appendix C.

9. All temporary erosion control systems shall conform to the requirements of Section 665 of the Alabama Highway Department Standard Specifications.

SECTION 3.0 SUBDIVISION REGULATIONS AND ZONING ORDINANCE PROCEDURES

The existing Subdivision Regulations and Zoning Ordinance for the City of Montgomery establish a process of review and inspection to ensure that improvements required for development are adequate to protect public health, life, and property. Therefore, the legal framework for implementing a stormwater management program is in place. This legal framework will serve as the basis for implementing the Stormwater Management Program in Montgomery.

An applicant wishing to develop a particular area in Montgomery can take two possible pathways. If the area to be developed is not subdivided, the procedures for subdivision plat approval apply. If the area to be developed is already subdivided, the procedures for review and approval of development plans should be followed. A schematic diagram of these two pathways is presented on Figure 3-1, and procedures for these two pathways are identified below.

3.1 Subdivision Plat Procedures

Three different levels of subdivision plat review procedures may apply. These levels are (1) pre-application, (2) preliminary plat approval, and (3) final plat approval.

3.1.1 Pre-Application Procedures

This level of application review provides the subdivider, at his option, with an opportunity to informally describe his plat to the Planning Commission. The pre-application procedures are identified in Section 2.A of the Subdivision Regulations. The data required for this step may be presented in the form of a freehand sketch showing the basic lot sizes: lots need not be shown on the basic street layout. Pre-application approval does not ensure preliminary plat approval, but it is an opportunity for potential problems to be identified early in the review process.

3.1.2 Preliminary Plat Procedures

Preliminary plat procedures for Montgomery are identified in Section 2.b of the Subdivision Regulations. The preliminary plat, together with other supplementary material as deemed necessary by the Planning Commission, shall be prepared as specified in Section 3 of the Subdivision Regulations. Four copies of the preliminary plat and supplementary material specified shall be submitted with the names of surrounding property owners, if not previously furnished, to the Planning Director at least 7 days prior to the meeting of the Planning Commission at which the preliminary plat is to be considered.

The following steps constitute the preliminary plat review procedures.

1. Application is submitted to the Planning and Development Department for processing prior to a Planning Commission hearing.

2. The application is reviewed by the Planning and Development Department for compliance with the Zoning Ordinance and Subdivision Regulations.

3. The application is then forwarded to the City Engineering Department, Fire Department, Traffic Engineering Department, Water & Sewer Board, and Health Department for their review and comment.

4. The application is then placed on the Planning Commission agenda for a public hearing, at which time the application is approved or denied, or granted approval with special requirements or stipulations put on the plan by the Planning Commission or by the Commission at the request of a City Department.

5. After the public hearing and approval, the developer can then submit his construction plan to the City Engineering Department for approval.

6. The subdivider or developer shall not begin any construction without first notifying City Engineering, Water & Sewer Board, Traffic Engineering, or the County if in the police jurisdiction.

7. The plans are reviewed by the City Engineering Department, Water & Sewer Board, and Traffic Engineering for compliance with City design standards or requirements, such as those for sidewalks, sanitary facilities, storm drainage, street and road development and improvement, water mains, street lights, street signs, and other required street monuments and markers, as well as the easement size required for the improvement.

8. After the review of the construction plans by the Traffic Engineer and Water & Sewer Engineer, a letter of approval from these departments is filed with the City Engineering Department.

9. If any special requirements or stipulations are placed on the approval, they should be stated in the letter to the City Engineering Department.

10. After City Engineering has received the approval letter from Traffic Engineering and the Water & Sewer Board, and after review and approval of the construction plans, a letter of approval can be granted the developer.

11. If the subdivider wishes to file a final plat prior to completing the required improvements, a Performance Bond will be required at this point.

3.1.3 Final Plat Procedures

Final plat procedures for Montgomery are identified in Section 2.0 of the Subdivision Regulations.

The final plat to be prepared, as specified in Section 3 of the Subdivision Regulations, shall conform substantially to the preliminary plat as approved; if desired by the subdivider, it may constitute only that portion of the approved preliminary plat which he proposes to record and develop at the time; all of this provided, however, that such portion conforms to all requirements of the Subdivision Regulations.

The following steps constitute the final plat review procedure.

1. The application is submitted to the Planning and Development Department for processing prior to Planning Commission hearing.

2. The application is reviewed by the Planning and Development Department for compliance with the Zoning Ordinance and Subdivision Regulations.

3. The application is then forwarded to the City Engineering Department, Fire Department, Traffic Engineering Department, Water & Sewer Board, and Health Department for their review and comment.

4. The application is then placed on the Planning Commission agenda for a public hearing, at which time the application is approved or denied, or granted approval with special requirements or stipulations on the plan by the Planning Commission or by the Commission at the request of a City Department.

5. After the Planning Commission approves the final plat, it is resubmitted to City Engineering, Traffic Engineering, and Water & Sewer Board for final approval before the plat is to be recorded. At this time, all improvements should have been made and construction completed. The plat is signed by City Engineering, Traffic Engineering, and Water & Sewer Board prior to the approval by the Secretary of the Planning Commission for recording. Prior to the approval by City Engineering, a one-year Performance Bond is to be filed with the Department to cover the maintenance of the development if any should be required in the first year of use.

6. After the plat has been recorded, it is assigned street numbers, and a copy of the recorded plat is filed in the Planning and Development Department, City Engineering Department, Fire Department, Building Department, Traffic Engineering Department, and County Health Department for future use.

3.2 Development Plan Procedures

The following procedures for review and approval of development plans located in the following districts have been established as required in the Subdivision Regulations (Section 6, Non-Residential Subdivisions) and Zoning Ordinance (Ord. 31-73).

Commercial	Industrial	Office	Residential
B-2	M-1	0-1	R-99-p
B-3	M-2	0-2	R-99-s
B-4	M-3		R-20-t
B-5			Group Housing Projects

1. A development plan for this purpose shall include, but not be limited to, the following plans, designs, specifications, and information:

   a. Site plan with grades or contours and drainage plan.

   b. Building site locations.

   c. Location and size of all utilities, existing and proposed.

   d. All curb-cuts, driveways, parking areas, and types of construction material for same.

   e. All pedestrian walks, yards, and open spaces.

f. Location of all railroad tracks and spurs.

g. Location, height, and material of all walks, fences, and screen planting.

h. Location, size, character, height, and orientation of all signs.

i. Traffic analysis, showing the effect of the proposed development on neighboring streets.

j. All plans submitted will be to scale, with all dimensions of proposed buildings and any other necessary dimensions such as side yard setbacks, and other setbacks as required by the Zoning Ordinance.

k. A Vicinity Sketch at a scale of 1" = 200' or larger covering a distance of 500 feet in each direction, and the names of existing businesses or offices, etc., located adjacent to the site in question.

2. The plat or development plan shall conform to the Zoning Ordinance with respect to any and all requirements for area, building coverage, parking, and loading and unloading facilities, as well as the type of uses permitted or limited.

3. Before granting approval of any development plan in the above-named districts, the Planning Commission shall receive a report from the City Engineer, Fire Department, Traffic Engineer, Water & Sewer Board, and the Planning and Development Department, that the development plan conforms with all requirements of the City Code and that the plan will be consistent with the general health, safety, and welfare of the City.

4. Building plans can be submitted to the Building Department prior to Planning Commission action to expedite receiving a building permit.

5. Four (4) copies of the site development plan will be submitted to the Planning and Development Department, along with the $50.00 processing fee (made payable to the City of Montgomery).

6. All plans submitted will comply with the above guidelines before they are circulated to the required departments for review and before they are placed on the Planning Commission agenda for action.

SECTION 4.0 OBTAINING A BUILDING PERMIT

As discussed in Section 3.0 of this chapter, an applicant wishing to develop a particular area may be required to submit a subdivision plat for a development plan. In addition, the Montgomery Zoning Ordinance requires a building permit prior to the excavation for the construction of any building or other structure, including accessory structures, to store building materials or erect temporary field offices, or to commence the moving, alteration, or repair of any structure, including accessory structures. Information required to obtain approval of plans and issuance of a building permit are identified under Item 3 in Article II of the Montgomery Zoning Ordinance.

A detailed diagram of the pre-construction review process for the issuance of a building permit is shown on Figure 3-2. The following information related to stormwater quantity must be submitted to the Montgomery Engineering Department:

1. Elevations based on mean sea level as established by U.S.C. and G.S.

2. Grading and drainage plan drawn on a map showing existing topography with 2-foot maximum contours, (1-foot in flat land).

3. Design criteria and supporting data. Calculations for each drainage structure, pipe, and open channel. This can be shown on plans or in a separate report.

4. Offsite drainage areas discharging to or through the tract, or affecting the site.

5. Record of ground-water table and method of dipsosal.

6. Location, type, size, invert elevation, and slope of all drainage structures, pipes, and open channels. Dimensioned cross section and flow line profiles of proposed and existing channels will be shown, including top of right and left bank profiles. Where drainage ditches are used, design velocity and method of erosion control to be used on banks and bottoms will be indicated.

7. Detail drawings of all permanent stormwater management structures.

8. Information on all streets as to full right-of-way, grading, sidewalks, base, wearing surface, curb and gutter, and center line gradients, existing and proposed.

9. Details of driveway aprons.

10. The following easements must be shown for the appropriate stormwater structures:

a. Concrete-lined ditches. For equipment access and ditch maintenance activities, a 15-foot area is required from the top of the slope to the edge of the easement on one side, and a 3-foot area from the top of the slope to the edge of the easement on the other side. For ditches with a bottom width of more than 12 feet, a 15-foot-wide access ramp leading at an appropriate location from the above-mentioned working area in the easement for entry into the ditch is necessary. Maximum side slopes are 1.5 foot horizontal to 1 foot vertical (1.5:1).

b. Grass-lined ditches. For ditches up to 16 feet in width (top of bank to top of bank) a minimum distance of 15 feet is required on one side and 6 feet on the other. If the ditch is more than 16 feet in top width, the required working area is 15 feet on one side and 10 feet on the other.

c. General. The above-specified distances are for minimum easement widths in which fences are not to be located. The width of an easement should be continuous throughout the new development, and the maximum side slope for the top of ditch working areas shall be no greater than a distance of 3 feet horizontal to 1 foot vertical (3:1).

Ten-foot private drainage easements on the rear of each lot and 20-foot along the boundary line of the subdivision will be shown. Twenty-foot easements from the street to the rear lot line easements at each end of the blocks and all natural drains will be shown. Easements may be reduced where drainage pipe is installed. All culverts are to be provided with headwalls at each end and outfall aprons where necessary. Where culverts are on lot lines between dwellings, necessary easements and the culvert extending to the rear property lines will be shown.

d. Storm sewers. The following easements are required for storm sewer closed conduits.

(1) Circular pipes 15 inches to 30 inches in diameter requires 10-ft easement width.

(2) Circular pipes 42 inches to 72 inches in diameter requires 15-ft easement width.

(3) Box culverts or others = outside dimension plus a 5-ft easement on each side.

11. Plans, hydraulic computations, and high water information on all stormwater outfalls and open channels affecting the tract.

12. Proposed lot and block grading shown on plans.

The following information related to stormwater quality should be submitted to the Engineering Department:

1. The site grading plan showing existing topography and planned grades for stormwater drainage patterns, along with proposed street or highway profiles.

2. The plan for stormwater conveyance structures, including at least the following items:

   a. Location of open channels, culverts, stormwater inlets, storm sewers, and storage structures.

   b. Discharge capacity and design velocities for each component of a stormwater conveyance system.

   c. Supporting design computations for each stormwater conveyance structure identified.

3. Proposed erosion and sediment control plan, including at least the following items:

   a. Average annual soil loss for existing and construction site conditions without controls, determined using the Universal Soil Loss Equation (USLE).

   b. Surface stabilization control measures required to keep the average annual soil loss for construction site conditions less than 15 tons/acre/year (using the USLE). The maximum allowable exposure for unprotected soil is 90 days. Specifications for the following temporary onsite erosion control measures from the Alabama Highway Department Standard Specifications shall apply:

Riprap	Section 610
Ground preparation and fertilizers for erosion control	Section 651
Seeding	Section 652
Sprigging	Section 653
Solid sodding	Section 654
Mulching	Section 656
Grassy mulch	Section 657
Hydro-seeding and mulching	Section 658
Erosion control netting	Section 659

c. Seeding and/or sodding requirements for all exposed areas during construction activities.

d. Schedule or sequence of operation clearing and/or grading, timing of structural installations, duration of soil exposure (the maximum allowable exposure duration is 3 months), and critical area stabilization. Anticipated completion dates for critical area stabilization, paving, seeding, and mulching or sodding are to be indicated.

e. Sediment control structures in sufficient detail to facilitate their installation. Appropriate design calculations are required for each structure.

f. General notes for erosion and sediment control identifying important details for implementation of the plan.

Procedures for the hydrologic and hydraulic design of a stormwater management system shall be in accordance with the procedures identified in this manual unless otherwise approved by the Engineering Department of Montgomery.

SECTION 5.0 OBTAINING A CERTIFICATE OF OCCUPANCY

Periodic inspection will be conducted by the Engineering Department throughout the construction phase of a stormwater management project. The schedule of these inspections will be established by the Engineering Department in accordance with the complexity of the project.

Upon the completion of construction, a Certificate of Occupancy must be approved according to the requirements of Item 4 in Article II of the Montgomery Zoning Ordinance. A schematic diagram of the final inspection process is shown on Figure 3-3.

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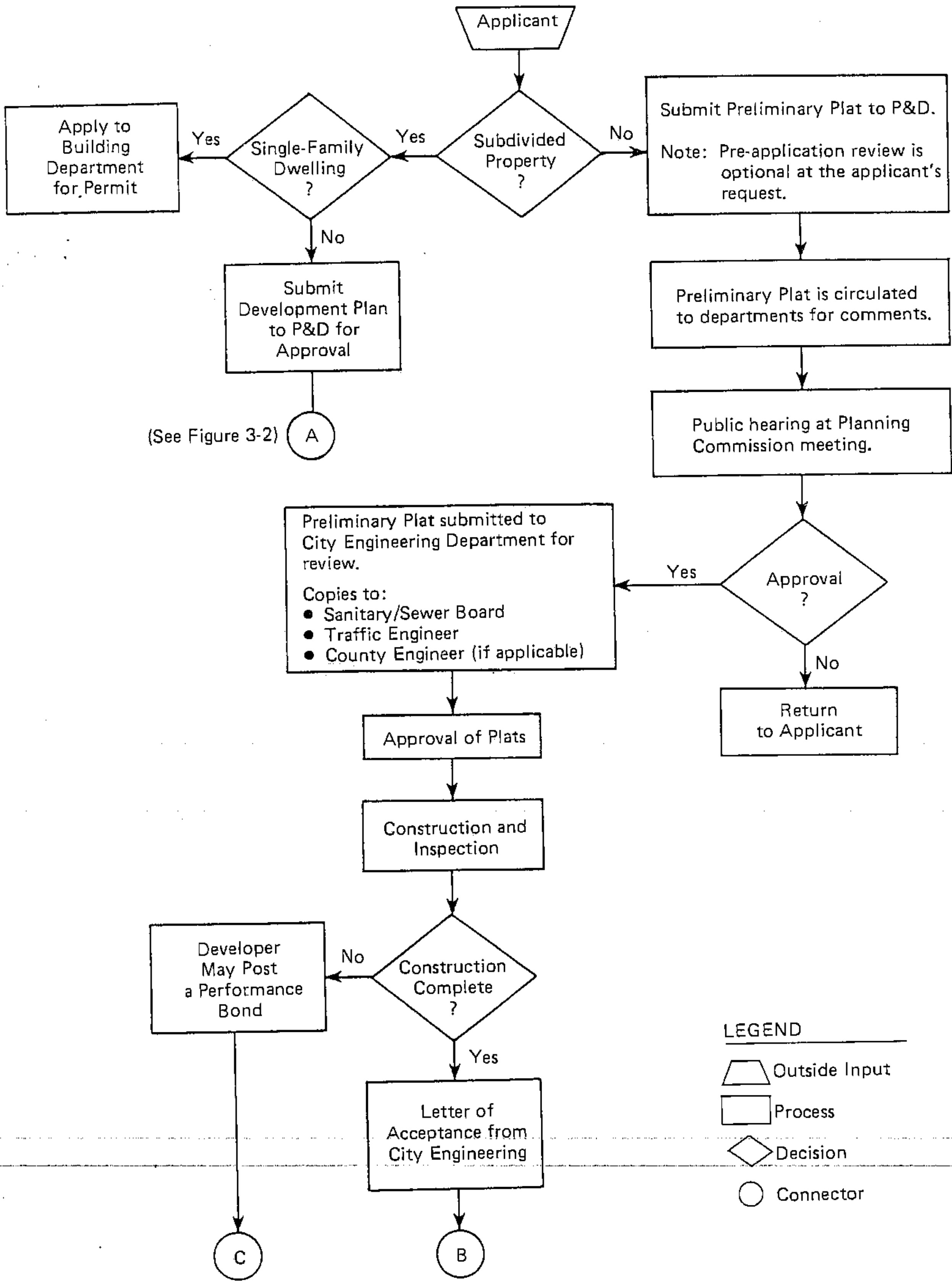
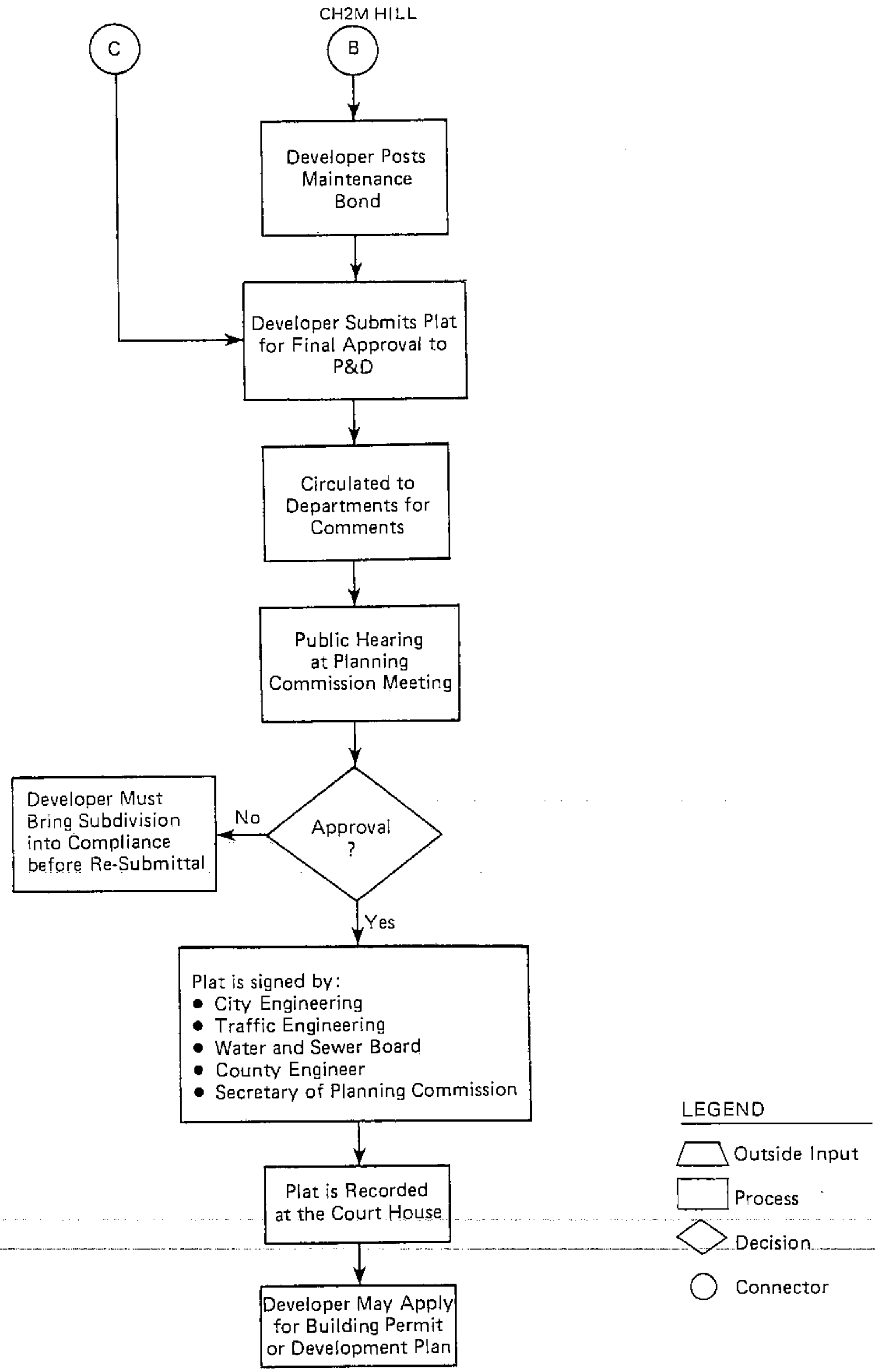


Note: P&D = Planning and Development Department

FIGURE 3-1. Flow chart for procedures required by the Subdivision Regulations and Zoning Ordinance.



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Note: P&D = Planning and Development Department

FIGURE 3-1. Flow chart for procedures required by the Subdivision Regulations and Zoning Ordinance. (continued)

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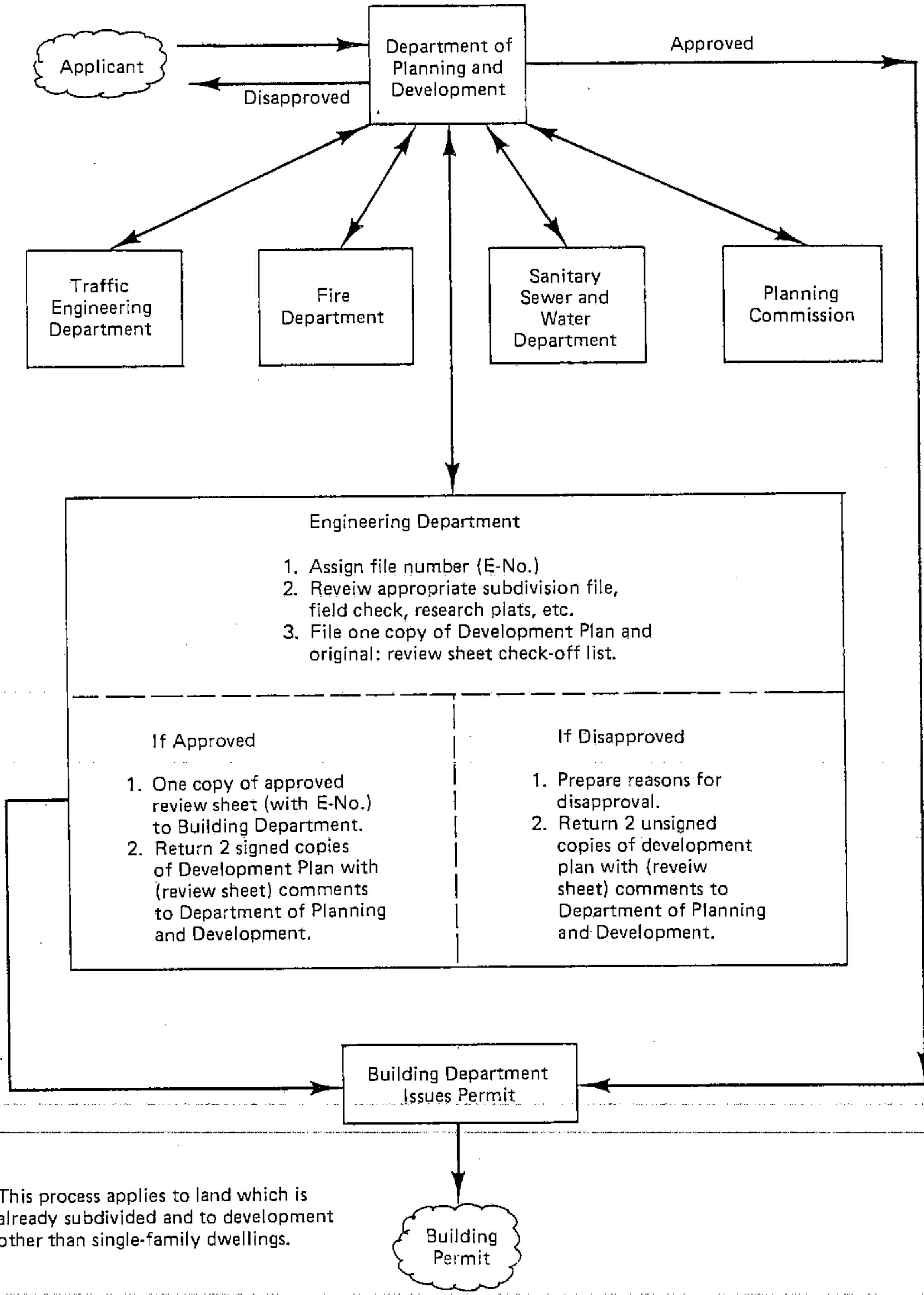

Note: This process applies to land which is already subdivided and to development other than single-family dwellings.

FIGURE 3-2. Pre-construction review process.

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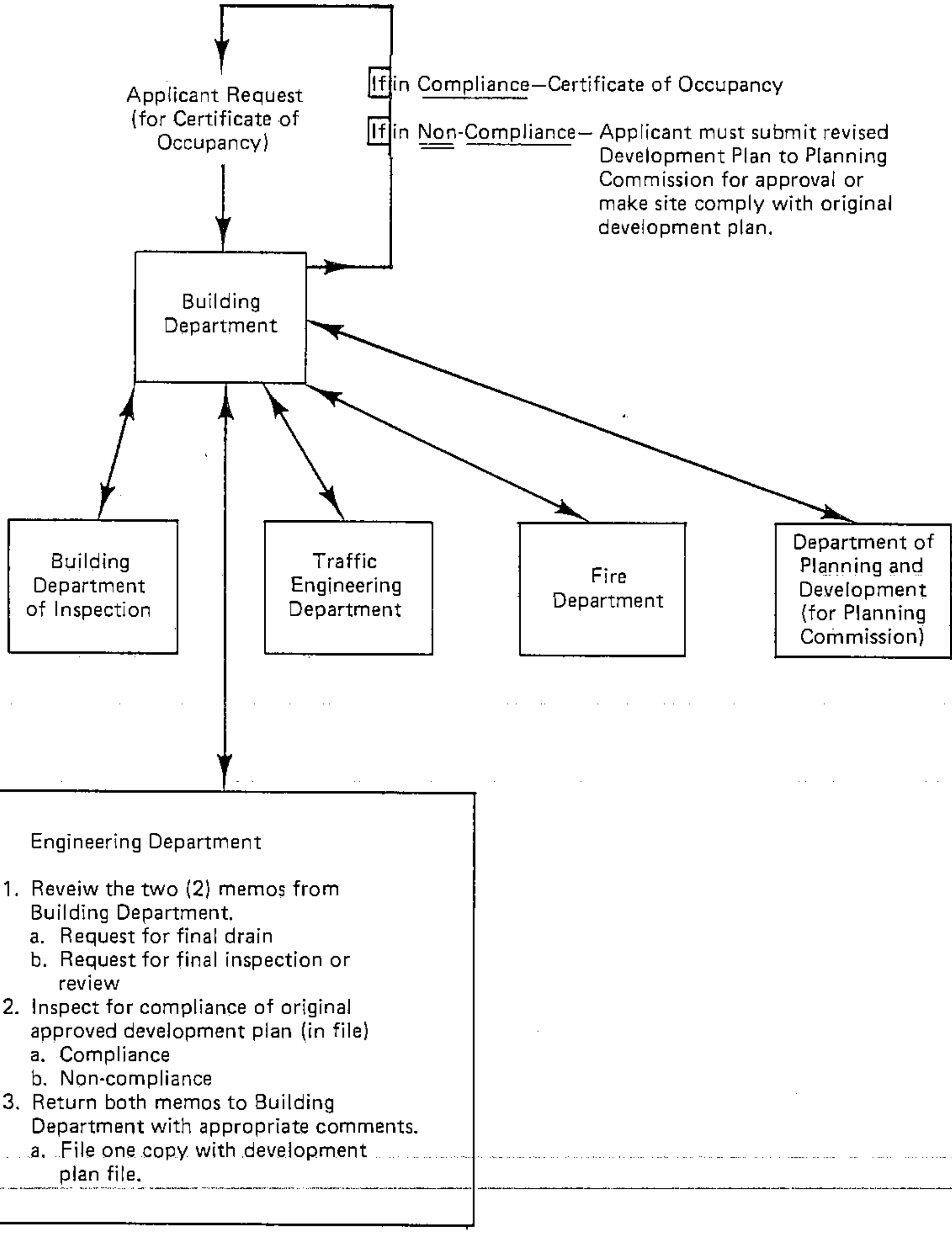


FIGURE 3-3. Final inspection request.

SECTION 1.0 INTRODUCTION

This chapter presents published hydrologic data for the Montgomery area. The published data, which are required for the design tools in subsequent chapters, include precipitation depth-duration-frequency relationships, Soil Conservation Service (SCS) soil data, existing and zoned land use conditions, topographic information, major watershed boundaries, and observed streamflow data. This chapter also offers guidance for collecting site-specific field data. A summary of addresses for locating published or readily available hydrologic data is presented in Table 4-1.

SECTION 2.0 PRECIPITATION

2.1 Data Collection

In the Montgomery area, the official climatological data are collected by the National Weather Service Office (WSO) from a station located on the north side of Dannelly Field. Dannelly Field is the municipal airport for Montgomery, and is located approximately 6 airline miles SSW of the downtown bend in the Alabama River. Observations at Dannelly Field began on February 1, 1944. Prior to this time, the weather station was located at Gunter Field and six different locations in the City of Montgomery. All official weather statements and data for Montgomery presently come through the WSO at Dannelly Field.

The climatological data collected at the local WSO are evaluated and summarized on a daily, monthly, and annual basis in publications available to the general public from the WSO at Dannelly Field or from the U.S. Department of Commerce, National Climatic Center, Federal Building, Asheville, North Carolina 28801. There is a small charge for these publications.

2.2 Climatological Characteristics

The monthly and annual precipitation averages from 1940 to 1979 are given in the "1979 Local Climatological Data, Annual Summary with Comparative Data" (U.S. Department of Commerce, 1979) and are shown in Table 4-2. The average annual precipitation depth for that period of time was 50.95 inches. A maximum annual precipitation depth of 72.98 inches occurred in 1975. A minimum annual precipitation depth of 26.82 inches was recorded in 1954. Average monthly precipitation depths vary from a high of 6.17 inches in March to 2.29 inches in October as shown on Figure 4-1.

From late June through the first half of August, nearly all precipitation is from local, mostly afternoon, thundershowers and there are apt to be considerable differences in day-to-day amounts of rainfall in different parts of the Montgomery area. In late August and in September, summer conditions of temperature and humidity persist as air continues to drift in from the Gulf, but local thundershowers become less frequent because of the shortening of the days and the decrease in heat received from the sun. As this later summer season progresses, the local heat thundershowers give way to thundershowers which occur ahead of slight pre-autumn drops in temperature, and to occasional general rains associated with storms on the Gulf. Rains during October are frequently showers or thundershowers occurring ahead of temperature drops, which become more frequent and more pronounced as winter approaches. The same is largely true of November. All types and intensities of rain, excepting the heat thundershowers of summer, may occur at any time from December through March or early April. Floods in the rivers are correspondingly more frequent during this period. Rain from late April through early June is typically in the form of showers or thundershowers occurring in advance of approaching cool waves, which become weaker and less frequent as summer approaches. It is during this spring season, and during the late summer and early autumn as noted above, that droughts sometimes occur.

2.3 Frequency Data

Precipitation frequency data which are presented in this manual were obtained from two publications:

1. Hershfield, D. M., 1961. "Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 years." Technical Paper No. 40 (TP-40), Weather Bureau, U.S. Department of Commerce, Washington, D.C.

2. Frederick, R. H., et al., 1977. "Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States." NOAA Technical Memorandum NWS HYDRO-35, National Weather Service, NOAA, U.S. Department of Commerce, Silver Springs, MD.

Since 1961, TP-40 has been the standard for precipitation-frequency values for duration from 5 minutes to 24 hours over the eastern United States. For durations of less than 1 hour, HYDRO-35 provides new 5- to 60-minute precipitation frequencies for the 37 states from North Dakota to Texas and eastward. These are presented in the form of maps for the 5-, 15-, and 60-minute durations at the 2- and 100-year return periods, together with equations and nomographs for intraduration and intrareturn period interpolations.

HYDRO-35 is the latest in the precipitation-frequency literature for the United States that began in the 1930's when David L. Yarnell (1935) first published generalized precipitation-frequency maps for durations of 5 minutes to 24 hours at return periods of 2 to 100 years. Precipitation intensity-duration-frequency curves were derived for this manual using HYDRO-35 data for storm durations of 60 minutes or less and TP-40 for storm durations of 24 hours or less. On the basis of data from Hydro-35 and TP-40, 2-year and 100-year precipitation depths for durations of 24 hours, 60-minute, 15-minute, and 5-minute depths for the 5-, 10-, 25-, and 50-year return periods were calculated using a mathematical solution to the Gumbel extreme value equations for a partial duration series. Precipitation depths for intermediate durations of 10 and 30 minutes were calculated using empirical relationships published in HYDRO-35, while depths for intermediate durations of 2, 3, 6, and 12 hours were calculated using empirical relationships published in TP-40. The intensity-duration-frequency curves derived by this procedure for the City of Montgomery are presented on Figure 4-2. The depth-duration-frequency data for Montgomery are inserted on Figure 4-2. These precipitation data will be utilized as input to the design procedures presented in Chapter 5.

SECTION 3.0 SOILS

General soil characteristics must be identified and quantified for use with the hydrologic design tools presented in subsequent chapters of Part II. The soil characteristics play a primary role in the analysis to determine what portion of the rainfall becomes runoff and how much soil loss can be expected during storm conditions. The data for this analysis are summarized in this section.

3.1 Soil Runoff

Rainfall excess is that portion of total precipitation which becomes stormwater runoff after a storm event. As will be discussed in Chapter 5, infiltration is one of the major abstractions which determines the quantity of rainfall excess for a particular watershed. Since infiltration rates vary with different soil series, identification of specific soil types is a prerequisite to hydrologic analysis.

The SCS has classified more than 4,000 soil series into four hydrologic soil groups denoted by the letters A, B, C, and D (defined in detail in Chapter 5). Of the soil series classified by SCS, the 49 found in Montgomery County are listed in Table 4-3. Most of the soils in Montgomery County have moderately low to high runoff potential since the corresponding soil series have moderate to very slow infiltration rates. Detailed soil maps for delineating hydrologic soil groups are available in the "Soil Survey, Montgomery County, Alabama," SCS, Series 1957, No. 7, issued September 1960. An example of a typical hydrologic soil map for a drainage study is shown as Figure 3-4 from the "West End Ditch Drainage Study" (CH2M HILL, 1979). Although colors are not necessary to denote the soil groups, some form of shading would be effective in facilitating data interpretation.

Of the nine general soil associations found in Montgomery County, five occur in the City of Montgomery. These associations, shown on Figure 4-4 and summarized in Table 4-4, give only a general idea of the soil groups found in different parts of the City. This gives a broad view of the soil types, which may serve as a guide to obtaining site-specific data from detailed soil maps.

3.2 Soil Erodibility

Soil loss through erosion is a problem in the Montgomery area. Methods for estimating sediment yield and controlling erosion will be discussed in Chapters 6 and 9, respectively. The tendency for a soil series to erode when exposed to surface runoff must be determined so that remedial measures, if necessary, may be taken to control soil loss through best management techniques. To aid the engineer in establishing erosion characteristics for particular soil series found in Montgomery, the erosion factor, K, for use in the Universal Soil Loss Equation, and the soil-loss tolerance factor, T, have been summarized in Table 4-5. The primary method for estimating gross erosion will be discussed in Chapter 6.

SECTION 4.0 LAND USE AND TOPOGRAPHY

Existing and ultimate land use combine with soil types to define runoff coefficients for different hydrologic analysis methods. Problems with existing drainage facilities can be identified using existing land use patterns. Design of drainage structures which are to adequately carry anticipated runoff from future developments must be based on a hydrologic analysis that includes assumed ultimate land use patterns.

4.1 Existing Land Use

The existing land use for a large watershed can be determined by using existing aerial photographs, shot in March 1974, which are available from the City Engineering Department. Consequently, additional field visits are typically necessary to verify the land uses shown and to determine any land use changes since the date of photography. Each mylar covers a quarter-section area at a scale of 1 inch = 100 feet. The identified land use patterns for a watershed should be identified on a map depicting the study area. An example of an existing land use map, from the "West End Ditch Drainage Study" (CH2M HILL, 1979), is shown as Figure 4-5. General land use classifications typically used in hydrologic analyses are discussed in Chapter 5.

4.2 Ultimate Land Use

Ultimate land use is not as easily defined as existing land use since some assumptions must be made regarding future growth. The primary source of information is the latest revised zoning map for the City of Montgomery. The City Planning Commission, local Chamber of Commerce, and private developers frequently have useful insights on the future growth of Montgomery. The Zoning Ordinance for the City of Montgomery defines the various zoning designations and aids the engineer in projecting the effects of future growth on stormwater runoff. For each zoned district, various classes of structures are either permitted or prohibited, and required lot areas are given along with parking requirements. These, along with other data, are summarized in the Ordinance. Using these data, informed estimates can be made on the future conditions.

4.3 Drainage Topography

The City of Montgomery is located in a gently rolling area of central Alabama with no local topographic features that appreciably influence weather and climate. The highest ground elevation occurs in the business district of Montgomery. The terrain gently slopes down in all directions away from the central section of the City.

The drainage area for the City of Montgomery can be divided into 24 urban watersheds, as delineated on Figure 4-6 in Appendix D. A list of these major urban watersheds is presented in Table 4-6. For the purposes of a drainage study, each of these urban watersheds should be divided into appropriate subbasins for the purpose of conducting a hydrologic analysis. An example subbasin map for the West End Ditch urban watershed is presented as Figure 4-7 from "West End Ditch Drainage Study" (CH2M HILL, 1979).

Although U.S.G.S. quadrangle maps are available for the Montgomery area, their use in determining watershed boundaries is somewhat limited in some areas due to the flat lay of the land. However, quad maps are useful in preliminary boundary determinations. The following quads are available for the Montgomery area:

- Montgomery, Alabama 7½'
  1958

- Mount Meigs, Alabama 7½'
  1958

- Montgomery South, Alabama 15'
  1958 Photorevisited 1972

- Montgomery North, Alabama 15'
  1950 Photorevisited 1972

Some 2-foot contour interval topographic maps (1 inch = 100 feet) are available from the City Engineering Department. These maps were prepared in February 1968 using photogrammetric methods (date of photography--March 1963). Although these topographics are more detailed and more current than U.S.G.S. quads, field verification work is still necessary when delineating the watershed and/or subbasin boundaries.

SECTION 5.0 STREAMFLOW

As described in Chapter 1, interior drainage for the City of Montgomery is accomplished in part by a network of natural and concrete-lined ditches and sloughs. These combine to form larger creeks and streams prior to final discharge to either the Tallapoosa or Alabama Rivers. Streamflow gauging stations, maintained by the U.S.G.S., Water Resources Division, are or have been located at the sites listed in Table 4-7. An example of a typical U.S.G.S. streamflow record summary is shown in Table 4-8 for water year October 1977 to September 1978 on Catoma Creek near Montgomery. A flood hydrograph record is published each year by the U.S.G.S. for Hannon Slough at Bear School. An example flood hydrograph record for Hannon Slough at Bear School during 1978 is shown in Table 4-9. Historical streamflow data can be used as a check to determine if the computed values using other methods are reasonable. Streamflow data can also be used to calibrate computer hydrologic models. Information regarding gauging station data may be obtained from the U.S. Geological Survey, Water Resources Division, Montgomery, Alabama.

During periods of intense rainfall resulting in severe flooding, the U.S.G.S. records and documents high water marks along major sloughs, ditches, and streams in the Montgomery area. One event of particular interest is the flood which occurred on May 9, 1978. Following this event, seedline elevations were determined along most major drainageways in Montgomery. The results of this collected data, along with photographs of the flood, are summarized in the U.S.G.S. Report No. 79-218, "Flood of May 9, 1978, Montgomery, Alabama." Using a technique described in Chapter 5, historical high water marks can be used to provide an order-of-magnitude check on pre-development peak flows determined by other methods.

SECTION 6.0 FIELD DATA ACQUISITION

Gathering accurate stormwater flow characterization data, both quantity and quality, is not a simple task. Information on rainfall, soil groups, land use, topography, etc., must be tabulated and summarized for use during any hydrologic analysis. This section discusses a number of data collecting techniques and contains examples of data worksheets and summary sheets. Since they are presented for illustration purposes only, the example data sheets could be revised to reflect the specific needs of individual engineers.

6.1 Precipitation Data

As discussed earlier in this chapter, precipitation data are available through the local office of the WSO at Dannelly Field, TP-40, and HYDRO-35. Most manual hydrologic analysis methods utilize the intensity-duration-frequency relationships as shown on Figure 4-2 or depth-duration-frequency relationships inserted on Figure 4-2. In some cases, the precipitation data on Figure 4-2 are adequate, while in other cases, the design storm distribution with respect to time must be developed for the entire storm duration. A discussion of procedures to develop design storm hyetographs is presented in Chapter 5.

6.2 Watershed Data

Data which describe the physical features of a watershed should be organized in a form which may be used by the engineer. Data organized in a table can reflect general overall information for the different subbasin in a watershed; i.e., the curve numbers for one section of the watershed may be significantly less than for other sections, indicating that subbasin yields may vary considerably within the watershed.

6.2.1 Drainage Area. The contributing area along with major subbasins must be delineated using the best available topographic maps for the area along with field verification. Planimetric or other procedures are used to determine the contributing area (acres or square miles) as required by the analysis method used. Since land area is the primary physical feature affecting drainage, it is listed in two of the example worksheets shown in this chapter.

6.2.2 Runoff Curve Number. The runoff curve number (CN) is an SCS parameter used to estimate the quantity of runoff from a watershed. Chapter 5 discusses the method for calculating CN for any watershed. When a particular drainage area has a wide range of hydrologic soil groups, land uses, and vegetative covers, a composite curve number for the watershed can be calculated. To calculate this value, the following general procedures can be used:

1. Delineate the different hydrologic soil groups in the area.

2. Determine the existing and ultimate land uses for the watershed.

3. Determine the distribution of hydrologic soil groups about the different land use areas.

4. Calculate the composite CN.

Typical summary sheets for the above data are presented in Tables 4-10 and 4-11. An example CN calculation is presented in Table 4-12.

It is convenient to summarize watershed data on one sheet so that the person evaluating the hydrologic characteristics of the watershed can make a quick check and comparison of his results. At times, unreasonable results can be identified by examining this summary sheet as shown in Table 4-13. Table 4-14 tabulates the final stream cross section data (to be discussed next), existing and ultimate time of concentration, the CN, and the average basin slope.

6.2.3 Conveyance Control Structures. The hydrologic response of a watershed is related to the routing characteristics of the ditches, sloughs, creeks, and culverts which comprise the stormwater conveyance system. Therefore, a physical inventory of drainage facilities is necessary to estimate the hydraulic conditions.

Characteristics which are used for most hydraulic analyses include the total channel cross section (including left and right flood plains), channel slope, Manning's "n" for the channel and flood plains and distance between surveyed cross sections. The channel cross section can be determined using stadia-azimuth from an established baseline along the channel. The cross section should extend into the flood plain enough to describe the estimated flow area during overbank flow conditions. The Corps of Engineers computer program HEC-2 will insert artificial levees at each end of the cross section if the survey terminates the cross section at an elevation below the computed water surface profile. This may lead to erroneous results. Baseline stadia-azimuth data can be converted to a station-elevation form referenced from the first point on the left end of the cross section. Channel cross sections are required on representative locations throughout the total reach. Three general types of cross sections can be identified: (1) where the channel changes in slope, cross sectional area, or channel or flood-plain roughness, (2) where levees begin or end, and (3) at bridges. It is recommended that cross sections be located at least every 800 feet.

It is necessary to collect field data on all major bridges and culverts along primary drainage paths. These structures, if undersized, may cause a backwater to occur during storm events. This can significantly affect the overall hydrologic response of the watershed. Bridge and culvert information required for a HEC-2 hydraulic analysis should include the following items:

1. Determine the channel cross sections downstream, upstream, and along the top of the structure.

2. Determine the dimensions of the orifice(s).

3. Measure the total width of all piers.

4. Determine the channel invert elevation at downstream and upstream limits of structure.

5. Measure the length of the structure (between upstream and downstream ends).

6. Determine the maximum low chord elevation or the elevation at which HEC-2 will begin pressure flow computations. This is typically the culvert crown or bridge beam bottom.

7. Determine the minimum top of road or the elevation at which HEC-2 will begin weir flow computations. Special care should be exercised in this instance if the bridge is perched, which means the approach road to the bridge is at or below the flood-plain ground level and only in the immediate area of the bridge does the road rise above the ground level to span the water course. A typical flood flow situation with this type of bridge includes low flow under the bridge and overbank flow around the bridge.

8. Describe the shape of all piers and wingwalls. This can be used to fine-tune the bridge coefficients.

9. Describe the anticipated weir condition to help determine weir coefficient.

10. If a culvert is present, determine if it is operating under inlet or outlet control, as defined in Chapter 7.

Prior to collecting this field data, it is a good idea to know what flow regime will be the most important to analyze. The evaluator can then concentrate on the specific information necessary for that analysis. For example, if culverts operate under pressure and weir flow conditions, field activities can concentrate on defining the allowable headwater. Since HEC-2 does not take into account the orifice shape for pressure flow, other factors must be considered. For example, if the analysis concerns culverts flowing partially full, a detailed description of the shape of the opening is necessary. More detailed information on channel cross section and bridge data is given in two Hydrologic Engineering Center publications:

1. "HEC-2 Water Surface Profiles," HEC, U.S. Army Corps of Engineers (1976).

2. "Application of the HEC-2 Bridge Routines," HEC, U.S. Army Corps of Engineers (1974).

All elevations used in the cross section and culvert data collection shall be referenced to the National Geodetic Vertical Datum of 1929 (NGVD). Vertical benchmarks have been located throughout the City by U.S.G.S., the City of Montgomery Engineering Department, and the U.S. Army Corps of Engineers. A listing of the existing benchmarks is available through the office of the City Engineer.

Manning's n is used in hydraulic calculations to describe the hydraulic capacity of a channel or flood plain. The selection of n factors is based on engineering judgment and field observations of streams and flood-plain areas. A discussion of Manning's n is presented in Chapter 7. Manning's n values for conveyance system design in Montgomery are presented in Table 7-2. It should be noted that proper selection of left and right flood-plain n values is as important as the value for the main channel. In some instances, n may vary with respect to location along the cross section. The data collection and summary should reflect this change.

SECTION 7.0 REFERENCES

1. CH2M HILL, 1979. "West End Ditch Drainage Study," Engineering Report for the City of Montgomery, Alabama.

2. David Volkert and Assoc., Inc., 1976. "Montgomery Area Drainage Study, 1976 Summary," December.

3. Frederick, R. H., et al., 1977. "Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States," NOAA Technical Memorandum NWS HYDRO-35, National Weather Service, U.S. Department of Commerce, Silver Springs, MD.

4. Hershfield, D.M., 1961. "Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years." Technical Paper No. 40 (TP-40), Weather Bureau, U.S. Department of Commerce, Washington, D.C.

5. U.S. Army Corps of Engineers, 1976. "HEC-2 Water Surface Profiles, Users Manual," Hydrologic Engineering Center, Davis, CA.

6. U.S. Army Corps of Engineers, 1974. "Application of the HEC-2 Bridge Routines," Hydrologic Engineering Center, Davis, CA.

7. U.S. Department of Agriculture, Soil Conservation Service, 1960. "Soil Survey, Montgomery County, Alabama." Series 1957, No. 7, September.

8. U.S. Department of Agriculture, Soil Conservation Service, 1972. Hydrology, National Engineering Handbook, Section 4, Washington, D.C., NTIS No. PB-244 463.

9. U.S. Department of Agriculture, Soil Conservation Service, 1980. "Soils Reference No. 12-3," July.

10. U.S. Department of Agriculture, Soil Conservation Service, 1980. "Soils Reference No. 12-3, Supplement," December.

11. U.S. Department of Agriculture, Soil Conservation Service, 1975. "Urban Hydrology for Small Watersheds," SCS TR-55, Washington, D.C.

12. U.S. Department of Commerce, 1979, "Local Climatological Data, Annual Summary with Comparative Data, 1979," National Climatic Center, Federal Building, Ashville, NC.

13. U.S. Geological Survey, 1979a. "Water Resources Data for the State of Alabama, Water Year 1978."

14. U.S. Geological Survey, 1979b. "Flood of May 9, 1978, Montgomery, Alabama," Report No. 79-218 by G. H. Nelson and L. R. Bohman.

15. Yarnell, D. L., 1935. "Rainfall Intensity Frequency Data," Miscellaneous Publication No. 204, U.S. Department of Agriculture, Washington, D.C.

Table 4-1
ADDRESSES FOR OBTAINING VARIOUS TYPES
OF HYDROLOGIC DATA FOR
THE MONTGOMERY AREA

1. Alabama Soil and Water Conservation Committee
   1445 Federal Drive
   Montgomery, Alabama 36107

2. Agriculture Stabilization and Conservation Service
   Alabama State Office
   474 South Court Street
   Montgomery, Alabama 36104

3. Agriculture Stabilization and Conservation Service
   ASCS County Office
   4510 South Court Street
   Montgomery, Alabama 36105

4. Soil Conservation Service
   Area Office
   40A Gaylan Court
   Montgomery, Alabama 36109

5. U.S. Army Corps of Engineers
   Mobile District Office
   P.O. Box 2288
   Mobile, Alabama 36628
   Attention: SAMOP-S

6. U.S. Army Corps of Engineers
   Montgomery Installation
   3200 Brewbaker Avenue
   Montgomery, Alabama 36108

7. U.S. Department of the Interior
   U.S. Geological Survey
   Water Resources Division
   1765 Highland Avenue
   Montgomery, Alabama 36107

8. National Weather Service
   474 South Court Street
   Montgomery, Alabama 36104

9. National Weather Service
   Dannelly Field Station (Index No. 5550, Division 06)
   Dannelly Field Municipal Airport
   Montgomery, Alabama 36108

10. U.S. Department of Commerce
    National Climatic Center
    Federal Building
    Asheville, North Carolina 28801

Table 4-1--Continued

11. Geological Survey of Alabama
    P.O. Drawer O
    University, Alabama 35486

12. City Engineering Department
    City of Montgomery
    P.O. Box 1111
    Montgomery, Alabama 36192

13. City Planning and Zoning Department
    City of Montgomery
    P.O. Box 1111
    Montgomery, Alabama 36192

14. Montgomery Area Chamber of Commerce
    41 Commerce Street
    Montgomery, Alabama 36102

Table 4-2
MONTHLY AND ANNUAL PRECIPITATION DATA FOR
MONTGOMERY, ALABAMA, BETWEEN 1940 AND 1979

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1940	3.93	5.48	5.94	3.73	1.43	6.24	6.15	1.98	0.29	0.41	2.79	6.79	45.16
1941	3.03	3.01	4.84	1.96	0.68	5.02	8.04	2.94	4.35	1.10	1.34	10.32	46.63
1942	4.00	3.67	8.27	1.87	4.03	6.29	3.95	4.00	2.44	1.77	1.42	8.10	49.76
1943	4.57	1.43	12.92	3.35	3.07	3.29	3.03	4.53	2.12	1.05	3.49	2.77	45.62
1944[a]	4.92	6.24	10.27	10.34	1.96	2.52	7.37	8.75	3.15	0.82	5.40	3.11	64.85
1945	2.98	4.76	2.28	11.66	2.21	6.07	7.56	5.31	3.50	2.94	3.21	5.68	58.16
1946	7.10	3.39	9.13	2.05	7.84	9.89	7.12	5.66	4.86	2.22	3.75	2.28	65.29
1947	7.58	1.47	6.05	7.07	6.23	1.99	5.96	2.46	0.81	1.52	7.39	4.70	53.23
1948	3.92	4.53	8.53	2.04	2.55	3.84	5.11	4.82	3.29	1.89	21.32	3.31	65.15
1949	3.90	4.97	4.98	5.55	3.05	7.64	6.93	2.79	1.78	3.27	0.32	2.83	48.01
1950	2.21	3.44	5.04	2.89	2.85	2.40	7.92	3.06	8.83	0.83	1.04	4.33	44.84
1951	3.54	1.84	4.04	4.02	1.47	3.31	2.46	2.46	7.70	1.29	2.47	4.05	38.65
1952	2.80	4.03	5.51	3.87	5.92	1.14	1.58	4.34	2.01	1.00	1.14	7.18	40.52
1953	3.97	7.34	2.32	9.29	3.23	3.18	4.49	1.78	10.62	0.48	2.56	9.82	59.08
1954	0.72	3.40	4.67	2.06	1.48	1.55	3.38	1.68	0.44	1.56	1.85	4.03	26.82
1955	4.63	3.03	2.18	6.41	7.03	4.82	4.69	1.91	0.77	1.15	1.71	1.36	39.69
1956	1.79	5.07	8.69	2.03	1.31	3.69	8.92	3.53	10.55	5.76	1.10	8.88	61.32
1957	2.82	2.40	5.43	7.55	8.29	5.11	3.24	2.14	9.55	0.87	5.69	3.57	56.66
1958[a]	2.64	4.48	10.83	4.02	3.52	3.42	7.70	0.78	4.78	0.68	2.45	2.63	47.93
1959	3.41	4.47	4.82	4.01	6.53	1.91	3.30	1.38	4.37	9.06	1.08	2.46	46.80
1960	6.51	4.14	5.52	3.56	3.25	4.20	4.79	4.26	6.44	1.88	1.67	1.93	48.15
1961	2.18	13.38	10.41	4.22	1.80	5.10	3.78	6.92	2.08	0.08	3.52	11.35	64.82
1962	5.20	2.51	7.45	6.41	1.14	6.21	3.77	3.41	3.28	0.67	5.60	1.82	47.47
1963	7.14	3.30	3.28	1.36	3.06	4.46	3.60	2.84	3.90	0.02	3.22	5.34	41.52
1964	6.49	4.49	5.01	15.64	1.92	2.72	5.19	2.55	5.38	6.27	3.46	5.29	64.41
1965	6.10	6.37	8.08	0.88	1.33	2.76	4.99	3.98	6.47	1.14	1.34	3.57	47.01
1966	6.20	8.06	4.26	1.24	2.36	3.20	2.58	1.79	5.85	6.08	4.65	5.11	51.38
1967	2.77	4.46	2.00	1.05	5.48	3.69	7.07	2.67	3.75	4.38	2.58	8.51	48.41
1968	2.78	1.87	2.80	3.31	1.94	1.89	4.96	2.50	1.06	0.80	5.90	5.65	35.46
1969	1.85	3.64	4.73	1.82	4.97	1.75	3.24	5.06	6.06	1.60	0.73	3.88	39.33
1970	2.83	3.77	6.20	2.01	3.49	7.83	5.89	3.70	2.02	4.98	1.50	4.18	48.40
1971	2.53	6.61	10.77	3.81	3.73	3.18	4.94	3.22	5.86	0.29	2.27	6.24	53.45
1972	6.35	3.98	5.25	1.45	1.70	5.49	5.84	3.03	1.61	1.56	3.71	7.14	47.11
1973	3.78	4.62	9.84	7.74	3.97	5.37	7.87	2.73	4.07	0.46	4.31	3.70	58.46
1974	5.63	2.09	2.69	3.69	3.30	1.57	5.41	10.42	5.38	1.45	3.59	4.67	49.89
1975	6.75	7.80	7.26	6.63	4.39	4.76	7.58	4.18	8.56	6.42	4.01	4.64	72.98
1976	3.03	1.75	9.41	1.65	6.78	2.74	3.03	2.21	2.79	1.78	5.32	4.94	45.43
1977	4.87	3.19	7.16	1.53	1.62	1.82	6.62	3.55	5.64	2.60	2.78	3.10	44.48
1978	6.95	2.29	2.61	4.57	12.01	3.87	4.02	3.52	2.18	0.01	3.09	4.24	49.36
1979	5.74	7.73	4.23	10.15	5.35	0.33	5.16	1.54	5.25	0.60	4.10	2.84	53.02
Record Mean	4.71	5.04	6.17	4.63	3.71	3.99	4.87	3.88	3.56	2.29	3.33	4.77	50.95

Table 4-3
MONTGOMERY COUNTY SOIL SERIES WITH
CORRESPONDING HYDROLOGIC GROUP CLASSIFICATION

Series Name	Hydrologic Group	Series Name	Hydrologic Group
Altavista	C	Leaf	D
Amite	B	Leeper	D
Augusta	C	Mantachie	C
Bibb	C	Myatt	D
Boswell	D	Ochlockonee	B
Bowie	B	Oktibbeha	D
Byars	D	Pheba	C
Cahaba	B	Prentiss	C
Catalpa	C	Rains	B/D[b]
Chastain	D	Roanoke	D
Chewacla	C	Ruston	B
Congaree	B	Sawyer	C
Cuthbert	C	Shubuta	C
Eutaw	D	Stough	C
Flint	C[a]	Sumter	C
Geiger	D	Susquehanna	D
Houston	D	Tuscumbia	D
Huckabee	A[a]	Una	D
Independence	A[a]	Vaiden	D
Iuka	C	Waugh	C
Izagora	C	Wehadkee	D
Kaufman	D	West Point	C
Kipling	D	Wickham	B
Klej	B	Wilcox	D
Lakeland	A		

Sources:
1. SCS NEH-4 (1972)
2. SCS Soil Survey, Montgomery County, Alabama (1960)
3. SCS Soils Reference No. 12-3 (July 1980)
4. SCS Soils Reference No. 12-3, Supplement (December 1980)

[a] Hydrologic group classification has not been determined. Values shown were selected based on the rate of infiltration descriptions found in the SCS Soil Survey for Montgomery County.

[b] Two soil groups indicate the drained/undrained situation.

Table 4-4
GENERAL SOIL ASSOCIATIONS FOUND
IN THE CITY OF MONTGOMERY

Soil Associations	Description
1	NEARLY LEVEL, WELL-DRAINED TO SOMEWHAT POORLY DRAINED SOILS ON FIRST BOTTOMS: CONGAREE-CHEWACLA-WEHADKEE This soil association makes up less than one percent of the City of Montgomery and occurs at the northeast corner of the City limits, along the Tallapoosa River. County-wide, it extends from the northeastern corner of the county to a point north of Montgomery. It consists of nearly level, well-drained areas that are dissected by poorly drained sloughs and drainageways. The Congaree soils are well drained, and the Wehadkee soil is poorly drained. About half of the association consists of Congaree soils. The soils in this association have a surface soil that is generally grayish-brown to pale brown silt loam or silty clay loam. The subsoil is dark yellowish-brown to gray silt loam to silty clay loam. Congaree fine sandy loam occurs in small areas, mainly adjacent to stream banks.
2	LEVEL AND VERY GENTLY SLOPING, WELL-DRAINED TO POORLY DRAINED SOILS ON STREAM TERRACES: CAHABA-WICKHAM-ROANOKE. This soil association occurs primarily in a belt that is about 2 to 5 miles wide. This belt extends eastward from the junction of Catoma Creek and the Alabama River almost to Mount Meigs. One small area is about 3 miles northeast of Cantelou. This association covers about 12.5 percent of the City of Montgomery. This association consists mainly of broad, flat, well-drained areas that are dissected by poorly drained sloughs. Adjacent to the sloughs are moderately well-drained gentle slopes. The Cahaba soils are at slightly higher elevations than the other soils in the association. Much of this association is underlain by thick beds of gravel at depths of 4 to 10 feet. A number of gravel pits are commercially operated.

Table 4-4--Continued

Soil Associations	Description
	The well-drained soils normally have a grayish-brown fine sandy loam to silt loam surface soil and a yellowish-brown to yellowish-red subsoil. The poorly drained soils are generally gray to dark gray in the surface soil and are gray and mottled in the subsoil.
	Included with the major soils in this association are the Izagora, Byars, Myatt, Altavista, Waugh, Independence, and Huckabee soils. These included soils make up about 30 percent of the acreage. The Independence and Huckabee soils are excessively drained. The Cahaba and Wickham soils are well drained, and the Byars, Myatt, and Roanoke soils are poorly drained.
3	Association 3 generally not found in the City of Montgomery.
4	LEVEL TO SLOPING, WELL-DRAINED SOILS ON HIGH STREAM TERRACES: AMITE-CAHABA.
	This soil association consists of one large area: a belt about 4 miles wide that extends from Montgomery eastward almost to the county line. It makes up about 34.6 percent of the City. Broad, level to gently sloping areas characterize most of this association. Two fairly large areas, however, have nearly level ridgetops with sloping to steep, highly dissected side slopes. One of these areas is near Montgomery, and the other is near Merry. The elevation of these two areas is 15 to 20 feet higher than that of the rest of the association.
	The soils in this association have a surface soil that is dominantly grayish-brown to dark grayish-brown fine red in color, and from sandy clay loam to sandy clay in texture. West of Merry, the Cahaba soils are finer textured and are at higher elevations than are the Cahaba soils in soil association 2.
	The poorly drained soils along the narrow drainageways consist of sandy alluvium, which is variable in texture and color. These soils receive seepage water and overflow from surrounding soils. Iuka soils, local alluvium phases, occur in slight depressions and along drainageways in poorly drained areas. These soils have a dark-brown surface soil and a mottled subsoil.

Table 4-4--Continued

Soil Associations	Description
5	NEARLY LEVEL TO SLOPING SOILS ON THE UPLAND AND ASSOCIATED FIRST BOTTOMS AND LOCAL ALLUVIUM OF THE PRAIRIE SECTION: SUMTER-OKTIBBEHA-LEEPER. This soil association consists of four areas that make up about 34.2 percent of the City. These areas occur mainly in a wide belt that runs through the central part of the county, and are located in the southeast and southwest sections of the City. They are separated from each other by wide areas of alluvial soils and soils on low terraces. The largest acreage is in the vicinity of Pike Road, Snowdoun, and Dannelly Field Municipal Airport. This association consists mainly of fairly wide, nearly level to very gently sloping ridgetops, gently sloping to sloping side slopes, and fairly wide strips of alluvium along the drainageways. The soils are acid to alkaline, and normally have a dark-brown to very dark brown clay to silty clay surface soil. In some of the more eroded areas of Sumter soils, the surface soil may be gray or light gray. Vaiden, Houston, West Point, and Catalpa soils make up 10 percent of the association. Other minor soils are the Eutaw and the Tuscumbia. The Catalpa soil occupies floodplains and is the best drained soil on the first bottoms in the prairie section. The West Point soils are at the head of drainageways and, to some extent, at the base of slopes. Adjacent to the Vaiden soils are the Eutaw soils, which are the poorest drained members of the Oktibbeha-Vaiden-Eutaw catena. The Eutaw soils are at a somewhat lower elevation than the Vaiden soils and are on smoother relief. The Sumter, Oktibbeha, Vaiden, and Houston soils occur on the ridgetops and side slopes. They have a clayey subsoil that is red in the Oktibbeha soils, light gray in the Sumter soils, yellowish brown in the Vaiden soils, and dark olive gray in the Houston soil.
6	NEARLY LEVEL, MODERATELY WELL-DRAINED TO SOMEWHAT POORLY DRAINED SOILS ON LOW STREAM TERRACES AND FIRST BOTTOMS: IZAGORA-GEIGER-UNA.

Table 4-4--Continued

Soil Associations	Description
	This soil association makes up about 17.8 percent of the City. A large area extends southward from the Alabama River almost across the county. It extends in two prongs--one up Catoma Creek and its tributaries, and the other along Pintlalla Creek and its tributaries. This association consists of fairly wide strips of moderately well-drained to somewhat poorly drained soils on low stream terraces and flood plains. Most of the acreage in the county that has retarded drainage occurs in this association. The Kipling, Leeper, and Tuscumbia are the most extensive minor soils. The soils in this association have a dominantly grayish-brown fine sandy loam or silty clay surface soil. Their subsoil is mottled sandy clay loam to clay. The Izagora and Kipling soils are the better drained soils.
7,8,9	Associations 7, 8, and 9 generally not found in the City of Montgomery.

Source: USDA, SCS Soil Survey, Montgomery County, Alabama (1960).

Table 4-5
MONTGOMERY COUNTY SOIL SERIES WITH
CORRESPONDING K AND T EROSION FACTORS

Series Name	Horizon Depth (inches)	Erosion Factors K	Erosion Factors T	Series Name	Horizon Depth (inches)	Erosion Factors K	Erosion Factors T
Altavista	0-12	.20	4	Amite (Red Bay)	0-6	.10-.15	5
	12-42	.24			6-20	.15	
					20-72	.17	
Augusta	0-9	.15	4	Bibb	0-37	.20	5
	9-70	.24			37-60	.37	
Boswell	0-5	.37-.43	5	Bowie (Dothan)	0-13	.15-.20	4
	5-70	.32			13-60	.28	
Byars	0-13	.17	5	Cahaba	0-9	.20	4
					9-53	.28	
					53-80	.24	
Catalpa	0-60	.28	5	Chastain	0-10	.32	5
	6-60	.28			10-72	.37	
Chewacla	0-8	.24-.28	4	Congaree	0-8	.37	5
	8-24	.32			8-80	.37	
	24-34	.28					
	34-58	.32					
Cuthbert (Luverne)	0-7	.37	3	Eutaw	0-9	.32	4
	7-80	.28			9-80	.28	
Flint (Annemaine)	0-9	.37-.43	4	Geiger (Garner)	0-4	.32	5
	9-49	.37			4-65	.32	
	49-80	.32					

Table 4-5--Continued

Series Name	Horizon Depth (inches)	Erosion Factors K	Erosion Factors T	Series Name	Horizon Depth (inches)	Erosion Factors K	Erosion Factors T
Houston	0-10	.37	4	Huckabee	0-17	.17	5
	10-72	.32		(Bigbee)	17-80	.17	
Independence	0-17	.17	5	Iuka	0-13	.17-.24	5
(Bigbee)	17-80	.17			13-22	.28	
					22-60	.20	
Izagora	0-11	.28-.37	4-3	Kaufman	0-6	.32	5
	11-91	.32			6-80	.32	
Kipling	0-3	.32-.37	4	Klej	0-28	.17	5
	3-72	.32		(Ocilla)	28-67	.24	
Lakeland	0-43	.17	5	Leaf	0-9	.32	5
	43-80	--			9-72	.32	
Leeper	0-8	.28	5	Mantachie	0-11	.28	5
	8-50	.28			11-61	.28	
Myatt	0-10	.32	5	Ochlockonee	0-44	.20	5
	10-50	.28			44-72	.17	
	50-72	.24					
Oktibbeha	0-4	.32-.37	3	Pheba	0-8	.49	3
	4-70	.32			8-21	.49	
					21-60	.43	
Prentiss	0-26	.24	3	Rains	0-12	.17	5
	26-73	.24			12-40	.24	
					40-79	.28	

Table 4-5
MONTGOMERY COUNTY SOIL SERIES WITH
CORRESPONDING K AND T EROSION FACTORS

Series Name	Horizon Depth (inches)	Erosion Factors K	Erosion Factors T	Series Name	Horizon Depth (inches)	Erosion Factors K	Erosion Factors T
Altavista	0-12	.20	4	Amite (Red Bay)	0-6	.10-.15	5
	12-42	.24			6-20	.15	
					20-72	.17	
Augusta	0-9	.15	4	Bibb	0-37	.20	5
	9-70	.24			37-60	.37	
Boswell	0-5	.37-.43	5	Bowie (Dothan)	0-13	.15-.20	4
	5-70	.32			13-60	.28	
Byars	0-13	.17	5	Cahaba	0-9	.20	4
					9-53	.28	
					53-80	.24	
Catalpa	0-60	.28	5	Chastain	0-10	.32	5
	6-60	.28			10-72	.37	
Chewacla	0-8	.24-.28	4	Congaree	0-8	.37	5
	8-24	.32			8-80	.37	
	24-34	.28					
	34-58	.32					
Cuthbert (Luverne)	0-7	.37	3	Eutaw	0-9	.32	4
	7-80	.28			9-80	.28	
Flint (Annemaine)	0-9	.37-.43	4	Geiger (Garner)	0-4	.32	5
	9-49	.37			4-65	.32	
	49-80	.32					

Table 4-5--Continued

Series Name	Horizon Depth (inches)	Erosion Factors K	Erosion Factors T	Series Name	Horizon Depth (inches)	Erosion Factors K	Erosion Factors T
Houston	0-10	.37	4	Huckabee	0-17	.17	5
	10-72	.32		(Bigbee)	17-80	.17	
Independence	0-17	.17	5	Iuka	0-13	.17-.24	5
(Bigbee)	17-80	.17			13-22	.28	
					22-60	.20	
Izagora	0-11	.28-.37	4-3	Kaufman	0-6	.32	5
	11-91	.32			6-80	.32	
Kipling	0-3	.32-.37	4	Klej	0-28	.17	5
	3-72	.32		(Ocilla)	28-67	.24	
Lakeland	0-43	.17	5	Leaf	0-9	.32	5
	43-80	--			9-72	.32	
Leeper	0-8	.28	5	Mantachie	0-11	.28	5
	8-50	.28			11-61	.28	
Myatt	0-10	.32	5	Ochlockonee	0-44	.20	5
	10-50	.28			44-72	.17	
	50-72	.24					
Oktibbeha	0-4	.32-.37	3	Pheba	0-8	.49	3
	4-70	.32			8-21	.49	
					21-60	.43	
Prentiss	0-26	.24	3	Rains	0-12	.17	5
	26-73	.24			12-40	.24	
					40-79	.28	

Table 4-5--Continued

Series Name	Horizon Depth (inches)	Erosion Factors K	Erosion Factors T	Series Name	Horizon Depth (inches)	Erosion Factors K	Erosion Factors T
Roanoke	0-9	.24	5	Ruston	0-16	.28-.32	5
					16-41	.28	
					41-47	.32	
					47-80	.28	
Sawyer	0-5	.43	3	Shubuta	0-8	.37	5
	5-29	.37			8-70	.28	
	29-80	.32					
Stough	0-20	.28-.37	3	Sumter	0-10	.37	3
	20-68	.37			10-28	.37	
Susquehanna	0-5	.17-.43	3	Tuscumbia	0-4	.28	3
	5-77	.32			4-50	.28	
Una	0-6	.28	3	Vaiden	0-4	.37	4
	6-57	.28			4-80	.32	
Waugh (Angie	0-10	.37	3	Wehadkee	0-8	.24	5
	10-65	.32			8-40	.32	
West Point (Catalpa)	0-6	.28	5	Wickham	0-9	.20	5
	6-60	.28			9-40	.24	
Wilcox	0-5	.37	4				
	5-50	.32					
	50-57	.28					

Sources: 1. USDA, SCS Soils Reference No. 12-3 (July 1980).
2. USDA, SCS Soils Reference No. 12-3, Supplemnt (December 1980).

Table 4-6
MAJOR URBAN WATERSHEDS IN
MONTGOMERY, ALABAMA

1 - New Town Ditch	13 - Audubon Ditch
2 - Northwest Montgomery	14 - Hannon Slough
3 - Cypress Pond Ditch	15 - Seibles Road
4 - Civic Center	16 - Baldwin Slough
5 - Riverside-Maxwell	17 - Whites Slough
6 - West End Ditch	18 - Montgomery East Ditch
7 - Pineview Homes Ditch	19 - Eastdale Ditch
8 - Smiley Court	20 - Wares Ferry Ditch
9 - Genetta Ditch	21 - Three Mile Branch
10 - Caney Branch	22 - Kilby Ditch
11 - Cloverland Ditch	23 - Children's Zoo Ditch
12 - Beauvoir Gardens	24 - Chisholm Ditch

Source: David Volkert and Associates, Inc. (1976).

Table 4-7
U.S. GEOLOGICAL SURVEY GAUGING STATIONS
IN THE VICINITY OF MONTGOMERY, ALABAMA

Location	U.S.G.S. Station Number	Period of Record	Drainage Area (sq mi)	Remarks
1. Tallapoosa River near Montgomery	02419890	Oct 1972-Current	4,600	Gauge heights only
2. Alabama River at Montgomery	02419988	Dec 1980-Current	15,000	Gauge heights only
3. Alabama River near Montgomery	02420000	Oct 1927-Current	15,100	Discharge records
4. Catoma Creek near Montgomery	02421000	Jun 1952-Sept 1971 and Oct 1974-Current	298	Dicharge records
5. Hannon Slough (at Bear School) at Montgomery	02420987	Oct 1974-Sept 1976 and Oct 1977-Current	1.35	Flood hydrograph only

Source: Geological Survey (1979a).

MOBILE RIVER BASIN

02421000 CATOMA CREEK NEAR MONTGOMERY, AL

LOCATION.--Lat 32°18'26", long 86°17'58", in center sec. 6, T. 15 N., R. 18 E., Montgomery County, Hydrologic Unit 03150201, on right bank on downstream side of bridge on U.S. Highway 331, 5 mi (8 km) south of Montgomery.

DRAINAGE AREA.--298 mi^2 (772 km^2).

WATER-DISCHARGE RECORDS

PERIOD OF RECORD.--June 1952 to September 1971. Flood hydrograph, water years 1972-74. October 1974 to current year.

REVISED RECORDS.--WSP 1384: Drainage area.

GAGE.--Water-stage recorder. Datum of gage is 151.02 ft (46.031 m) National Geodetic Vertical Datum of 1929.

REMARKS.--Water-discharge record fair.

AVERAGE DISCHARGE.--23 years (water years 1953-71, 1975-78), 374 ft^3/s (10.59 m^3/s), 17.04 in/yr (433 mm/yr).

EXTREMES FOR PERIOD OF RECORD.--Maximum discharge, 48,600 ft^3/s (1,380 m^3/s) Feb. 25, 1961, gage height, 28.65 ft (8.733 m); no flow for many days in some years.

EXTREMES OUTSIDE PERIOD OF RECORD.--Flood of Nov. 28, 1948, reached a stage of 27.5 ft (8.38 m) at present site and datum; discharge, 38,300 ft^3/s (1,080 m^3/s).

EXTREMES FOR CURRENT YEAR.--Peak discharges above base of 5,000 ft^3/s (142 m^3/s) and maximum (*):

Date	Time	Discharge (ft^3/s)	Discharge (m^3/s)	Gage height (ft)	Gage height (m)
Jan. 26	1500	*17000	481	*24.76	7.547
May 5	0400	5570	158	19.10	5.822
May 10	0400	13900	394	24.05	7.330

Minimum discharge, 0.18 ft^3/s (0.005 m^3/s) Sept. 23, 24.

DISCHARGE, IN CUBIC FEET PER SECOND, WATER YEAR OCTOBER 1977 TO SEPTEMBER 1978
MEAN VALUES

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	20	9.7	17	290	394	917	45	45	18	6.0	7.8	1.0
2	20	10	16	130	1690	580	39	36	26	5.8	5.5	1.3
3	90	42	15	70	1660	590	39	731	159	6.0	7.8	1.0
4	60	522	16	56	1180	852	35	3390	236	7.2	75	.88
5	41	1550	62	50	549	556	32	5300	125	4.8	447	.77
6	24	1900	77	54	278	252	31	3690	47	4.0	31	.73
7	17	1420	41	70	204	295	30	1340	29	4.0	49	.69
8	13	468	39	150	162	1450	28	1610	280	4.0	11	.69
9	87	155	33	250	136	1900	27	7470	907	4.2	57	.69
10	42	92	25	130	120	2100	26	12000	906	9.4	190	.78
11	23	62	21	80	107	1210	44	6060	722	4.2	48	1.3
12	16	45	19	60	95	554	57	2130	158	3.0	18	.77
13	13	37	18	90	104	294	587	390	74	2.9	9.1	16
14	10	32	44	80	140	918	986	406	48	2.6	5.7	6.0
15	8.6	30	28	64	124	1620	738	319	33	5.0	3.9	1.3
16	8.1	28	24	56	98	1170	269	156	26	6.9	2.8	.76
17	7.8	26	87	110	83	744	114	175	22	3.8	2.8	.57
18	7.3	25	163	200	79	240	614	226	19	4.5	2.3	.41
19	7.7	23	180	350	82	164	2350	228	17	3.5	1.9	.34
20	6.8	22	106	640	88	126	2080	288	14	2.6	6.4	.45
21	6.8	21	55	720	76	119	1510	216	13	3.0	2.2	.30
22	6.2	23	35	350	65	114	607	125	12	5.0	2.1	.20
23	5.7	26	28	110	56	103	168	88	11	4.0	1.5	.18
24	6.2	23	24	140	50	92	106	50	9.0	2.8	1.2	1.5
25	81	24	83	2000	47	78	79	40	8.4	9.8	1.2	19
26	24	22	250	15000	58	72	65	34	18	12	1.1	27
27	23	20	110	9580	51	62	56	30	12	27	1.0	3.3
28	17	20	54	3920	398	55	51	27	7.2	61	1.0	2.0
29	14	18	46	862	---	50	43	25	6.0	41	1.0	1.1
30	12	17	150	315	---	46	38	22	5.8	7.5	1.5	16
31	11	---	340	230	---	42	---	20	---	6.9	1.6	---
TOTAL	729.2	6712.7	2206	36207	8174	17365	10894	46667	3968.4	274.4	997.4	107.01
MEAN	23.5	224	71.2	1168	292	560	363	1505	132	8.85	32.2	3.57
MAX	90	1900	340	15000	1690	2100	2350	12000	907	61	447	27
MIN	5.7	9.7	15	50	47	42	26	20	5.8	2.6	1.0	.18
CFSM	.08	.75	.24	3.92	.98	1.88	1.22	5.05	.44	.03	.11	.01
IN.	.09	.84	.28	4.52	1.02	2.17	1.36	5.83	.50	.03	.12	.01

CAL YR 1977 TOTAL 110848.90 MEAN 304 MAX 7900 MIN 1.8 CFSM 1.02 IN 13.84
WTR YR 1978 TOTAL 134302.11 MEAN 368 MAX 15000 MIN .18 CFSM 1.24 IN 16.77

TABLE 4-8. Example stream flow discharge records for water year October 1977 to September 1978 on Catoma Creek near Montgomery, Alabama.

MG14186.AO

DISCHARGE AT PARTIAL-RECORD STATIONS AND MISCELLANEOUS SITES--Continued

Flood hydrograph partial-record stations--Continued

MOBILE RIVER BASIN

02420987 HANNON SLOUGH (AT BEAR SCHOOL) AT MONTGOMERY, AL

LOCATION.--Lat 32°20'11", long 86°16'10", in SW¼ sec. 28, T. 16 N., R. 18 E., Montgomery County, Hydrologic Unit 03150201, at foot bridge for school crossing 800 ft (244 m) downstream from McGehee Road in Montgomery.

DRAINAGE AREA.--1.35 mi² (3.50 km²).

PERIOD OF RECORD.--October 1974 to September 1976, and October 1977 to current year (flood hydrograph only).

GAGE.--Water-stage recorder. Altitude of gage is 212 ft (65 m) from topographic map.

REMARKS.--Records fair.

EXTREMES FOR PERIOD OF RECORD.--Maximum discharge, 1,660 ft³/s (47.0 m³/s) May 9, 1978, maximum gage height, 9.52 ft (2.902 m) June 4, 1976.

EXTREMES FOR CURRENT YEAR.--Peak discharges above base (not determined) and maximum (*):

Date	Time	Discharge (ft³/s)	Discharge (m³/s)	Gage height (ft)	Gage height (m)	Date	Time	Discharge (ft³/s)	Discharge (m³/s)	Gage height (ft)	Gage height (m)
Jan. 25	1430	580	16.4	6.70	2.042	May 9	0715	*1660	47.0	8.89	2.710
Apr. 18	0645	481	13.6	6.37	1.942	Aug. 4	2015	900	25.5	7.50	2.286
May 8	1600	674	19.1	6.96	2.121						

GAGE HEIGHT, IN FEET, AND DISCHARGE, IN CUBIC FEET PER SECOND, AT INDICATED TIME, 1978

Date	Time	Gage height	Discharge
1-24	0030	2.22	0
1-24	1315	2.33	0
1-24	1330	3.63	30
1-24	1400	3.59	28
1-24	1430	3.64	30
1-24	1445	3.72	35
1-24	1500	4.09	59
1-24	1515	4.51	99
1-24	1545	5.30	210
1-24	1615	5.41	232
1-24	1730	4.69	121
1-24	1800	4.59	109
1-24	1815	4.78	132
1-24	1830	5.20	194
1-24	1915	5.48	246
1-24	2015	4.93	152
1-24	2130	4.82	137
1-24	2200	4.84	140
1-24	2315	4.44	92
1-24	2400	4.46	94
1-25	0030	4.35	83
1-25	0130	3.97	50
1-25	0200	3.80	40
1-25	0230	3.59	28
1-25	0830	3.47	23
1-25	0915	3.46	22
1-25	0930	3.49	24
1-25	0945	3.73	36
1-25	1015	3.92	47
1-25	1030	4.18	66
1-25	1100	4.64	115
1-25	1145	5.04	168
1-25	1215	5.41	232
1-25	1230	5.89	343
1-25	1245	6.05	385
1-25	1300	5.84	330
1-25	1315	5.45	240
1-25	1330	5.09	176
1-25	1345	4.87	144
1-25	1400	5.26	204
1-25	1415	6.33	469
1-25	1430	6.70	580
1-25	1445	6.07	391
1-25	1500	5.51	252
1-25	1515	5.07	173
1-25	1530	4.73	126
1-25	1600	4.22	70
1-25	1630	3.90	46
1-25	1700	3.66	32
1-25	1745	3.59	28

Date	Time	Gage height	Discharge
1-25	1800	3.68	33
1-25	1830	3.59	28
1-25	1845	3.59	28
1-25	1900	3.83	42
1-25	1915	3.79	39
1-25	1930	3.65	31
1-25	1945	3.59	28
1-25	2400	3.45	22
1-26	0015	3.44	22
1-26	0330	3.17	11
4-18	0615	2.32	0
4-18	0630	6.34	472
4-18	0645	6.37	481
4-18	0700	5.71	298
4-18	0715	5.26	204
4-18	0730	4.96	156
4-18	0830	4.04	55
4-18	0900	3.63	30
4-18	0930	3.58	27
4-18	1215	3.42	21
4-18	1400	3.19	12
4-18	1530	3.06	9
4-18	1545	3.09	10
4-18	1600	3.93	48
4-18	1630	3.58	27
4-18	1930	3.41	20
4-18	2100	3.20	12
4-18	2230	3.06	9
4-18	2245	3.07	9
4-18	2315	5.04	168
4-18	2345	5.62	275
4-18	2400	5.61	273
4-19	0015	5.25	202
4-19	0030	4.92	151
4-19	0045	4.55	108
4-19	0115	4.07	58
4-19	0130	3.81	41
4-19	0145	3.60	28
4-19	0530	3.41	20
4-19	0730	3.18	12
5-08	1530	2.54	0
5-08	1545	5.60	320
5-08	1600	6.96	674
5-08	1630	5.78	315
5-08	1645	5.27	205
5-08	1700	4.91	149

Date	Time	Gage height	Discharge
5-08	1800	4.21	69
5-08	1830	3.81	41
5-08	1845	3.67	32
5-08	1900	3.60	28
5-08	2300	3.44	22
5-08	2400	3.34	18
5-09	0115	3.18	12
5-09	0245	3.06	9
5-09	0300	4.24	72
5-09	0315	5.31	212
5-09	0345	5.17	189
5-09	0400	7.51	905
5-09	0415	7.28	802
5-09	0430	6.42	496
5-09	0445	5.58	266
5-09	0500	5.24	200
5-09	0530	5.05	170
5-09	0545	5.55	260
5-09	0600	6.66	568
5-09	0615	6.59	547
5-09	0630	7.78	1040
5-09	0700	8.82	1620
5-09	0715	8.89	1660
5-09	0730	8.73	1600
5-09	0800	6.91	654
5-09	0815	5.98	365
5-09	0830	5.23	199
5-09	0845	4.85	141
5-09	0900	4.57	106
5-09	0930	4.17	66
5-09	1000	3.85	43
5-09	1015	3.68	33
5-09	1030	3.61	29
5-09	1600	3.47	23
5-09	1915	3.16	11
8-04	1945	2.44	0
8-04	2000	4.62	112
8-04	2015	7.50	900
8-04	2030	7.12	738
8-04	2045	6.19	427
8-04	2100	5.40	230
8-04	2115	4.79	133
8-04	2130	4.35	83
8-04	2145	3.89	45
8-04	2200	3.65	31
8-04	2400	3.56	26
8-05	0015	3.54	26
8-05	0315	3.18	12

TABLE 4-9. Example USGS flood hydrograph record for Hannon Slough at Bear School during 1978.

Table 4-10
SUMMARY SHEET FOR COMPUTING CURVE NUMBERS

Land Use	Hydrologic Soil Groups A %	A CN	A Product	B %	B CN	B Product	C %	C CN	C Product	D %	D CN	D Product	Remarks
Residential Low Density Medium Density High Density													
Business/Commercial													
General Industry Light Industry Industrial Parks													
Institutional													
Open Area													
Swamps, Ponds, Lakes													
TOTALS													

Note: Computations using equation 5-3:

Table 4-11
PEAK RATES OF DISCHARGE FROM SMALL WATERSHEDS
USING SCS TECHNICAL RELEASE 55

Landowner ______________________ Sheet No. __________ of __________
County ______________________ Computed by __________ Date __________
Drainage Area = __________ Acres Checked by __________ Date __________
Average Watershed Slope = __________ % which is a Flat __________, Mod. __________,
or Steep __________ Slope from Table E-1 SCS Technical Release No. 55

Hydrologic Soil Group 1	Land Use 2	Treatment or Practice 3	Hydrological Condition 4	Runoff Curve Number 5	Area (Ac.) 6	Col. 5 X Col. 6 7

TOTALS = __________

Weighted Runoff Curve No. = $\frac{\text{Total Col. 7}}{\text{Total Col. 6}}$ = __________ = __________; Use __________

Peak Adjustment Factors:
A. From Table E-1, SCS TR-55, Slope Adj. = __________
B. From Fig. 4-1, SCS TR-55 with ______ % Impervious Area, Peak Factor = ______
C. From Fig. 4-2, SCS TR-55 with ______ % H.L.M., Peak Factor = ______

From Fig. D-2, SCS TR-55 for ________ Slope, D.A. = ________ Ac. and CN = ________
q = __________ cfs/inch of runoff

Q (Adjusted Peak Discharge in cfs/in. R.O.) = q x (Adjustment Factors)
Q = __________ cfs/in. R.O. x (______) (______) (______) = ________ cfs/in. R.O.

Rainfall Freq. =	________ Yr.	________ Yr.	________ Yr.
Rainfall 1/ =	________ Inches	________ Inches	________ Inches
Runoff 2/ =	________ Inches	________ Inches	________ Inches
Peak Discharge = Q x Runoff =	________ cfs	________ cfs	________ cfs

1/ From Figure 4-2.
2/ From Figure 5-4 or 5-5.

Table 4-12
EXAMPLE COMPUTATIONS TO DETERMINE THE EXISTING COMPOSITE
FOR SUBBASIN 2 OF THE WEST END DITCH

	Hydrologic Soil Groups											
	A			B			C			D		
Existing Land Use	%	CN	Product	%	CN	Product	%	CN	Product	%	CN	Product
Residential							5	91	455			
Business/Commercial				18	86	1,548	11	91	1,001			
Institutional							10	80	800	4	85	340
Open Area				10	55	550	33	70	2,310	9	77	693
Totals				28		2,098	59		4,566	13		1,033

Note: Computations using equation 5-3:

$$\overline{CN} = \frac{2{,}098 + 4{,}566 + 1{,}033}{100} = \frac{7{,}697}{100} = 76.97 \cong \underline{77} \text{ Existing composite CN}$$

Table 4-13

EXAMPLE SUMMARY SHEET FOR TABULATING MAJOR HYDROLOGIC SUBBASIN AREA, LAND USE (existing and ultimate), HYDROLOGIC SOIL GROUPS AND AVERAGE BASIN SLOPES

		Land Use as Percent of Total									Hydrologic Soil Group Percent of Total				Average Basin Slope Percent of Total			
		Residential Density																
Subbasin	Area (sq mi)	Low	Moderate	High	Commercial	Industrial	Agricultural	Institutional	Forest	Open	A	B	C	D	0-5	5-10	10-15	>15

Table 4-14
SUMMARY SHEET FOR TABULATING CROSS SECTION DATA,
TIME OF CONCENTRATION, EXISTING AND ULTIMATE CURVE NUMBER
WITH CORRESPONDING PERCENT CHANGE

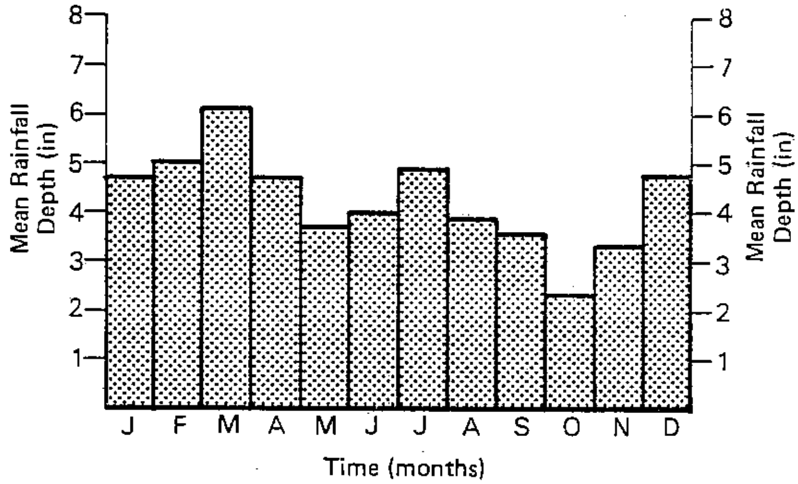
Subbasin Identification Number	Uniform Channel Segments				Time of Concentration (hrs)	Curve Numbers			Remarks
	Length (feet)	Left n Value	Channel n Value	Right n Value		Existing	Ultimate	% Change	

MG14186.AO

Station: Dannelly Field

Period of Record: 1873-1979

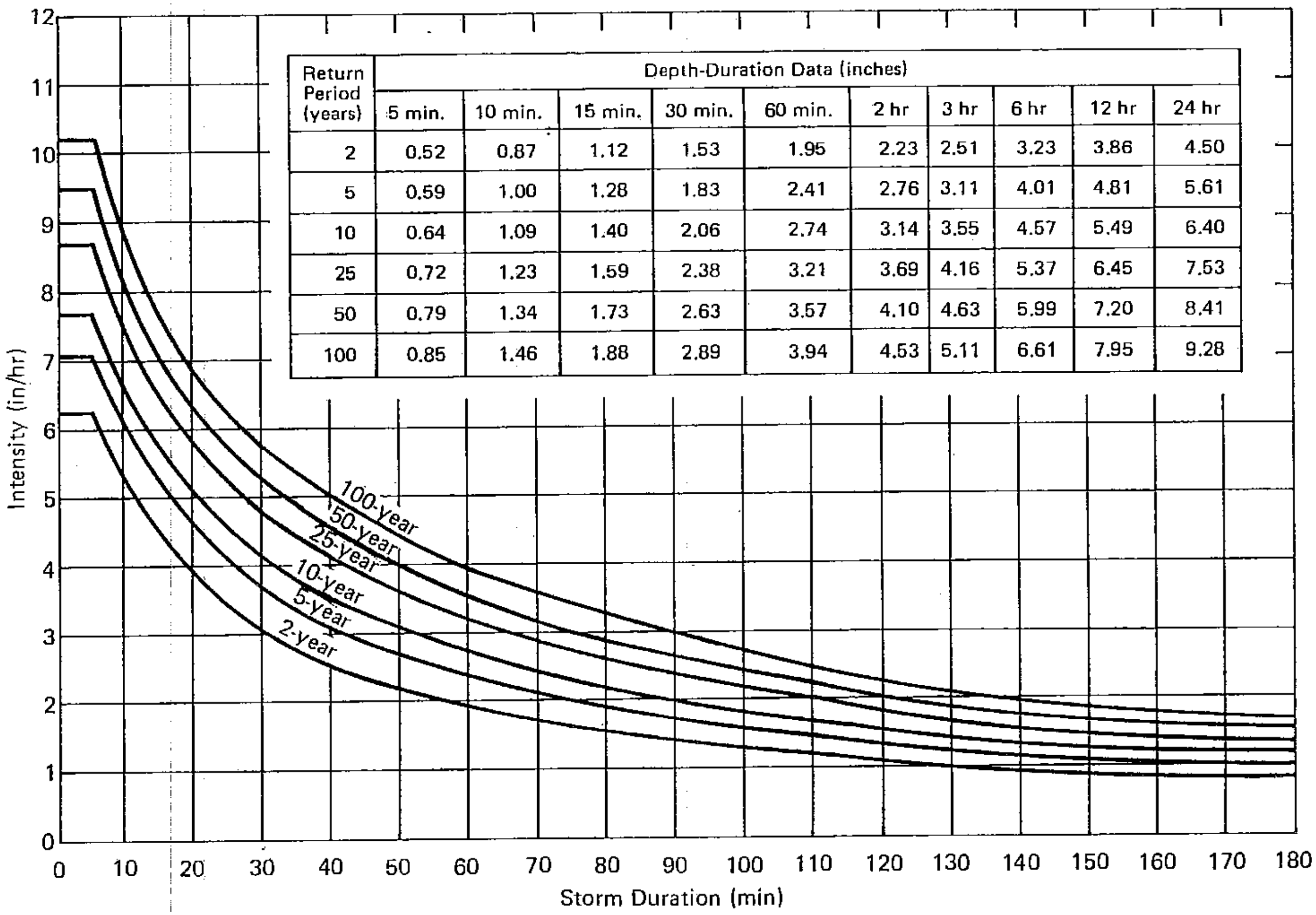
Recorded Mean Annual Depth: 50.95 inches



Source: U.S. Dept. of Commerce (1979).

FIGURE 4-1. Average monthly rainfall distribution for Montgomery, Alabama.

Sources: NWS Hydro-35, and USWB TP-40.



Return Period (years)	Depth-Duration Data (inches)									
	5 min.	10 min.	15 min.	30 min.	60 min.	2 hr	3 hr	6 hr	12 hr	24 hr
2	0.52	0.87	1.12	1.53	1.95	2.23	2.51	3.23	3.86	4.50
5	0.59	1.00	1.28	1.83	2.41	2.76	3.11	4.01	4.81	5.61
10	0.64	1.09	1.40	2.06	2.74	3.14	3.55	4.57	5.49	6.40
25	0.72	1.23	1.59	2.38	3.21	3.69	4.16	5.37	6.45	7.53
50	0.79	1.34	1.73	2.63	3.57	4.10	4.63	5.99	7.20	8.41
100	0.85	1.46	1.88	2.89	3.94	4.53	5.11	6.61	7.95	9.28

FIGURE 4-2. Rainfall intensity-duration-frequency curves and depth-duration-frequency data for Montgomery, Alabama.

CH2M HILL

MG14186.AO

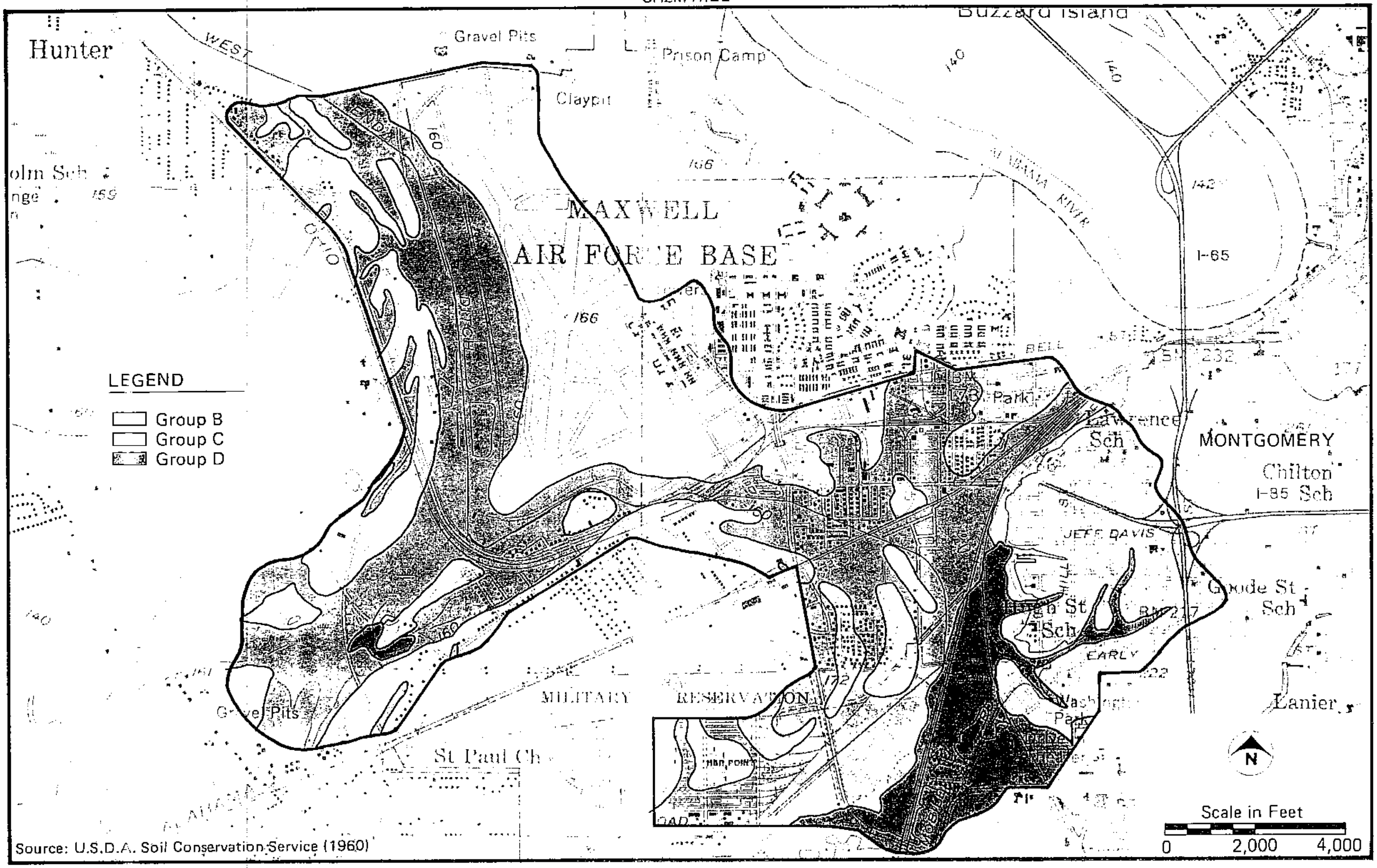


FIGURE 4-3. Example hydrologic soil map for West End Ditch.

Figure 4-4.
General soil map for the City of Montgomery, Alabama.

(Map inserted in Appendix D.)

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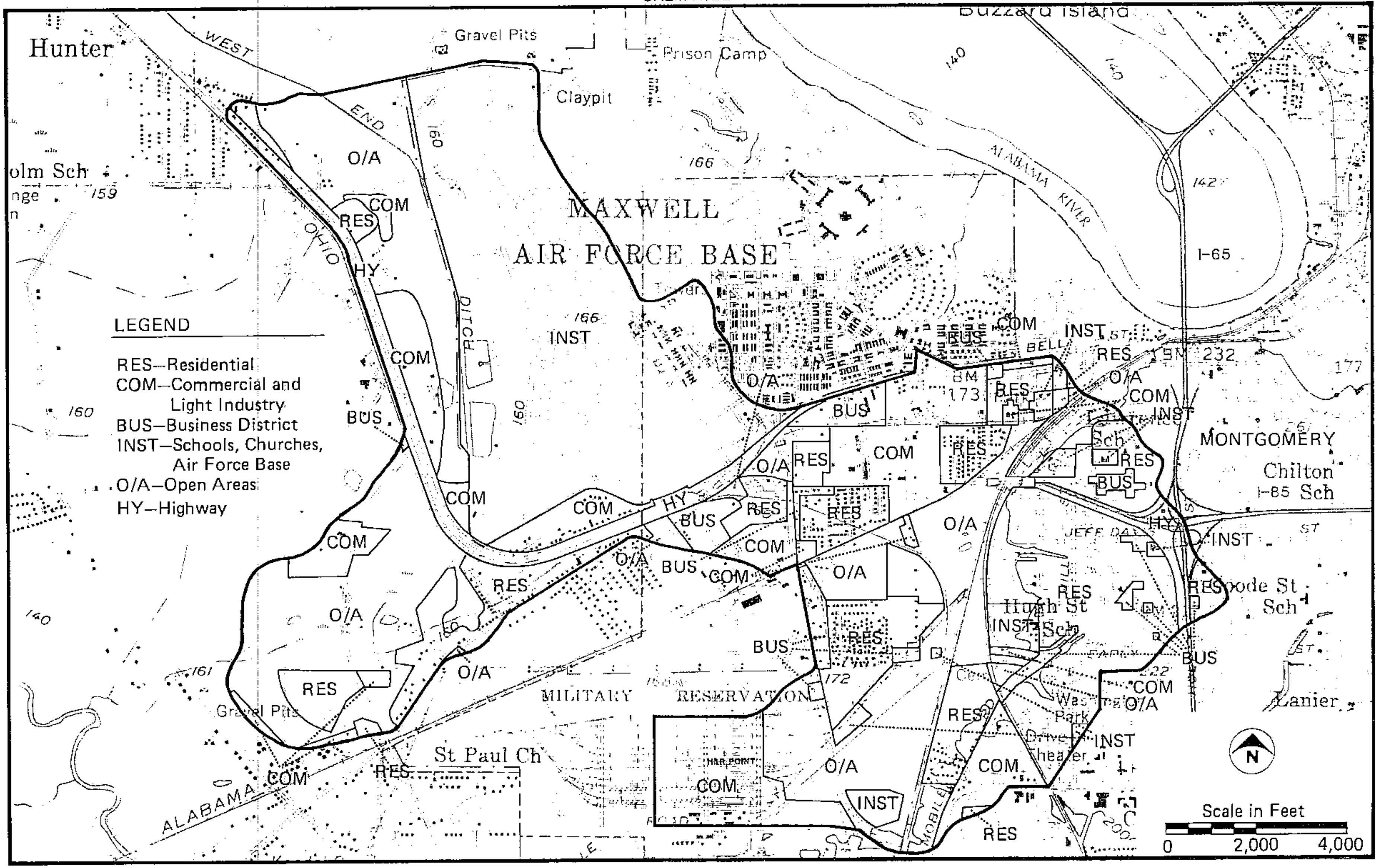

FIGURE 4-5. Example existing land use map for West End Ditch.

Figure 4-6.
Approximate delineations of major urban watersheds in Montgomery, Alabama.

(Map inserted in Appendix D.)

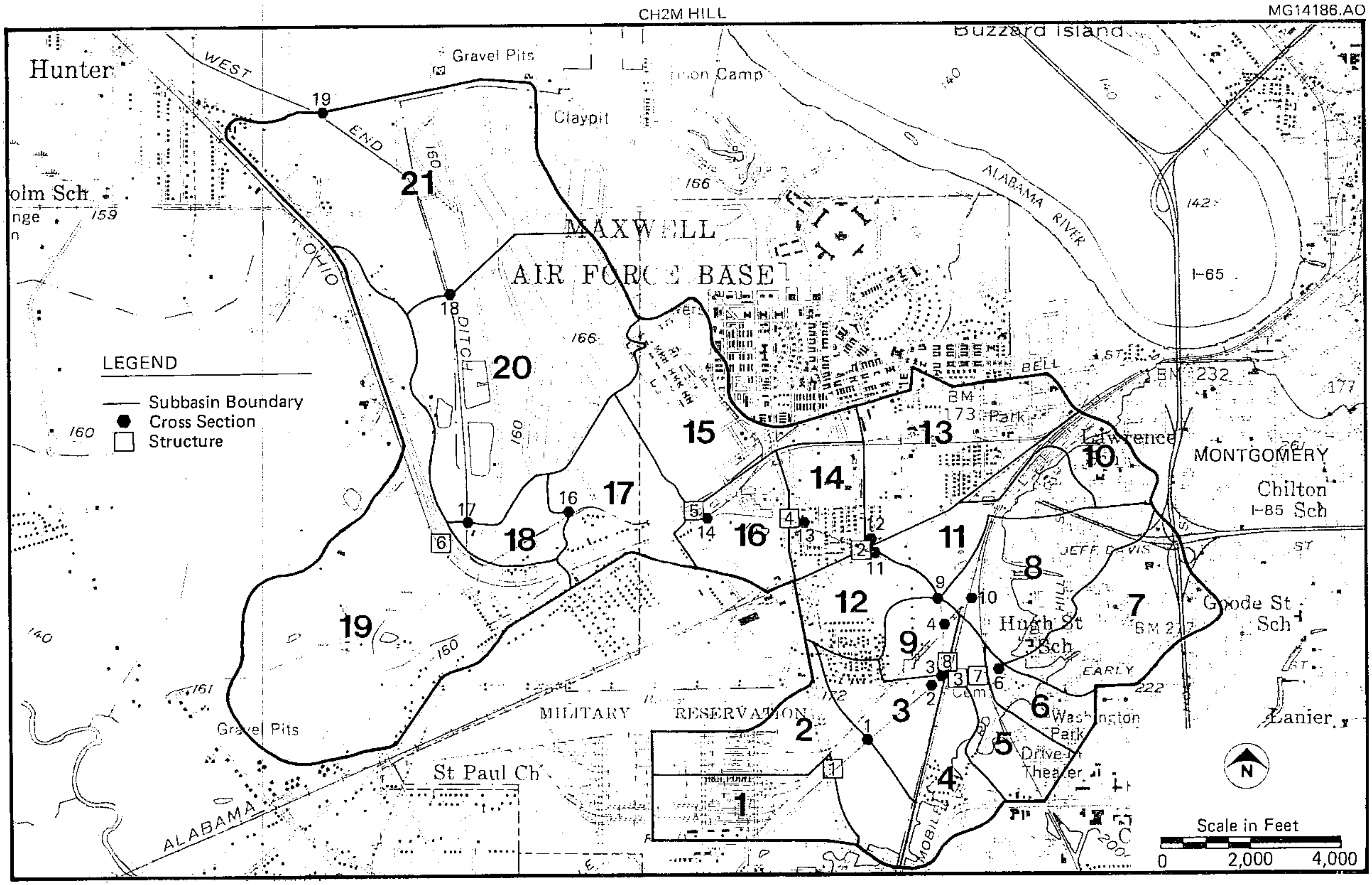

FIGURE 4-7. Example subbasin delineations with channel cross sections and structures for West End Ditch.

SECTION 1.0 INTRODUCTION

The estimation of stormwater volumes and flow rates is a prerequisite to the hydraulic design of any stormwater management system. Since the governing relationships of urban hydrology are quite complex and, unlike problems in engineering mechanics, cannot be solved directly using fundamental laws of physics, a wide variety of conceptual models and empirical approaches have been developed. It is the purpose of Chapter 5 to document those conceptual hydrologic models and empirical approaches which are recommended as hydrologic design tools in the City of Montgomery.

The hydraulic design of a stormwater conveyance system generally requires an estimate of the peak rate of runoff generated by the design event. This is one of the oldest problems in applied hydrology and is still one of the most difficult to solve accurately. When streamflow measurements are not available, it is probably wise to compare several methods for estimating peak flows. Hydrologic methods for estimating peak flows presented in this chapter include the rational method, multiple regression equations, and the SCS unit hydrograph method. While it is good engineering practice to consider more than one method when estimating stormwater runoff, it is not recommended that values from several methods be averaged. Instead, a single method should be selected which best fits the characteristics of the subject watershed.

Stormwater detention/retention system design requires that the complete time distribution of runoff for the design event (i.e., runoff hydrograph) be known. In general, a runoff hydrograph is determined by application of synthetic unit hydrographs. Several synthetic unit hydrograph techniques are presented in this chapter.

Sequences of calculations in urban hydrologic studies are similar regardless of the tools chosen, because a precipitation input must be routed through watersheds, channels, and reservoirs. Stormwater runoff is likely to be estimated using the following sequence of calculations:

1. Collect and analyze watershed data.
2. Develop a design storm.
3. Calculate the peak runoff rate or determine excess precipitation.
4. Develop a unit hydrograph for the watershed.
5. Develop the direct runoff hydrograph.
6. Perform channel and reservoir routings as needed.

Chapter 4 should be referenced as a guide for collecting the watershed data required in Chapter 5. The hydrologic analysis of a specific watershed requires that the stormwater problem be defined and that a performance standard for the system be established.

SECTION 2.0 DESIGN STORM CRITERIA

Experience has indicated that the overall drainage network can be divided into a major and a minor component. The minor system generally carries onsite stormwater to a through-site major conveyance system. A minor system generally includes stormwater inlets, street gutters, roadside ditches, and onsite collector pipes which discharge to a through-site trunk storm sewer, open channel, or detention system. The major system includes not only the trunk line drains, which receive stormwater from the minor system, but also the area affected by backwater flooding when the capacity of the minor system is exceeded. The proper identification of a major overland relief system for onsite drainage can significantly reduce flood damage to homes, businesses, and other property. As a general rule for use in Montgomery, a major through-site drainage system begins when the tributary drainage area is equal to or greater than 1 square mile.

2.1 Design Storm Return Periods

Criteria for selecting a design storm return period in Montgomery are presented in Table 5-1. As identified in Table 5-1 a major through-site drainage system (tributary drainage area is equal to or greater than 1 square mile) shall be designed to confine the 100-year storm within the conveyance system. Drainage for areas less than 1 square mile but greater than or equal to 15,000 square feet shall be designed to confine the 25-year storm within the conveyance system. In addition, the path of flow when the confined conveyance capacity is exceeded must be designated for the 100-year storm. Drainage for private property areas which drain less than 15,000 square feet shall be designed to confine the 10-year storm within the conveyance system, except when sump conditions exist, where a 25-year storm is required.

Other factors which may affect choice of the design storm return period include:

1. Use of less frequent, more intense rainfall for design of those parts of the system for which future relief would not be economically feasible.

2. Use of less frequent, more intense rainfall for design of special structures such as expressway drainage pumping systems where runoff exceeding capacity would seriously disrupt an important facility. Design frequencies of 50 years or more may be justified in such cases, particularly in

small drainage areas, even though the project may be located in a district justifying only 10-year return period for normal drainage.

2.2 Design Storm Rainfall Data

Once the design storm return period has been selected, the type of rainfall data required depends on the type of drainage structure being considered. For example, stormwater conveyance systems may require only an estimate of the rainfall intensity as an input to the rational method. On the other hand, if a detention/retention system is required, a runoff hydrograph must be estimated for the given design storm. When only the rainfall intensity or depth is required, it can be obtained directly from Figure 4-2, given the watershed time of concentration. The watershed time of concentration is determined using information presented in Subsection 4.2.2 of this chapter. To estimate the runoff hydrograph a rainfall hyetograph may be required. The development of a rainfall hyetograph is discussed below.

The rainfall hyetograph for a particular design storm is developed from the depth-duration curve for a particular storm return period. An example depth-duration curve is plotted on Figure 5-1 for a 25-year return period. This depth-duration curve does not represent a logical, chronological sequence of rainfall during a storm; therefore, the rainfall depth increments must be rearranged into a sequence which might actually occur. A balanced storm procedure can be used to accomplish this rearrangement or, if a 24-hour duration storm is desired, an SCS type II rainfall distribution can be used. Both procedures are illustrated below. In either case this synthetic design storm will be more radical than most actual storms of the same total depth and duration because the short-duration events are assumed to be nested within the long-duration storm. This is accepted practice because nesting intense, short-duration events within the longer-duration events, ensures that critical events for the small headwater areas of a watershed will not be overlooked. In addition, if the design storm duration is long enough, the critical duration for sizing a storage basin is ensured.

The calculations for developing a 25-year, 24-hour design storm hyetograph are presented in Table 5-2. The first step is to tabulate rainfall depth versus duration data using the depth-duration curve for Montgomery, which is shown on Figure 5-1. These numbers are listed in columns 1 and 2 of Table 5-2 and have been slightly adjusted to produce a continuously decreasing incremental depth curve, as listed in column 3 of Table 5-2. Note that the incremental depths have been placed in between time intervals to indicate that this rainfall depth accumulated during a particular time interval. The balanced storm method for developing a synthetic hyetograph requires that incremental depths be rearranged to approximate a logical, chronological sequence of those depths. This rearrangement is listed in column 4 of Table 5-2. The most intense half-hour depth of rainfall (2.38 inches) was arbitrarily placed between the 11.5- and 12.0-hour time interval. The second most intense half-hour depth of rainfall (0.62 inches) was placed one-half-hour behind the most intense increment (between the 12.0- and 12.5-hour time interval). The third most intense half-hour depth of rainfall (0.52 inches) was placed one-half-hour before the most intense increment (between the 11.0- and 11.5-hour time interval). This process of placing half-hour increments before and after the most intense increment is termed the "balanced storm method" and continues until each half-hour increment of rainfall has been rearranged in column 4 of Table 5-2. The cumulative depth of this balanced synthetic hyetograph is listed in column 5 of Table 5.2.

An alternative method for developing a synthetic design storm hyetograph was developed by the SCS. Based on experience, the SCS found that hyetographs developed using published data were quite similar. For areas subject to both short-duration summer thunderstorms and long-duration frontal storms, as in Montgomery, the rainfall distribution versus time is termed a "type II" distribution by the SCS. For areas where intense short-duration storms are not prevalent the rainfall distribution versus time is termed a "type I" distribution by the SCS. The SCS has published these type I and II rainfall distributions as dimensionless cumulative depth curves. The SCS 24-hour, type II dimensionless cumulative depth curve is presented on Figure 5-2. Given the ratio of accumulated rainfall at time x to total rainfall (i.e., P_x/P_{24} shown on the ordinate of Figure 5-2 where P_x = cumulative rainfall depth at time period x and P_{24} = 24-hour rainfall depth for the selected design storm return period) and 24-hour rainfall depth for the design storm, a complete synthetic rainfall hyetograph can be calculated. This procedure is illustrated in columns 6 and 7 of Table 5-2.

A comparison of the synthetic cumulative rainfall distributions calculated using the balanced storm method and the SCS type II dimensionless cumulative depth curve is shown on Figure 5-3. This figure indicates that there is very little difference between the cumulative rainfall estimated using either method for a 25-year, 24-hour storm in Montgomery. Therefore, if a 24-hour synthetic rainfall hyetograph is required, the SCS procedure is the preferred method since it is much easier.

SECTION 3.0 RAINFALL EXCESS

Rainfall excess is that portion of total precipitation which becomes stormwater runoff after a storm event. Those portions of a storm's total precipitation volume which do not become stormwater runoff are called abstractions. Abstractions include interception, evaporation, infiltration, surface storage, surface detention, and bank storage. When a stormwater system is designed using a single design storm event, time scales are short enough that only infiltration and initial abstractions need to be considered. Initial abstractions include interception and surface storage before runoff can begin. For practical purposes all other abstractions can be considered negligible when a single design storm event is considered.

The infiltration rate is, in general, dependent on physical properties of the soil, vegetative cover, antecedent soil moisture conditions, rainfall intensity, and the slope of the infiltrating surface. This combination of factors which affects the infiltration rate for a particular watershed interacts to cause a generally complex spatial distribution of infiltration capacity.

3.1 SCS Curve Number Excess Rainfall Model

The Soil Conservation Service (SCS) of the United States Department of Agriculture combines infiltration losses with initial abstractions and estimates onsite rainfall excess using a curve number excess rainfall model. Conceptually, this model provides a high rate of rainfall abstraction at the beginning of a storm. This rate of rainfall abstraction decreases in a convex manner while the rate of precipitation increases. When the precipitation rate becomes greater than the watershed's capacity for further abstractions, the rainfall becomes effective and stormwater runoff begins. The SCS curve number excess rainfall model is expressed in the following equation:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad (5\text{-}1)$$

where

Q = accumulated rainfall excess or runoff, in inches

P = accumulated rainfall, in inches

S = maximum watershed rainfall retention factor

The rainfall retention factor, S, is related to the watershed curve number (CN) by the following relationship:

$$S = \frac{1{,}000}{CN} - 10 \qquad (5\text{-}2)$$

The watershed CN is a dimensionless coefficent that reflects watershed cover conditions, hydrologic soil group, land use, and antecedent moisture conditions.

Three levels of antecedent moisture conditions are considered by the SCS model. Antecedent moisture condition I (AMC-I) is the lower limit of antecedent rainfall or the upper limit of S. Antecedent moisture condition II (AMC-II) represents average antecedent rainfall conditions, and antecedent moisture condition III (AMC-III) is the upper limit of antecedent rainfall or the lower limit of S. For design purposes, only antecedent moisture condition II is presented in this manual. If additional information related to curve numbers for other antecedent mositure conditions is required, the SCS National Engineering Handbook, Hydrology, Section 4, (NEH)-4 (1972) should be consulted.

3.2 SCS Curve Numbers

The SCS has classified more than 4,000 soil series into four hydrologic soil groups which are denoted by the letters A, B, C, and D, as defined in Table 5-3. It is important to note that soils in the "A" hydrologic soil group have a low runoff potential, whereas soils in the D group have a high runoff potential. Dual hydrologic groups can be assigned to certain wet soils that may be drained or undrained. In such cases the first letter applies to the drained condition, and the second to the undrained condition. Criteria which are used when assigning dual groups are as follows:

1. Soils are rated D in their natural condition.

2. Drainage is feasible and practical.

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

The hydrologic soil grouping considers only soil properties which influence the minimum rate of infiltration obtained for a bare soil after prolonged wetting. The influence of ground cover on stormwater runoff is treated independently when a watershed CN is selected (as discussed below). A tabular presentation of hydrologic soil groups for soil series which have been identified in Montgomery County are presented in Table 4-3.

Given the hydrologic soil group of an area, the land use, and vegetative cover, a curve number for antecedent moisture condition II can be selected from Tables 5-4 or 5-5. The classification of vegetative covers by their hydrologic properties is defined in Table 5-6. When a particular drainage area has a wide range of hydrologic soil groups, land uses, and vegetative covers, a composite curve number for the watershed can be calculated using an areal weighting procedure. Equation 5-3 can be utilized for this purpose:

$$\overline{CN} = \frac{\sum_{i=1}^{n} CN_i\ A_i}{\sum_{i=1}^{n} A_i} \qquad (5\text{-}3)$$

where

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

CN_i = curve number for an area i with uniform characteristics

A_i = drainage area of an area i with uniform characteristics, expressed as a percentage of the total drainage area

n = total number of areas with uniform characteristics

Given a composite curve number for a watershed, the accumulated excess rainfall can be estimated as a function of accumulated rainfall using equations 5-1 and 5-2. A graphical solution for this excess rainfall model is presented on Figures 5-4 and 5-5.

When curve numbers are determined for an urban area, consideration should be given to the degree to which heavy equipment is likely to compact the soil, and the degree to which grading has mixed the surface and subsurface soils. In addition, the amount of pervious area which is barren (with little sod established) should be considered. Any one of the above factors could cause a soil normally in hydrologic group A or B to be classified in group B or C, respectively.

The percentage of impervious area for the various types of residential areas or the land use condition for the pervious area may vary from the conditions presented in Table 5-4. For such cases, the SCS has developed a figure for estimating the composite curve number of a watershed, given the predevelopment pervious area curve number and the percentage of impervious area after development. This information is presented on Figure 5-6. Figure 5-6 was developed assuming that the impervious area has a CN value of 98.

3.3 Example Problems

Example 5-1. SCS Excess Rainfall Computation

The existing hydrologic soil group and land use data for subbasin 2 of the West End Ditch are given in Table 4-11. Using equation 5-3, a composite CN for existing conditions was found to be 77. Calculate the excess rainfall for existing conditions from subbasin 2 for a 25-year, 30-minute storm using the SCS excess rainfall model.

1. From Figure 4-2, the depth of a 25-year, 30-minute storm in Montgomery is 2.38 inches.

2. The rainfall retention factor is determined using equation 5-2:

$$S = \frac{1,000}{77} - 10 = 2.99 \text{ inches}$$

3. The excess rainfall is determined using equation 5-1:

$$Q = \frac{[2.38 - (0.2)(2.99)]^2}{2.38 + (0.8)(2.99)}$$

$$Q = \frac{3.18}{4.77} = 0.67 \text{ inches}$$

As an alternative to this calculation, Figure 5-4 can be used as a quick method for determining the excess rainfall by the SCS method.

Example 5-2. SCS Curve Number Computation for Land Development

An undeveloped land area is comprised entirely of soils which are classified in hydrologic soil group C. The land is presently tree-covered with good ground cover. It is estimated that 50 percent of the area will be impervious and connected hydraulically after development. Determine the composite CN after development and calculate the excess rainfall for a 25-year, 24-hour design storm using the SCS excess rainfall model.

1. From Figure 4-2 the depth of a 25-year, 24-hour storm in Montgomery is 7.53 inches.

2. From Table 5-4 the undeveloped or pervious area CN = 70.

3. From Figure 5-6 the composite CN = 84 for developed conditions.

4. From Figure 5-4 using CN = 84 and P = 7.53 inches read Q = 5.7 inches or compute Q as follows:

$$S = \frac{1,000}{84} - 10 = 1.90 \text{ inches}$$

$$Q = \frac{[7.53 - (0.2)(1.90)]^2}{7.53 + (0.8)(1.90)} = \frac{51.11}{9.05} = 5.65 \text{ inches}$$

Example 5-3. Rainfall Excess Hyetograph Computations

The ultimate hydrologic soil group and land use data for subbasin 2 of the West End Ditch are listed below. Using equation 5-3, a composite CN for ultimate land use conditions is found to be 84. Develop a rainfall excess hyetograph for five 30-minute time increments near the peak of the 25-year, 24-hour design storm hyetograph presented in Table 5-2.

	A			B			C		
Land Use	%	CN	Product	%	CN	Product	%	CN	Product
Residential									
Medium Density				5	91	455			
Business/Commercial	28	86	2,408	11	91	1,001			
General Industry				23	86	1,978			
Institutional				10	80	800	11	85	935
Open Area				10	70	700	2	77	154
TOTALS	28		2,408	59		4,934	13		1,089

Computations: $\overline{CN} = \frac{2,408 + 4,934 + 1,089}{100} = 84.3 \sim 84$ ultimate composite CN

Time (hours)	Balanced Storm Cumulative Depth (inches)	Cumulative Rainfall Excess (inches)	Increment of Excess Rainfall (inches)
10.5	1.60	0.48	
11.0	1.91	0.68	0.20
11.5	2.43	1.06	0.38
12.0	4.81	3.10	2.04
12.5	5.43	3.67	0.57
13.0	5.75	3.97	0.30

Note: The above data were taken from Table 5-2.

$$S = \frac{1,000}{84} - 10 = 1.90 \text{ inches}$$

$$Q = \frac{(P - 0.38)^2}{P + 1.52} = \text{excess rainfall, in inches}$$

SECTION 4.0 PEAK RUNOFF RATES

4.1 Recommended Procedures

Four procedures should be considered in the Montgomery urban area for estimating stormwater peak runoff rates: the rational method, unit hydrograph theory, flood frequency regression equations, and historical high water marks. The rational method is by far the most common method for estimating peak flows in urban hydrology. Although it will continue to be a valuable hydrologic tool in the Montgomery urban area, the rational method is not always the best approach for every problem and it is good engineering practice to compare several approaches when they are applicable.

Experience has shown that for tributary areas greater than 50 acres, the cost of a stormwater system justifies significantly more study, thought, and judgment by the engineer than is permitted with the rational method. When the subject watershed area exceeds 50 acres, unit hydrograph theory or the SCS graphical procedures (see Table 5-7) represent better practice and must be used in Montgomery. Theoretically, it provides the engineer with a higher degree of dependability, and it can be used directly to estimate the entire runoff hydrograph in addition to peak flows. Flood frequency regression equations can be utilized as an independent check on estimated peak flows; however, they are generally developed for medium- or large-size watersheds (greater than 300 acres) and often represent undeveloped watershed conditions. When available, historical high water marks can provide an order-of-magnitude check on pre-development peak flows determined by other methods. The recommended applications for these four peak flow estimating procedures are summarized in Table 5-7.

An empirical approach for estimating peak stormwater discharge rates which has been widely used in the Montgomery area is the Burkli-Zeigler formula. The Burkli-Zeigler formula was published in the U.S. prior to the rational method. It was based on the Hawksley formula for determining the waterway area of a sewer given the drainage area and length of sewer in feet per foot of fall. In deriving the forumla, observations were limited to drainage areas less than 50 acres. The Burkli-Zeigler formula is similar to the rational method; however, a watershed time of concentration is not required and rainfall intensity is arbitrarily assigned a maximum storm of record. Since the Burkli-Zeigler formula does not account for the highly variable conditions in urban hydrology, it is not recommended for use in the Montgomery area.

The Talbot formula is another empirical approach published prior to the rational method. The Talbot formula utilizes the Burkli-Zeigler formula to determine an approximate relationship between waterway area and drainage area. The Talbot formula does not provide an estimate of peak stormwater flows. Since it is only an approximation of the waterway area based on the Burkli-Zeigler formula, the Talbot formula is also not recommended for use in the Montgomery area.

4.2 Rational Equation

According to the rational equation, the peak rate of stormwater runoff can be estimated as the product of a runoff coefficient, a rainfall intensity, and the drainage area. The rational equation is frequently expressed mathematically as follows:

$$Q = CIA \qquad (5\text{-}4)$$

where

Q = peak runoff rate, in cfs

C = rational method runoff coefficient, dimensionless

I = average rainfall intensity for the design storm, in inches/hour

A = watershed drainage area, in acres

To aid in the analysis of parameters utilized in the rational equation, the following form of the rational equation is utilized in this manual:

$$Q_T = C_T \, I_T(t_c) \, A \tag{5-5}$$

where

Q_T = peak runoff rate, in cfs, for the design storm return period, T

C_T = rational method runoff coefficient, expressed as a dimensionless ratio (i.e., the ratio of rainfall excess to total rainfall) for the design storm return period, T

$I_T(t_c)$ = average rainfall intensity, in inches/hour, during a period of time equal to t_c for the design storm return period, T (i.e., I_T is a function of t_c)

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

t_c = watershed time of concentration, in minutes, defined as the time required for runoff to travel from the hydraulically most distant point of a watershed to the design point

To be dimensionally correct, a conversion factor of 1.008 should be used to convert acre-inches per hour to cfs; however, this factor is usually neglected. The basic assumptions behind the rational method include the following:

1. Runoff is linearly related to rainfall.

2. The rainfall occurs uniformly over a given watershed.

3. The peak runoff rate occurs when the entire area is contributing flow.

4. The excess rainfall hyetograph is one of constant intensity for a duration equal to t_c.

5. The frequency of the peak runoff rate is the same as the frequency of the average rainfall intensity.

4.2.1 Runoff Coefficients. In general, the runoff coefficient is the variable in the rational method which is least subject to a precise determination. The runoff coefficient must account for rainfall interception, surface storage, and infiltration to accurately estimate excess rainfall during a single design storm event. Variables which should be considered when estimating a runoff coefficient include soil type, land use, antecedent moisture condition, duration of rainfall, and the intensity of rainfall as reflected by the design storm return period. The first three of these variables can be accounted for specifically by estimating an SCS curve number for a watershed as described previously in Section 3.2 of this chapter. The rainfall duration and intensity for a specific design storm are determined by calculating the watershed time of concentration, which provides an estimate of the total rainfall depth, for that design storm return period. Since by definition the runoff coefficient is the dimensionless ratio of rainfall excess to total rainfall, the runoff coefficient can be expressed by the following equation:

$$C_T = \frac{Q_T}{P_T} \qquad (5\text{-}6)$$

where

C_T = runoff coefficient for the design storm return period, T

Q_T = rainfall excess for the design storm return period, T

P_T = total rainfall depth for the design storm return period, T

4.2.1.1 SCS Curve Numbers--If the design storm excess rainfall is estimated using the SCS curve number excess rainfall model as expressed by equation 5-1, equation 5-6 can be simplified algebraically to yield the following equation for estimating the runoff coefficient:

$$C_T = 1 - \frac{S}{P_T}\left[1.2 - \frac{S}{P_T + 0.8S}\right] \qquad (5\text{-}7)$$

where

$S = \frac{1{,}000}{CN} - 10$ (equation 5-2)

CN = SCS curve number (see Tables 5-4 and 5-5 and Figure 5-6)

A recent paper by Johnson and Meadows (1980) reported the above technique for estimating the rational method runoff coefficient. The great advantage in this relationship is the fact that variations in C_T due to rainfall duration and design storm return period are considered in a precise manner. It also greatly reduces the judgment commonly required to estimate runoff coefficients. Once the design event return period, T, time of concentration, t_c, average rainfall intensity, $I_T(t_c)$, and the SCS runoff curve number, CN, are established, a unique value for C_T can be calculated.

4.2.1.2 Tables--When insufficient data are available to determine a runoff curve number using equation 5-6, typical values of runoff coefficients for various land uses, soil types, and watershed slopes, as presented in Table 5-8, should be used. These runoff coefficients apply when a design storm return period of 10 years or less is considered. When higher return period runoff coefficients are desired, the coefficients presented in Table 5-8 should be multiplied by the frequency factors presented in Table 5-9. This accounts for the fact that during intense storms, the rate of rainfall loss will not increase as much as does the rainfall rate. Therefore, the runoff coefficient must increase for higher return periods. The following relationship is used to combine the data presented in Tables 5-8 and 5-9:

$$C_T = C_{10} X_T \qquad (5\text{-}8)$$

where

C_T = runoff coefficient for the design storm return period, T

C_{10} = runoff coefficient for a design storm return period of 10 years or less (see Table 5-8)

X_T = design storm frequency factor for the return period, T (see Table 5-9)

In no case should the value of C_T be increased above 1.0. For watersheds with multiple land uses, soil types, and watershed slopes, a composite runoff coefficient should be calculated using equation 5-3 and replacing the CN value with C_T.

4.2.2 Watershed Time of Concentration. A second variable in the rational equation which requires considerable engineering judgment is the watershed time of concentration, t_c. The time of concentration for a particular watershed cannot be determined precisely from field data or rainfall runoff records. Numerous empirical relationships are available for estimating the value of t_c from site-specific watershed data.

Before presenting several of these relationships, it is important to note that t_c in the rational equation is not necessarily the total duration of the design storm. It is merely a time period for defining the maximum average rainfall intensity for a particular design storm return period. This maximum average rainfall intensity bears no direct relationship to the sequence of rainfall depths during an actual storm (see discussion in Section 2). Rossmiller (1980) accounts for this fact by defining t_c as the rainfall intensity averaging time.

For the purposes of this manual, the rational method time of concentration will be defined as the time required for runoff to travel from the hydraulically most distant point on a watershed to the design point. The rational method time of concentration is typically comprised of two components: (1) a conveyance system inlet time and (2) a reach travel time. The conveyance system inlet time in an urban area consists of the time required for runoff to travel from the uppermost part of the watershed to an established surface conveyance system (i.e., overland flow) plus the travel time through minor ditches and street gutters to the inlet of a larger conveyance system. The reach travel time begins when runoff enters the larger conveyance system (e.g., at the inlet to a storm sewer or open channel system) and continues until runoff reaches the desired design point in the watershed. The following two equations summarize the relationships between these various components of the time of concentration.

$$t_c = t_I + t_3 \tag{5-9}$$

$$t_I = t_1 + t_2 \tag{5-10}$$

where

t_c = watershed time of concentration

t_I = conveyance system inlet time

t_1 = overland flow travel time

t_2 = ditch and street gutter travel time

t_3 = conveyance system travel time

Several methods are available for estimating each of the above parameters. While it is good engineering practice to consider more than one method for estimating these time parameters, it is not recommended that time values from several methods be averaged. Instead a single method should be selected which bests fits the characteristics of the subject watershed.

Approximate velocites of overland flow and channel flow should be considered as an order-of-magnitude check on the estimated watershed time of concentration. For paved areas, Jens and McPherson (1964) recommend velocities of 0.33 ft/sec to 0.82 ft/sec as the length of flow to the inlet varies from 100 to 500 feet, respectively. For turfed areas they recommend velocities of less than 0.2 ft/sec when the length of flow is less than 100 feet; and when the length of flow is 400 to 500 feet the velocity of turfed areas should be about 0.25 ft/sec. For bare ground the velocity should be intermediate between these values, depending on surface roughness. Emmett (1970) measured velocities of overland flow ranging from 0.02 to 0.05 ft/sec for rangeland hillslopes. As a general rule, velocities as determined by the time of concentration should be less than 8 ft/sec for small artificial channels and less than 6 ft/sec for small natural channels.

Selected methods which are considered applicable for estimating the various components of the time of concentration in the Montgomery area are presented in the discussion which follows. For design purposes, the minimum acceptable time of concentration in Montgomery is 5 minutes.

<u>4.2.2.1 Overland flow travel time, t_1</u>--Methods for estimating the overland flow travel time include the following:

1. Federal DOT equation
2. SCS overland flow method
3. Kerby equation

The <u>Federal DOT equation</u> was developed by the Federal Aviation Agency for Airport Drainage (Federal DOT, 1970). The Federal DOT equation provides an estimate of the overland flow travel time as follows:

$$t_1 = \frac{1.8\ (1.1 - C)\ D^{0.5}}{S^{0.33}} \tag{5-11}$$

where

t_1 = overland flow travel time, in minutes

C = runoff coefficient for the rational equation, dimensionless

D = distance of overland flow, in feet

S = overland flow slope, in percent

A graphical solution to equation 5-11 is presented on Figure 5-7.

The SCS overland flow method is the overland flow component of the SCS upland flow method, which is presented later as a method for estimating the inlet time. The SCS overland flow time is determined by first estimating the average overland flow velocity and then calculating the travel time, given the distance of travel. The overland flow velocity is estimated using the information presented on Figure 5-8, which requires a site-specific determination of the watershed overland slope in percent and the overland ground cover type. The overland flow travel time is then calculated using the following equation:

$$t_1 = \frac{L_1}{60v_1} \tag{5-12}$$

where

t_1 = overland flow travel time, in minutes

L_1 = length of overland flow path, in feet

v_1 = overland flow velocity estimated using Figure 5-8, in feet per second

The Kerby equation is based on information presented by Hathaway (1945) concerning the drainage of military airfields. The Kerby equation for estimating the overland flow travel time, t_1, is given as follows:

$$t_1 = 0.827 \left(\frac{NL_1}{S^{0.5}}\right)^{0.467} \tag{5-13}$$

where

t_1 = overland flow travel time, in minutes

N = Kerby equation roughness coefficient determined using Table 5-10

L_1 = length of overland flow path, in feet

S = overland flow slope, in feet per foot

It should be noted that the overland flow component may be significant in very small watersheds because other travel components are either small or zero. Conversely, the overland flow travel time should generally be estimated only for small overland flow areas since overland runoff is usually concentrated into small gullies or channels within several hundred feet of its origin.

In addition, it should be recognized that the overland path of flow is not always perpendicular to the natural watershed contours found on a topographic map. For example, when a development is graded such that overland drainage is intercepted by swales or diversion structures, this should be accounted for when the overland flow travel time is estimated.

4.2.2.2 Inlet Travel Time--As indicated by equation 5-10, the conveyance system inlet time may include two components. If the ditch and street gutter travel time is zero, the overland flow travel time is the inlet time for a particular conveyance system. Methods for estimating the conveyance system inlet time include the following:

1. SCS upland flow method

2. SCS curve number method

The SCS upland flow method considers flow of the following types: overland, through grassed waterways, over paved areas, and through small upland gullies. The overland flow component was presented previously. The procedure for estimating an inlet time is identical to that presented for overland flow. This involves using Figure 5-8 to estimate the velocity of flow for each flow component which contributes to the inlet time; however, equation 5-12 requires a slight modification as follows:

$$t_I = \sum_{i=1}^{n} \frac{L_i}{(60)\, v_i} \qquad (5\text{-}14)$$

where

t_I = conveyance system inlet time, in minutes

n = number of inlet flow segments

L_i = length of the ith inlet flow segment, in feet

v_i = average flow velocity for the ith segment of inlet flow, in fps

If the inlet time includes flow components in street gutters or paved ditches, equation 5-14 is still valid; however, average flow velocities cannot be estimated by the SCS upland method. Information concerning velocities of flow in street gutters is presented in Section 3 of Chapter 7. Average flow velocities in paved ditches can be estimated using Manning's equation, which is briefly discussed later in this section. A detailed discussion of Manning's equation is presented in Chapter 7.

The <u>SCS curve number method</u> for estimating a conveyance system inlet time was originally derived for application to undeveloped and agricultural watersheds. The method was intended to span a broad set of conditions ranging from steep to flat slopes and from heavily forested to smooth roughness characteristics. The curve number method was originally applicable to watershed areas less than 2,000 acres and for curve numbers greater than 50 and less than 95. The SCS has recently modified the curve number method so that it is applicable to small urban watersheds after development (i.e., curve numbers greater than 95).

The SCS curve number inlet time is estimated using the following two equations:

$$t_I = (1.67)\ t_L (60) \tag{5-15}$$

$$t_L = \frac{L^{0.8}\ (S + 1)^{0.7}}{1900\ Y^{0.5}} \tag{5-16}$$

where

t_I = conveyance system inlet time, (this value becomes t_c when the reach travel time is zero), in minutes

t_L = watershed lag time, in hours

L = hydraulic length of the watershed, in feet

Y = average watershed land slope, in percent.

$S = \frac{1,000}{CN} - 10$, (equation 5-2) where CN = SCS curve number

Information for estimating the watershed CN is discussed in Subsection 3.2 and is presented in Tables 5-4 and 5-5 and Figure 5-6.

When the SCS curve number method is used to estimate the inlet time for an urbanized watershed, two adjustment factors must be considered. These adjustment factors account for (1) the extent to which a natural stream channel has been modified and (2) the precentage of increased impervious area compared to natural conditions. The adjustment factors are presented independently on Figures 5-9 and 5-10. Figure 5-9 presents lag factors for adjusting the inlet time calculated from equation 5-15, when the percentage of natural hydraulic length which has or will be modified is known. The adjustment for channel improvements is made as follows. If 50 percent of the channel has been modified from its natural condition and the future condition curve number is computed to be 80, then the inlet time computed by equation 5-15 is multiplied by 0.7 (from Figure 5-9). Similarly, Figure 5-10 presents lag factors for adjusting the inlet time calculated from equation 5-15 given the percentage of impervious area which is added to the site after development. If the future-condition curve number is 100 or the impervious area is zero, no inlet time adjustment factor is required.

It must be noted that Figures 5-9 and 5-10 are used only with future-condition curve numbers; the t_I adjustment factors cannot be used to directly compute the decrease in t_I using present-condition curve numbers. To determine the change in t_I from present to future conditions, the present value is computed; then using future conditions, t_I is computed and adjusted using Figures 5-9 and 5-10 accordingly.

<u>4.2.2.3 Conveyance system travel time, t_3</u>--The conveyance system travel time is usually determined with Manning's equation. Manning's equation provides an estimate of the average velocity of flow in a non-pressure flow pipe or open channel and is expressed as follows:

$$v = \frac{1.49}{n} R^{0.67} S^{0.5} \qquad (5\text{-}17)$$

where

v = average flow velocity, in fps

n = Manning's roughness coefficient

R = hydraulic radius for a channel cross section, calculated as cross sectional area divided by wetted perimeter in feet (see Table 7-8)

S = slope of the hydraulic gradient (water surface) in feet per foot

A nomograph for solving equation 5-17 is presented on Figure 5-11. Further details on the application of Manning's equation for design of stormwater conveyance systems are presented in Chapter 7. Given the average velocity of flow in a particular conveyance system segment, the reach travel time is calculated using the following equation:

$$t_3 = \frac{L_3}{60\ v_3} \tag{5-18}$$

where

t_3 = reach travel time, in minutes

L_3 = reach length, in feet

v_3 = average velocity of flow in the reach, in fps (from equation 5-17)

A given watershed may have several components to its conveyance system, each of which may require an independent estimate of the average flow velocity in that reach. In this case, t_3 becomes the summation of several independent determinations of the reach travel time for each component.

4.2.2.4 Direct Computation of t_c--In addition to summing independent estimates of the conveyance system inlet and reach travel times, the Kirpich (1940) equation can be used to estimate the watershed t_c directly. The Kirpich equation is based on data reported by Ramser (1927) for six small agricultural watersheds near Jackson, Tennessee. The slope of these watersheds was steep, the soils were droughty, the timber cover ranged from zero to 56 percent, and watershed areas ranged from 1.2 to 112 acres. Because this data base is so limited and site-specific, the Kirpich formula should be used to estimate t_c for rural areas only. The Kirpich formula is given as follows:

$$t_c = 0.0078 \left(\frac{L}{S^{0.5}}\right)^{0.77} \tag{5-19}$$

where

t_c = time of concentration, in minutes

L = length of travel, in feet

S = slope, in feet per foot

4.2.3 Average Rainfall Intensity. As noted previously, prior to selecting the average rainfall intensity for the rational method, the design storm return period and watershed time of concentration must be determined. Guidance for determining these two variables has been presented above. Given these two variables, an average rainfall intensity in inches per hour is selected directly from the rainfall intensity-duration-frequency curves presented on Figure 4-2. For design purposes, the minimum acceptable time of concentration in Montgomery is 5 minutes.

4.2.4 Watershed Drainage Area. The watershed drainage area to be used in the rational equation is that drainage area, in acres, which is tributary to the design point and produces the maximum peak flow rate. For example, it may be possible for runoff from the portion of a watershed which is highly impervious to have a greater peak flow rate than the peak which is estimated when the entire watershed area is considered. To evaluate whether this condition is possible for a particular watershed, a time of concentration should be estimated for several watershed areal configurations, and the maximum peak flow rate should be selected for design purposes. In summary, when the rational method is used the actual watershed drainage area may require a reduction by disregarding those areas where flow time is too slow to add to the peak.

4.3 Unit Hydrograph Theory

4.3.1 Runoff Hydrographs. As recommended in Table 5-7, unit hydrograph theory should be utilized for estimating peak runoff rates when the following conditions exist:

1. Watershed areas are greater than 50 acres.

2. Major through-site open channels and large closed conduits are to be designed.

3. Stormwater detention/retention basins are to be designed.

The primary reason for these recommendations is that unit hydrograph theory provides an estimate of the entire runoff hydrograph. Therefore, any of the procedures presented later in Section 5.0 of this chapter can be utilized for estimating peak flows. When only the peak runoff rate is required, a graphical approach developed by the SCS and presented below should be considered.

4.3.2 SCS Graphical Approach. The SCS graphical peak flow method was developed using the SCS type II storm (see Figure 5-2) a watershed area of 1 square mile, a curve number of 75, and sufficient rainfall volume to produce 3 inches of runoff (USDA, SCS TR-55 [1975]). All computations were performed using the SCS TR-20 (1973) computer program. The output from these computer runs is presented on Figure 5-12 relating the peak discharge in cubic feet per second per square mile of drainage area per inch of runoff volume (csm/inch) to the watershed time of concentration.

The watershed time of concentration shown on Figure 5-12 can be estimated using any of the procedures presented previously in Subsection 4.2.2 of this chapter. However, it is important to note that the t_c value used on Figure 5-12 includes no flow routing through the conveyance system beyond the drainage area which contributes direct runoff to the hydrograph. In other words, the path of flow utilized for calculating the time of concentration should correspond to the drainage area which contributes direct runoff to the conveyance system. The reach travel time, t_3, for a particular conveyance system must be accounted for separately by hydrologic routing techniques.

The SCS graphical peak flow is determined by entering Figure 5-12 on the abscissa (x-axis) with the value of t_c and reading the peak discharge in cfs per square mile per inch (csm/inch) from the ordinate (y-axis).

The peak discharge can then be determined as the product of the ordinate from Figure 5-12, the watershed drainage area, and the runoff volume for the selected design storm return period. This procedure can be expressed mathematically as follows:

$$Q_T = U(t_c)AQ_{24}(T) \qquad (5\text{-}20)$$

where

Q_T = peak runoff rate, in cfs, for the design storm return period, T

$U(t_c)$ = unit peak runoff rate, in csm/inch, for a given watershed t_c (from Figure 5-12)

A = watershed drainage area which contributes direct runoff to the conveyance system, in square miles

$Q_{24}(T)$ = watershed runoff volume, in inches, resulting from a 24-hour storm of return period T (determined using equation 5-1)

4.4 Flood Frequency Regression Equations

Two independent sets of flood frequency regression equations are applicable in the Montgomery area. The first set was were derived by the U.S. Geological Survey for small streams in Alabama. These Alabama regression equations are based on the analysis of observed streamflow data from 43 gauging stations (34 in Alabama, four in Georgia, three in Mississippi, and two in Tennessee) and a rainfall-runoff model which was used to extend the observed flood peak record. The second set of regression equations was obtained from a Log-Pearson type III frequency analysis of observed streamflow data from 60 relatively small urban watersheds located throughout the U.S.

As recommended in Table 5-7, flood frequency regression equations should serve as an order-of-magnitude check on the reasonableness of peak flows determined by other methods presented in this section. Regression equations are generally best suited to watersheds which are not subject to a change from the hydrologic conditions (e.g., increased imperviousness, or time of concentration) which existed when the streamflow measurements were observed. Therefore, regression equations are generally best suited to medium or large watersheds prior to development. However, the second set of equations presented in this section was developed to account for the impact of urbanization on the peak flows estimated by the regression equations. An urbanization factor for watershed drainage systems was developed for this purpose.

4.4.1 Alabama Regression Equations. Flood frequency regression equations for small streams in Alabama are reported in Table 5-11. Experience has indicated that these regression equations may not be applicable to the West End Ditch. This is probably due to the higher channel storage characteristics of West End Ditch compared to those of similar drainage areas in Montgomery. The contributing drainage area and channel slope are the only parameters that need to be determined for these equations. The channel slope is determined from the difference in elevations at points 10 and 85 percent of the distance along the main channel from the discharge site to the drainage divide. This difference in elevation is divided by the main channel length between the two points. Drainage areas used to derive these Alabama regression equations ranged from 346 to 10,176 acres, and channel slopes ranged from 12.7 to 286.2 ft/mile. These regression equations should not be used where manmade structures significantly affect peak discharges or where runoff is significantly altered by urbanization.

4.4.2 Urban Area Regression Equations. Espey and Winslow (1974) derived a set of urban watershed flood frequency regression equations utilizing published streamflow gauging station records for 60 urban watersheds located throughout the U.S. (27 in Texas, 26 on the east coast, two in Michigan, one in Illinois, and four in Mississippi). These urban watershed regression equations are presented in Table 5-12. Since they were derived using the Log-Pearson type III frequency distribution model, the mean annual peak flow has a return period of 2.33 years. The peak flow for a specified return period is estimated by determining the drainage area, impervious cover, 6-hour duration rainfall depth, main channel slope, and a conveyance system urbanization factor for the subject watershed. The conveyance system urbanization factor is comprised of two components; one accounts for channel characteristics, while the other accounts for vegetation characteristics. Once a value for each of these two components has been determined from data presented in Table 5-13, the conveyance system urbanization factor is determined using the following equation:

$$\Phi = \Phi_1 + \Phi_2 \tag{5-21}$$

where

Φ = conveyance system urbanization factor

Φ_1 = drainage system improvement factor

Φ_2 = channel vegetation factor

An alternative method for determining the conveyance system urbanization factor is presented on Figure 5-19. Given the weighted main channel Manning's n value and the watershed impervious cover, in percent, a value of the conveyance system urbanization factor can be obtained directly from Figure 5-19.

Despite the relatively short period of record for most of the urban watersheds considered in this analysis, the data can be used to form the basis for tentative conclusions regarding the flood frequency characteristics of urban watersheds. According to Espey and Winslow (1974) the data, as a whole, provide a stronger data base than that of individual watershed records for the development of empirical urban watershed flood frequency regression equations. They also note that the application of these equations to urban watersheds for the 50-year flood should be used cautiously and reviewed in light of historical rainfall or streamflow data in the area. Since a 25-year regression equation is not presented, these equations are applicable only for private property drainage systems serving areas less than 15,000 square feet (i.e., a 10-year design storm can be used).

4.5 Historical Highwater Marks

As indicated in Table 5-7, observed high water marks from historical floods can be utilized to provide an order-of-magnitude check on pre-development peak flows determined by other methods. Information related to historical high water marks is generally available from the U.S. Geological Survey or Montgomery Engineering Department. It is important to note that historical high water marks provide an independent check for pre-development peak flows only, and it is not always possible to assign a return period to the observed high water mark. Therefore, when available, historical high water marks provide an order-of-magnitude check on pre-develoment peak flows only.

Given the historical high water mark, a peak flow can be estimated using the following equation:

$$Q = A_x v \tag{5-22}$$

where

Q = estimate of the observed peak flow, in cfs

A_x = cross sectional area of the waterway section where the high water mark was observed, in ft^2

v = estimate of the mean velocity, using Manning's equation (see equation 5-17), in fps

4.6 Example Problems

Example 5-4. Runoff Coefficient Computations

Determine the runoff coefficient for ultimate land use conditions on subbasin 2 of West End Ditch, using a 25-year, 24-hour design storm.

1. From example 5-3, the composite CN for ultimate conditions on subbasin 2 is 84. Therefore:

$$S = \frac{1,000}{84} - 10 = 1.90 \text{ inches}$$

2. From Figure 4-2, the depth of a 25-year, 24-hour design storm P_T = 7.53 inches. Using equation 5-7, the 25-year, 24-hour runoff coefficient is:

$$C_T = 1 - \frac{1.90}{7.53}\left[1.2 - \frac{1.90}{7.53 + (0.8)(1.90)}\right]$$

$$C_T = \underline{\underline{0.75}}$$

Example 5-5. Time of Concentration Computations

The hydraulic path of stormwater flow in subbasin 2 of West End Ditch is approximately 4,800 feet in length with a total elevation change of approximately 13.6 feet. This hydraulic pathway can be broken into the following three segments:

Segment	Type of Flow	Segment Length (feet)	Elevation Change (feet)	Slope (%)
1	Overland (short grass)	400	6	1.5
2	Small Ditch n = 0.032 width = 4 feet depth = 2 feet	1,500	3.8	0.25
3	Vegetated Open Channel n = 0.025 width = 10 feet depth = 4 feet	2,900	3.8	0.13

Compute the various components of the time of concentration for ultimate land use in subbasin 2 and select a reasonable value.

1. Compute the overland flow travel time, t_1. According to equation 5-11:

   Do.T. Eqn

   $$t_1 = \frac{1.8(1.1 - 0.75)(400)^{0.5}}{(1.5)^{0.33}} = 11.0 \text{ minutes}$$

   Given a slope of 1.5 percent and short grass, using Figure 5-8, v_1 is found to equal 0.85 fps. Therefore, according to equation 5-12:

   S.C.S.

   $$t_1 = \frac{(400)}{(60)(0.85)} = 7.8 \text{ minutes}$$

   Using Table 5-10, N for equation 5-13 is approximately 0.20; therefore:

   $$t_1 = 0.827 \left(\frac{(0.20)(400)}{(0.015)^{.5}}\right) = 17.1 \text{ minutes}$$

2. Compute the inlet time to the main vegetated channel.

   Using the SCS upland flow method v_2, the small ditch velocity can be estimated using Figure 5-11. Given that n = 0.032, S = 0.0025 ft/ft, and R = 1.0, v = 2.3 fps; therefore:

$$t_2 = \frac{1{,}500}{(60)(2.3)} = 10.9 \text{ minutes}$$

and according to equation 5-14:

$$t_I = 7.8 + 10.9 = 18.7 \text{ minutes}$$

Using the SCS curve number method, equation 5-16 is used to calculate a watershed lag time as follows:

$$S = \frac{1{,}000}{84} - 10 = 1.90 \text{ inches}$$

$$t_L = \frac{(1{,}900)^{0.8}(1.90 + 1)^{0.7}}{(1{,}900)(1.5)^{0.5}} = 0.38 \text{ hours}$$

$$t_I = (1.67)(0.38)(60) = 38.1 \text{ minutes}$$

To account for channel improvements and impervious area, Figures 5-9 and 5-10 are used. Assuming that 50 percent of the hydraulic length is modified and that the area is 50 percent impervious, yields the following inlet time:

$$t_I = (38.1)(0.75)(0.75) = 21.4 \text{ minutes}$$

3. Compute the conveyance system travel time using Figure 5-11. Given that n = 0.025, S = 0.0013 ft/ft, and R = 1.87 feet, v = 3.3 fps. Therefore, using equation 5-18:

$$t_3 = \frac{(2{,}900)}{(60)(3.3)} = 14.6 \text{ minutes}$$

Since the drainage area is greater than 50 acres for subbasin 2, consider the SCS curve number approach as a reasonable estimate of t_I; therefore:

$$t_c = 21.4 + 14.6 = \underline{\underline{36.0 \text{ minutes}}}$$

For comparison purposes, t_c, determined using equation 5-19, is as follows:

$$t_c = 0.0078 \left(\frac{4{,}800}{(0.0028)^{0.5}}\right)^{0.77} = 51.2 \text{ minutes}$$

Since equation 5-19 was derived for rural conditions, it is likely to give a higher t_c than methods adapted for urban conditions. The average velocity for a t_c of 51.2 minutes is 1.4 fps, whereas for 36 minutes, it is 2.2 fps; therefore, 36 minutes seems reasonable for design purposes.

Example 5-6. Comparison of Peak Flow Procedures

Compute the ultimate land use peak runoff rate from subbasin 2 of West End Ditch for a 25-year design storm using the SCS graphical approach from TR-55. Compare this peak runoff rate to that calculated according to the rational equation. The drainage area of subbasin 2 is 0.268 square miles or 172 acres.

1. From Figure 4-2, the 25-year, 24-hour rainfall depth is 7.53 inches. For an ultimate land use CN of 84:

$$Q = \frac{[7.53 - (0.2)(1.90)]^2}{7.53 + (0.8)(1.90)} = 5.65 \text{ inches}$$

2. Using a t_c of 0.6 hours as determined by example 5-5 and Figure 5-12, the unit peak runoff rate for subbasin 2 is 450 csm/inch.

3. Using equation 5-20, the peak runoff is calculated as follows:

$$Q_T = (450)(0.268)(5.65) = \underline{\underline{681 \text{ cfs}}}$$

4. Since the drainage area of subbasin 2 is greater than 50 acres, the rational equation is not acceptable for design purposes in Montgomery. However, it can be used for comparison purposes.

   Using Figure 4-2 and a t_c of 36 minutes, $I_T(t_c) = 4.4$ inches/hour. From example 5-4, $C_T = 0.79$. Therefore, according to equation 5-5:

$$Q_T = (0.75)(4.4)(172) = 568 \text{ cfs}$$

Since this peak flow is determined for ultimate land use conditions, the flood frequency regression equation and historical high water mark procedures are not applicable.

SECTION 5.0 RUNOFF HYDROGRAPHS

A runoff hydrograph is a continuous plot of the surface runoff flow rate versus time. It is produced when a particular watershed is subjected to an input of excess rainfall. By definition, the volume of water contained in the runoff hydrograph is equal to the volume of water contained in the excess rainfall hyetograph. Since the subject watersheds in urban hydrology rarely have observed runoff hydrographs, synthetic methods for developing runoff hydrographs are usually required. It is the purpose of this section to document synthetic methods for developing design runoff hydrographs when observed runoff records are unavailable.

The engineer has a choice of several procedures when developing a runoff hydrograph for design purposes. These range from simple desktop procedures to complex computer programs. The desktop procedures which are considered in this section include the rational method hydrograph procedure, an empirical approach, unit hydrograph theory, and the SCS tabular method. A brief summary of hydrologic computer models which are readily available to the engineer is presented in Appendix C of this manual.

Of the four desktop procedures presented in this section, the rational method is the most simplistic and susceptible to inaccuracies on large watersheds. It is, therefore, recommended that the rational method be utilized only on watersheds with drainage areas of 20 acres or less.

This section begins with a discussion of appropriate hydrograph terminology. The rational method hydrograph procedure is then presented, followed by a presentation of unit hydrograph theory and several synthetic unit hydrograph procedures. The section concludes by presenting three methods for developing synthetic runoff hydrographs when the rational method hydrograph is not appropriate (i.e., the watershed drainage area is greater than 20 acres).

5.1 Hydrograph Terminology

A typical hydrograph resulting from an isolated period of rainfall consists of a rising limb, a crest segment, and a falling or recession limb. These regions of the hydrograph are illustrated on Figure 5-13. The shape of the rising limb is influenced primarily by the characteristics of the storm which produced surface runoff. The point of inflection on the recession limb of the hydrograph is commonly assumed to mark the time at which surface inflow to the channel system ceases. Thereafter, the recession curve represents the withdrawal of water from storage within the watershed. As a result, the recession limb is largely independent of the storm and influenced by watershed characteristics such as channel slope and storage availability.

Surface runoff can be contributed to a hydrograph via four main routes of travel: (1) overland flow, (2) interflow or subsurface flow, (3) ground water, and (4) channel precipitation. The distinctions drawn between these four components of flow are arbitrary. For convenience, it is common practice to consider the total surface runoff to be divided into two parts, the storm (or direct) runoff and base flow. Direct runoff thus includes overland flow, subsurface flow, and channel precipitation, while base flow is considered to be ground water. For the purposes of this manual, synthetic methods for developing direct runoff hydrographs only will be presented. If site-specific data indicate that a significant base flow exists for the watershed of concern, an observed base flow component should be added to the synthetic direct runoff hydrograph prior to the hydraulic design of a stormwater system.

The hydrograph terminology which is utilized throughout this manual is presented on Figure 5-13, along with appropriate SCS hydrograph equations. An excess rainfall hyetograph, which in this case is a single block of excess rainfall over a duration D, is shown in the upper part of Figure 5-13. The runoff hydrograph is presented directly below the excess rainfall 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, t_p. The total time duration of the hydrograph is known as the base time, t_b. The watershed lag time, t_L, is defined as the time from the center of mass of the excess rainfall to the runoff hydrograph peak. Other definitions for the watershed lag time which are not used in this manual are (1) the difference in time between the center of mass of the excess rainfall and the center of mass direct runoff produced and (2) the time interval from the maximum rainfall rate to the peak rate of runoff. The following equation summarizes the relationship between the time parameters for the rising limb of a direct runoff hydrograph:

$$t_p = \frac{D}{2} + t_L \qquad (5\text{-}23)$$

where

t_p = time to peak or time of rise of the runoff hydrograph, in hours

D = duration of runoff-producing rainfall, in hours

t_L = watershed lag time, in hours (determined using equation 5-16 or equation 5-26)

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

$$t_r = t_b - t_p \tag{5-24}$$

where

t_r = hydrograph recession time

t_b = hydrograph time base

(t_p is defined above)

5.2 Rational Method Hydrographs (drainage areas of 20 acres or less)

The rational method was developed about 100 years ago for the sole purpose of predicting peak flow rates for small watersheds. It does not provide any information pertaining to the runoff hydrograph shape. As noted previously in Section 4.0 of this chapter, the rational method is a valid hydrologic design tool for predicting peak flow rates from watershed areas up to 50 acres. However, when a runoff hydrograph is required, the rational method should be limited to watershed areas of 20 acres or less.

5.2.1 Storm Durations Equal to t_c. Consider a design storm with a constant average rainfall intensity, I, uniformly distributed over a particular watershed area, A, for a duration, D. If this storm duration were assumed to equal the time of concentration for that wathershed, a peak flow for that design storm could be estimated using the rational equation (equation 5-5). This peak flow would occur at the time rainfall stops since the storm duration is equal to the watershed time of concentration. If a triangular approximation to the hydrograph is assumed such that the volume of excess rainfall equals the volume of runoff, an equilateral triangle is an approximate runoff hydrograph for that design storm. This equilateral triangle has a base which is equal to two times t_c and the peak flow Q_T as given by the rational equation. This triangular hydrograph for a rainfall duration equal to t_c is illustrated on Figure 5-14. Since an equilateral triangle does not approximate the observed fact that the recession limb of a runoff hydrograph is typically longer than the rising limb, the rational method hydrograph is best suited to small watersheds where the resistance to flow is limited. As noted previously, this area is recommended as 20 acres or less.

5.2.2 Storm Durations Greater Than t_c. Once a design storm return period has been selected, a family of runoff hydrographs can be developed using the rational method. A family of such hydrographs is illustrated on Figure 5-15. When a conveyance system (e.g., pipes or open channels) is being designed, the rational method triangular hydrograph approximation with a design storm duration equal to the watershed time of concentration is selected from this family of curves, since the maximum expected flow is required for design purposes. However, if an allowable outflow rate from the watershed has been established, storm durations greater than the watershed time of concentration must be considered. The difference in area between the desired outflow hydrograph and each of the trapezoidal hydrographs must be determined as a preliminary estimate of the storage volume required to meet the peak outflow requirements. For example, according to the family of rational method hydrographs shown on Figure 5-15, a storm duration of 30 minutes produces the maximum storage volume required to produce the desired peak outflow of 12.5 cfs. More details concerning the design of stormwater storage systems are presented in Chapter 8.

5.3 Unit Hydrograph Theory

The concept of a unit hydrograph was published by L. K. Sherman in 1932. Sherman assumed that, for a given rainfall duration, the time base of the direct runoff hydrograph was constant for a given watershed. Although the tools and data available for developing unit hydrographs have expanded greatly since Sherman first proposed unit hydrograph theory, the concept has changed little.

A unit hydrograph is defined as a runoff hydrograph which is produced by 1 inch (25.4 mm) of excess rainfall distributed uniformly over a watershed and occuring at a uniform rate during a specified period of time. Sherman originally used the word "unit" to denote the specified duration of excess rainfall for a particular unit hydrograph. The word "unit" is often misinterpreted to represent 1 inch or a "unit depth" of effective rainfall rather than a "unit of time" for effective rainfall as originally intended.

The following assumptions constitute the basis of unit hydrograph theory:

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

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

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

4. The ordinates of the direct runoff hydrographs, when a common base time is considered, are directly proportional to the total volume of direct runoff represented by each hydrograph (i.e., principle of linearity or superposition).

5. For a given drainage basin, the hydrograph of runoff due to a given duration and volume of excess rainfall is invariable (i.e., principle of time invariance).

The above assumptions cannot precisely apply to natural rainfall and drainage basin characteristics. However, experience has shown that the unit hydrograph method gives results which are sufficiently accurate for most practical problems in stormwater management.

Two fundamental assumptions which must be kept in mind when applying unit hydrograph theory are the principle of linearity (assumption 4) and the principle of time invariance (assumption 5). Theoretically, each increment of excess rainfall can be routed through the subject watershed in accordance with the principle of linearity. In practice, however, linearity means that the product of a excess rainfall volume and the sequence of unit hydrograph ordinates (i.e., runoff rates in cfs per inch of excess rainfall) produces an estimate of the runoff hydrograph for that volume of excess rainfall. In addition, the principle of linearity allows the user to superimpose individual runoff hydrographs developed from a sequence of individual rainfall excess volumes (e.g., a design storm of rainfall excess increments arranged in units of time equal to the unit duration) and add them to estimate a total runoff hydrograph. The principle of time invariance requires that the condition of the drainage basin be fixed or specified for a particular unit hydrograph.

The unit hydrograph method was originally devised for large drainage basins. However, Brater (1940) showed that unit hydrograph theory was also applicable to small drainage basins varying in size from 4 acres to 10 square miles. Unit hydrographs should not be applied to runoff originating from snow or ice, nor when the duration of excess rainfall is greater than the time of rise for the appropriate unit hydrograph.

In practice, unit hydrographs can be developed either (1) from observed rainfall and streamflow records, or (2) from observed watershed characteristics and synthetic unit hydrograph procedures. This manual will limit the discussion of unit hydrographs to synthetic procedures, since observed rainfall and streamflow data are generally not available. If observed data are available, the manual user is referred to Wisler and Brater (1959), Chow (1964), or Viessman, et al., (1977) for procedures to develop observed unit hydrographs.

5.4 Synthetic Unit Hydrographs

A unit hydrograph for an ungauged urban stream can be estimated if the information derived from the analysis of gauging records is related to measurable watershed characteristics such as channel slope, channel length, and drainage area etc. The procedure generally requires that the unit hydrograph time parameters, peak flow, and shape or time distribution be estimated. The synthetic procedures presented in this manual include (1) SCS dimensionless unit hydrographs (2) unit hydrograph regression equations for small urban areas.

5.4.1 SCS Dimensionless Unit Hydrographs. The SCS dimensionless unit hydrographs were derived from a large number of observed unit hydrographs for watersheds varying widely in size and geographic locations. Two types of dimensionless unit hydrographs have been developed by the SCS; the first has a curvilinear shape, and the second is a triangular approximation to that curvilinear shape. In both cases, once the time to peak and peak flow for a particular unit hydrograph have been defined, the entire shape of that unit hydrograph can be estimated using the appropriate dimensionless unit hydrograph.

Figure 5-16 presents a dimensionless curvilinear unit hydrograph with 37.5 percent of its total volume on the rising side, which is one unit of time and one unit of discharge. The triangular approximation to this curvilinear hydrograph has the same percent of volume on the rising side of the triangle. Therefore, one unit of time, t_p, equals 0.375 units of the discharge volume and the time base of this triangle is 2.67 units of time (1.0/0.375), while the recession limb is 1.67 units of time ($t_b - t_p$).

Having established the shape and the time axis for the dimensionless unit hydrograph, there is only one value for the peak flow, q_p, which will make the volume under the unit hydrograph equal to 1 inch of excess rainfall. This peak is determined by the following equation:

$$q_p = \frac{484\ A}{t_p} \qquad (5\text{-}25)$$

where

q_p = unit hydrograph peak discharge, in cfs

A = drainage area, in mi²

t_p = time to peak, in hours (i.e., time from the beginning of rainfall excess to the peak flow rate)

Any change in the dimensionless unit hydrograph to reflect observed changes in the percent of volume under the rising limb of the hydrograph would cause a corresponding change in the shape factor associated with the triangular hydrograph and therefore a change in the constant, 484. This constant has been known to vary from 600 in steep terrain to 300 in very flat swampy terrain. If site conditions indicate the need to vary the dimensionless shape of the unit hydrograph, the ratio of the percent of total volume on the rising side of the unit hydrograph triangle to the rising side of a triangle for observed conditions can be a useful tool in arriving at the peak factor (USDA SCS NEH-4, 1972).

The dimensionless unit hydrograph shown on Figure 5-16 has a time to peak at one unit of time and the point of inflection at appoximately 1.7 units of time. Using the SCS average relationship of lag time to time of concentration which is given as follows,

$$t_L = 0.6\ t_c \qquad (5\text{-}26)$$

and the point of inflection given as 1.7 t_p, the following relationships are obtained.

$$t_c + \Delta D = 1.7\ t_p \qquad (5\text{-}27)$$

$$t_p = \frac{\Delta D}{2} + 0.6\ t_c \qquad (5\text{-}28)$$

which can be solved simultaneously to give:

$$\Delta D = 0.133\ t_c \qquad (5\text{-}29)$$

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

1. Estimate t_c using any appropriate method from Section 4.2.2 of this chapter.

2. Calculate ΔD using equation 5-29.

3. Calculate t_p using equation 5-28.

4. Calculate q_p using equation 5-25 with $Q = 1.0$.

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

6. Using Table 5-14 or Figure 5-17, find the $1/q_p$ 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 ratios by q_p.

8. Determine the volume under the unit hydrograph to ensure that it is equal to 1 inch.

Example 5-7 illustrates the SCS curvilinear dimensionless unit hydrograph procedure. 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 from Figure 5-16. When applying the triangular dimensionless unit hydrograph, the following relationships are required:

$$\Delta D = 0.24\ t_c \tag{5-30}$$

$$t_p = 3\Delta D \tag{5-31}$$

$$t_b = 2.67\ t_p \tag{5-32}$$

If $\Delta D \geq 0.5$ from equation 5-30, then:

$$\Delta D = 0.11\ t_c \tag{5-33}$$

$$t_p = 6\Delta D \tag{5-34}$$

t_b is determined using equation 5-32.

If further details of the triangular approximation to the SCS curvilinear unit hydrograph are required, the user is referred to Section 4 of the National Engineering Handbook (USDA, SCS NEH-4, 1972).

5.4.2 Unit Hydrograph Regression Equations. Unit hydrograph regression equations for small urban areas have been derived for two unit hydrograph durations, i.e., these are 10-minute and 30-minute durations. Each of these procedures is described in the following discussion.

5.4.2.1 Ten-Minute Regression Equations--The 10-minute unit hydrograph regression equations for small urban areas are published in an ASCE technical memorandum by Espey et al., (1977). The derived 10-minute regression equations were based on observed data from a total of 41 urban watersheds located throughout Texas, Tennessee, Mississippi, Pennsylvania, North Carolina, Colorado, Kentucky, and Indiana. The following hydrologic parameters were chosen to describe the shape of the 10-minute unit hydrograph:

T_R = time or rise, in minutes

Q = the peak discharge, in cfs

T_B = the time base, in minutes

W_{50} = the time, in minutes, between two points on the unit hydrograph at which the discharge is one-half of the peak discharge, Q_{50}

W_{75} = the time, in minutes, between the two points on the unit hydrograph at which the discharge is three-fourths of the peak discharge, Q_{75}.

These five unit hydrograph shape parameters are illustrated on Figure 5-18.

Five physiographic characteristics of a specific watershed are required as input to the regression equations for estimating the 10-minute unit hydrograph parameters described above. As presented and defined in Table 5-15, these physiographic characteristics are the main channel length (L), the main channel slope (S), the extent of impervious cover (I), a dimensionless watershed conveyance factor (Φ), and the watershed drainage area (A).

The watershed conveyance factor was considered in order to account for the reduction in the time of rise for a unit hydrograph due to channel improvements or storm sewers. As presented in Table 5-13, the watershed conveyance factor is comprised of two components, one to account for channel characteristics (Φ_1) and the other to account for vegetation characteristics (Φ_2). Once a value for each of these two components has been determined from data presented in Table 5-13, the watershed conveyance factor is determined using equation 5-21. As noted previously, Figure 5-19 can be utilized as an alternative method for determining the conveyance system urbanization factor.

Having obtained satisfactory site-specific values for these characteristics, the 10-minute unit hydrograph shape parameters can be estimated using the regression equations of

Table 5-15. The 10-minute unit hydrograph for that watershed is then constructed by distributing 1 inch of runoff volume under a hydrograph such that the calculated unit hydrograph parameters are accounted for. The dimensionless SCS unit hydrographs can be used as an aid for constructing the unit hydrograph accordingly. Generally, the lower portions of the ascending and descending unit hydrograph limbs are adjusted to obtain the 1-inch volume of runoff.

5.4.2.2 Thirty-Minute Regression Equations--The 30-minute unit hydrograph regression equations are very similar to the 10-minute equations; however, they do not include the watershed conveyance factor, Φ. These equations were published by Hamm et al., (1973) and are presented in Table 5-16. Separate sets of equations were derived for watersheds that have been extensively developed ($I \geq 20\%$), watersheds that have been developed to a lesser extent ($I < 20\%$), and watersheds for all ranges of developement. These equations are based on observed data from 37 urban watersheds located throughout Mississippi, Pennsylvania, North Carolina, Texas, Colorado, and Tennessee. A procedure similar to that presented for constructing the 10-minute unit hydrographs can be utilized for the 30-minute unit hydrographs.

5.5 Synthetic Runoff Hydrographs

When a rational method hydrograph is not appropriate for the hydraulic design of a stormwater system (i.e., the watershed drainage area is greater than 20 acres), a more detailed synthetic hydrograph is required. Three methods to develop synthetic runoff hydrographs are presented in this manual: (1) an empirical hydrograph, (2) application of unit hydrograph theory, and (3) tabular runoff hydrographs developed by the SCS.

The empirical approach is applicable only to preliminary design situations since it requires that the peak flow be determined as an input to the computation procedure. However, the empirical approach can serve as a guide for plotting a unit hydrograph developed using the 10-minute or 30-minute synthetic unit hydrograph equations presented in subsection 5.4.2.

Unit hydrograph theory is best suited for final design computations when a runoff hydrograph is required since each increment of excess rainfall is explicitly routed through the subject watershed.

The SCS tabular method presents a compilation of computer simulations using 24-hour, type II design storm. Since the 24-hour design storm nests the most intense rainfall in the middle of the storm, antecedent conditions are such that high peak flows are expected; therefore, the SCS tabular method should provide a conservative estimate of peak flows and is best suited for preliminary design problems. A comparison of peak flows obtained using the SCS procedures, unit hydrograph theory, and the rational equation is presented in example 5-9.

<u>5.5.1 Empirical Approach</u>. A shortcut empirical hydrograph approach for the preliminary design of stormwater detention facilities was presented by Malcom and New (1979). This empirical approach requires the user to develop an estimate of the hydrograph peak flow rate and total runoff volume for the design storm in question. Therefore, the empirical approach should be used only for preliminary design situations since it requires that the peak flow be determined as an input to the computation procedure. In other words, the empirical approach does not provide any new information related to the peak flow of a hydrograph.

The total runoff volume and peak flow rate for the hydrograph in question can be estimated using any of the methods presented in Sections 3.0 and 4.0 of this manual. Having developed an estimate of the hydrograph peak flow rate and runoff volume, a step-function empirical approximation to the dimensionless hydrograph presented by the Bureau of Reclamation (1973) and the SCS (1972) is used to develop a synthetic runoff hydrograph for the storm. The empirical synthetic runoff hydrograph is developed as follows:

1. Given the total runoff volume and peak flow for the given design storm on a watershed, the time to peak, t_p, is calculated as follows:

$$t_p = \frac{Q}{1.37\ Q_T} \qquad (5\text{-}35)$$

   where

   t_p = time to peak, in seconds

   Q = total runoff volume, in ft³

   Q_T = peak runoff rate, in cfs, for the design storm return period, T

2. Given the time to peak and peak flow, a synthetic runoff hydrograph is developed using the following two equations:

For $0 \le t \le 1.25\ tp$

$$Q(t) = \frac{Q_T}{2} \quad 1 - \cos \frac{\pi t}{t_p} \qquad (5\text{-}36)$$

For $t > 1.25\ t_p$

$$Q(t) = 5.37\ Q_T\ e^{-1.42 t/t_p} \qquad (5\text{-}37)$$

where

$Q(t)$ = hydrograph flow, in cfs, at the time in question

t = time in question, in seconds

5.5.2 Unit Hydrograph Theory. In general, the application of unit hydrograph theory to develop a synthetic runoff hydrograph consists of developing a design rainstorm (Section 2.0), deducting abstractions from that storm to obtain an excess rainfall hyetograph (Section 3.0), defining a synthetic unit hydrograph for the watershed (subsection 5.4), and routing the excess rainfall hyetograph through the watershed by applying the principle of linearity. Since each of these steps, except the last, have been presented previously, it is the purpose of this section to develop the procedure for routing an excess rainfall hyetograph through a watershed using unit hydrograph theory.

As presented in subsection 5.3, the principle of linearity means that for a common time base, the ordinates of all direct runoff hydrographs are directly proportional to the total volume of direct runoff (i.e., excess rainfall) generated by each storm considered. In practice, linearity means that the product of an excess rainfall volume and the sequence of unit hydrograph ordinates (i.e., runoff rates in cfs per inch of excess rainfall) produces an estimate of the runoff hydrograph for that volume of excess rainfall. In addition, linearity allows the user to superimpose individual runoff hydrographs developed from a sequence of individual rainfall excess volumes (i.e., an excess rainfall design storm arranged in time increments equal to the unit hydrograph duration) and add them to estimate a total runoff hydrograph for the design storm.

A synthetic runoff hydrograph is developed using unit hydrograph theory by the following steps:

1. Develop a unit hydrograph for the subject watershed using an appropriate procedure as presented in subsection 5.4 of this chapter.

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 2.0 of this chapter.

3. Develop an excess rainfall hyetograph using the SCS curve number approach as presented in Section 3.0 of this chapter.

4. Route the excess rainfall hyetograph through the subject watershed by multiplying the ordinates of the unit hydrograph by the respective excess rainfall increments. Thus, each increment of excess rainall produces a routed incremental hydrograph. Each routed incremental hydrograph is delayed by the design storm time interval (see Table 5-17 and example 5-8).

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 (see Table 5-17 and example 5-8). The routed incremental hydrographs of example 5-8 are plotted on Figure 5-20 so that the process of superposition can be visualized. Note that each routed incremental hydrograph begins at the same time as the increment of excess rainfall which produced it.

6. Check to ensure that the volume of the synthetic runoff hydrograph is equal to the volume of excess rainfall (use equation 5-38).

It is important to note that if a short-duration unit hydrograph is utilized to develop a long-duration synthetic hydrograph, the actual shape of the unit hydrograph is not nearly as important as the time to peak and peak flow rate for that unit hydrograph. It is, therefore, likely that a triangular unit hydrograph would produce approximately the same synthetic runoff hydrograph as a curvilinear unit hydrograph.

Regardless of the method used for determining the shape of a unit hydrograph, the final step of this procedure is to determine the volume of runoff actually contained under the synthetic runoff hydrograph. This calculation is performed to ensure that the runoff hydrograph has the same volume as the excess rainfall hyetograph which produced it. If the hydrograph flow is in cfs and the time measured in seconds, the volume under the hydrograph in inches is:

$$V = \frac{12\ \Delta t\ \Sigma q_i}{A(43560)} \qquad (5\text{-}38)$$

where

V = volume under the hydrograph, in inches

Δt = time increment of the runoff hydrograph ordinates, in seconds

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

A = watershed drainage area, in acres

A shortcut procedure for applying unit hydrograph theory when developing a synthetic runoff hydrograph was presented by the SCS (1972). This procedure is illustrated in Table 5-18 and is described in the following sequence of steps:

1. Tabulate the unit hydrograph time increments and ordinates in columns 1 & 2, respectively.

2. Record the excess rainfall increments in reverse order on a strip of paper having the same line spacing as the Table of Step 1.

3. Place the strip of paper at the top of column 3 as shown in Table 5-18 and multiply the unit hydrograph ordinate by the excess rainfall increment and record the product in column 3.

4. Slide the strip of paper down one row, perform the same multiplication for the first two lines, sum these two products, and record the sum in column 3.

5. Continue to slide the strip of paper down one row at a time, sum the products of each line, and record them in column 3. This process continues until all increments of excess rainfall have been multiplied by each ordinate of the unit hydrograph.

5.5.3 SCS Tabular Method. The SCS tabular hydrograph method packages the results of extensive hydrologic calculations obtained using a computer, to quickly approximate a synthetic runoff hydrograph. This tabular method provides an approximation of the runoff hydrograph which would have been developed if the SCS computer program TR-20 were applied to the subject watershed. The tabular method was developed using TR-20 to compute hydrographs from 1 square mile of drainage area for a range of watershed times of concentration

(0.10 to 2.0 hours) using a 24-hour, SCS type II design storm (Section 2.0). A constant runoff curve number of 75 and a rainfall volume large enough to produce 3 inches of excess rainfall were utilized for these computer simulations.

The SCS tabular hydrograph data for a 24-hour, type II design storm are presented in Table 5-19. To use the SCS tabular method, the following information is required:

1. Watershed drainage area, in square miles.
2. Watershed time of concentration, in hours.
3. Composite runoff curve number for the watershed.
4. Twenty-four-hour rainfall depth, in inches, for the desired design storm return period, T.
5. Excess rainfall or direct runoff from the watershed, in inches.

Having obtained the above information, a synthetic runoff hydrograph is estimated as follows:

1. Identify the tabular runoff hydrograph flow rates, in csm/in, which are associated with the appropriate watershed times of concentration listed in Table 5-19.
2. Multiply the tabular runoff hydrograph flow rates identified in step 1 by the watershed drainage area and runoff according to the following equation:

$$Q_i = U_i(t_c)\ AQ_{24}(T) \qquad (5\text{-}39)$$

where

Q_i = synthetic runoff hydrograph ordinate, in cfs, for time period i

$U_i(t_c)$ = SCS TR-55 tabular runoff hydrograph ordinate from Table 5-19 for time of concentration, t_c, and time period i, in csm/in (i.e., cfs per square mile per inch of runoff)

A = watershed drainage area, in square miles

$Q_{24}(T)$ = watershed runoff volume, in inches, resulting from a 24-hour storm of return period, T (determined using equation 5-1)

The limitations of the SCS tabular method are very clearly stated in SCS TR-55 (1975) as follows:

> "The tabular method should not be used when large changes in the curve number occur among subareas within a watershed and when runoff volumes are less than about 1.5 inches for curve numbers less than 60. For most watershed conditions, however, this procedure is adequate to determine the effects of urbanization on peak rates of discharge for subareas up to approximately 20 square miles in size."

5.6 Example Problems

Example 5-7. SCS Synthetic Unit Hydrograph Computations

Develop a synthetic unit hydrograph for ultimate land use conditions on subbasin 2 of West End Ditch using the SCS curvilinear approach.

1. Using a t_c of 0.60 hours as determined for example 5-5, ΔD is calculated using equation 5-29 as follows:

$$\Delta D = (0.133)(0.60) = 0.08 \text{ hours}$$

2. Using equation 5-28, t_p is calculated as follows:

$$t_p = \frac{0.08}{2} + (0.6)(0.60) = 0.40 \text{ hours}$$

3. The unit hydrograph peak flow is calculated using equation 5-25 as follows:

$$q_p = \frac{(484)(0.268)}{(0.40)} = \underline{324 \text{ cfs}}/\text{inch}$$

4. List the hydrograph time, t, in increments of D and calculate the ratio of the time in question to the time to peak as shown in the table below in columns 1 and 2.

5. Using Table 5-14 and Figure 5-17, find the ratio of the flow for the time in question to the unit hydrograph peak flow for the appropriate ratios of the time in question to the time to peak calculated in step 4 and tabulate in column 3 below.

6. Calculate the appropriate unit hydrograph ordinates by multiplying the appropriate ratios from column 3 by 324 cfs/inch and tabulate in column 4 below.

7. Determine the volume under the unit hydrograph.

According to equation 5-38:

$$V = \frac{(12)(0.08)(3,600)(2,159.3)}{(172)(43,560)} = 0.996 \cong \underline{\underline{1.00 \text{ inch}}}$$

(1) Time t (hours)	(2) (t/0.4)	(3) (q/q_p)	(4) Unit Hydrograph q (cfs/inch)
0	0	0	0
0.08	0.20	0.100	32
0.16	0.40	0.310	100
0.24	0.60	0.660	213
0.32	0.80	0.930	301
0.40	1.00	1.000	324
0.48	1.20	0.930	301
0.56	1.40	0.780	253
0.64	1.60	0.560	181
0.72	1.80	0.390	126
0.80	2.00	0.280	91
0.88	2.20	0.207	67
0.96	2.40	0.147	48
1.04	2.60	0.107	35
1.12	2.80	0.077	25
1.20	3.00	0.055	18
1.28	3.20	0.040	13
1.36	3.40	0.029	9
1.44	3.60	0.021	7
1.52	3.80	0.015	5
1.60	4.00	0.011	4
1.68	4.20	0.008	3
1.76	4.40	0.005	2
1.84	4.60	0.002	1
1.92	4.80	0.001	0.3
2.00	5.00	0.000	0

Note: $t_p = 0.4$ hours $\Sigma q_i = 2,159.3$

Example 5-8. Synthetic Runoff Hydrograph Computations Using Unit Hydrograph Theory

Develop a synthetic runoff hydrograph for a 2-hour, 25-year design storm and ultimate land use conditions on subbasin 2 of West End Ditch using unit hydrograph theory. Use the synthetic unit hydrograph developed for subbasin 2 by example 5-7. Compare the peak flow to the SCS graphical approach and rational equation.

1. Step 1 was completed by example 5-7.

2. Develop a design storm hyetograph using the most intense portion of the SCS 24-hour, type II dimensionless cumulative depth curve (Figure 5-2). A time interval of 0.08 hours is required to correspond to the 0.08-hour unit hydrograph developed by example 5-7. Calculations for step 2 are tabulated as follows in columns 1, 2, 3, 4, and 5 of the table on the following page.

3. Develop an excess rainfall hyetograph using the SCS curve number approach presented according to equations 5-1 and 5-2. Calculations for step 3 are tabulated above in columns 6, 7, and 8.

4. Route the excess rainfall hyetograph through subbasin 2 using the unit hydrograph developed by example 5-7. Each increment of excess rainfall from the design storm is multiplied by the unit hydrograph ordinates. This routed incremental hydrograph begins at the time interval during which the excess rainfall occurred. The calculations of step 4 are presented in Table 5-17.

5. Develop the composite synthetic runoff hydrograph by summing the ordinates of each routed incremental hydrograph from step 4. The results of these calculations are presented in Table 5-17. In addition, the process of superposition is illustrated on Figure 5-20.

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

   According to equation 5-38:

   $$V = \frac{(12)(5)(60)(4,936)}{(172)(43,560)} = 2.37 \text{ inches}$$

   Excess rainfall = 2.40 inches; therefore, the runoff volume is O.K.

7. The peak of the synthetic runoff hydrograph developed in Table 5-17 is 536 cfs. For comparison, according to the SCS graphical procedure, Q_T = 681 cfs, and according to the rational equation, Q_T = <u>568 cfs</u> for subbasin 2 of West End Ditch.

<u>Example 5-9</u>. Synthetic Runoff Hydrograph Computations Using the SCS Tabular Method

(1) Time (hours)	(2) SCS Type II 24-Hour Time (hours)	(3) P_y/P_{24} from Figure 5-2	(4) 25-Year, 24-Hour Depth (inches)	(5) Incremental Depth (inches)	(6) Cumulative Depth (inches)	(7) Cumulative Rainfall Excess (inches)	(8) Incremental Excess Rainfall (inches)
0	11.0	0.235	1.77	0.00	0.00	0.00	0.00
0.08	11.08	0.240	1.81	0.04	0.04	0.00	0.00
0.16	11.16	0.247	1.86	0.05	0.09	0.00	0.00
0.24	11.24	0.255	1.92	0.06	0.15	0.00	0.00
0.32	11.32	0.263	1.98	0.06	0.21	0.00	0.00
0.40	11.40	0.275	2.07	0.09	0.30	0.00	0.00
0.48	11.48	0.288	2.17	0.10	0.40	0.00	0.00
0.56	11.56	0.305	2.30	0.13	0.53	0.01	0.01
0.64	11.64	0.340	2.56	0.26	0.79	0.07	0.06
0.72	11.72	0.388	2.92	0.36	1.15	0.22	0.15
0.80	11.80	0.452	3.40	0.48	1.63	0.50	0.28
0.88	11.88	0.540	4.07	0.67	2.30	0.97	0.47
0.96	11.96	0.643	4.84	0.77	3.07	1.58	0.61
1.04	12.04	0.675	5.08	0.24	3.31	1.78	0.20
1.12	12.12	0.690	5.20	0.12	3.43	1.88	0.10
1.20	12.20	0.700	5.27	0.07	3.50	1.94	0.06
1.28	12.28	0.713	5.37	0.10	3.60	2.03	0.09
1.36	12.36	0.722	5.44	0.07	3.67	2.09	0.06
1.44	12.44	0.730	5.50	0.06	3.73	2.14	0.05
1.52	12.52	0.740	5.57	0.07	3.80	2.20	0.06
1.60	12.60	0.748	5.63	0.06	3.86	2.25	0.05
1.60	12.60	0.748	5.63	0.06	3.86	2.25	0.05
1.68	12.68	0.752	5.66	0.03	3.89	2.38	0.03
1.76	12.76	0.757	5.70	0.04	3.93	2.31	0.03
1.84	12.84	0.765	5.76	0.04	3.97	2.35	0.04
1.92	12.92	0.769	5.79	0.03	4.00	2.37	0.02
2.00	13.00	0.772	5.81	0.03	4.03	2.40	0.03

Note: $S = \frac{1000}{84} - 10 = 1.90$ inches

$$Q = \frac{(P - 0.38)^2}{P + 1.52} = \text{Excess Rainfall, in inches}$$

Develop a synthetic runoff hydrograph for a 25-year design storm and ultimate land use conditions on subbasin 2 of West End Ditch using the SCS tabular method. The following data is required:

1. Drainage area = 172 acres = 0.268 square miles.

2. Time of concentration = 0.6 hours (see example 5-5).

3. Ultimate land use CN = 84 (see example 5-3).

4. Twenty-four-hour, 25-year storm depth = 7.53 inches (see Figure 4-2).

5. $S = \frac{1,000}{84} - 10 = 1.90$ inches

   (equation 5-2)

   $Q = \frac{[7.53 - (.2)(1.90)]^2}{7.53 + (.8)(1.90)} = 5.65$ inches

   (equation 5-1)

SCS tabular method computations:

1. Identify the tabular runoff hydrograph flow rates from Table 5-19 in csm/in which are associated with a time of concentration of 0.60 hours. Since a t_c of 0.60 hours is not included in Table 5-19, flow rates must be interpolated to give the data tabulated on the following page.

2. Calculate synthetic runoff hydrograph ordinates using equation 5-39.

   $Q_i = U_i(t_c)(0.268)(5.65)$

   $Q_i = U_i(t_c)(1.514)$

3. The peak flow of this synthetic runoff hydrograph is 657 cfs. For comparison, the following peak flow estimates apply to subbasin 2 of West End Ditch.

   Unit Hydrograph Theory = 536 cfs
   (example 5-8)

   SCS Graphical Method = 681 cfs
   (example 5-6)

   Rational Equation = 568 cfs
   (example 5-6)

Hydrograph Time i (hours)	Ui(tc) Flow Rate, in csm/in From Table 5-19 For $t_c = 0.60$ hours	Q_i in cfs Calculated Using Equation 5-39
11.0	17	26
11.5	33	50
11.7	71	108
11.8	139	210
11.9	246	373
12.0	419	634
12.1	429	650
12.2	434	657
12.3	392	594
12.4	333	504
12.5	275	416
12.6	227	344
12.7	188	285
12.8	156	236
12.9	131	198
13.0	113	171
13.2	88	133
13.5	46	70
14.0	46	70
14.5	37	56
15.0	37	56
16.0	25	38
18.0	18	27
20.0	15	23

SCS Tabular Method = 657 cfs
(example 5-9)

This comparison is consistent with the logic behind each of these estimates. Since the SCS procedures each use a 24-hour duration design storm, the antecedent conditions are such that a higher peak would be expected. Therefore, the SCS procedures should be considered conservative, and are best suited to preliminary design problems.

SECTION 6.0 HYDROLOGIC CHANNEL ROUTING

Having developed a synthetic runoff hydrograph for a particular watershed, it may be necessary to route that hydrograph to another point in the drainage system without adding additional flow. This process is generally known as flood routing. Two categories of flood routing techniques are available to quantify the peak flow attenuation and time lag which is likely to occur as this runoff hydrograph travels through a drainage system. These two categories of flood routing techniques are:

1. Hydrologic routing techniques.

2. Hydraulic routing techniques.

Hydrologic routing considers only the conservation of mass, whereas hydraulic routing considers both the conservation of mass and the equations of motion. In practice, hydrologic routing techniques are usually adequate for stormwater design purposes. The scope of this section will be limited to hydrologic routing techniques, since hydraulic techniques generally require a computer and the emphasis of this manual is on desktop procedures. If the user desires information concerning hydraulic routing techniques, references by Henderson (1966), Viessman et al., (1977), and Chow (1964) should be consulted.

Hydrologic or hydraulic flood routing techniques may be further categorized depending on the type of drainage system being designed. The two categories of drainage systems which require unique flood routing techniques are (1) channel systems and (2) reservoir systems.

This section is limited to channel drainage systems and hydrologic routing techniques. Hydrologic routing techniques for stormwater reservoir systems are presented in Chapter 8.

The material presented in this section includes a general background of hydrologic routing procedures, a discussion of the Muskingum channel routing procedure, and a summary of the SCS tabular method of channel routing.

6.1 General Background

Any flood routing technique requires three types of input data:

1. An inflow hydrograph.

2. A stage-storage relationship.

3. A stage-discharge relationship.

The inflow hydrograph is determined using procedures presented in Section 5.0 of this chapter. The stage-storage and stage-discharge relationships are developed to account for the storage and discharge characterstics of the channel system in question. In practice, it may be convenient to combine these two relationships into a single storage-discharge relationship. The storage-discharge relationships for channels are usually quite different from those which represent reservoirs, because storage in a channel may depend on both the inflow and outflow to the channel. The storage-discharge relationship for channels is developed in this section using the Muskingum method.

Hydrologic flood routing techniques are all based upon the continuity equation. The continuity equation requires that the rate of change to storage in a drainage system must account for all mass flow into and out of that system. Mathematically, the continuity equation is expressed as follows:

$$I - O = \frac{\Delta S}{\Delta t} \qquad (5\text{-}40)$$

where

I = inflow rate to the drainage system, in cfs

O = outflow rate from the drainage system, in cfs

$\frac{\Delta S}{\Delta t}$ = the rate of change to storage in a drainage system

As noted above, the storage within a particular channel segment will generally depend on both the inflow to and the outflow from that channel segment. In practice, it is usually acceptable to approximate this two-component characteristic of channel storage by dividing the total storage volume into two components. The first component depends only on the outflow rate and is commonly known as prism storage. The second component of channel storage is wedge storage, which depends on the difference between inflow and outflow rates. This two-component approximation of the channel storage relationship is illustrated on Figure 5-21.

A general mathematical relationship for expressing this two-component channel storage volume is presented by Chow (1959) as follows:

$$S = \frac{b}{a}\left[XI^{m/n} + (1-X)O^{m/n}\right] \qquad (5\text{-}41)$$

where

S = channel storage volume

I = inflow rate to the channel

O = outflow rate from the channel

X = dimensionless factor which determines the relative weights of I and O on the channel storage volume

a & n = constants which reflect the stage-discharge characteristics of the channel segment

b & m = constants which reflect the stage-storage characteristics of the channel segment

6.2 Muskingum Method

The Muskingum method of hydrologic channel routing was first developed by the U. S. Army Corps of Engineers in connection with flood control schemes for the Muskingum River Basin, Ohio. According to the Muskingum method, the ratio m/n in equation 5-41 equals 1, and the ratio b/a in equation 5-41 equals K, the storage time constant for a particular channel segment. Therefore, according to the Muskingum method, channel storage can be expressed mathematically as follows:

$$S = K[XI + (1 - X)O] \qquad (5\text{-}42)$$

or

$$S = K[O + X(I - O)] \qquad (5\text{-}43)$$

where

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

(S, I, O, and X are defined above.)

The values assigned to the K and X parameters of the Muskingum method are user-specified. If observed inflow and outflow hydrographs for a channel segment are available, the values of K and X can be derived such that the calculated outflow hydrograph agrees as closely as possible with the observed outflow hydrograph. Since the intent of this manual is to present methodologies which can be applied to ungauged watersheds, methodologies for determining K and X values from observed data are not presented. If the user has observed hydrographs and desires to derive K and X values, material presented by Henderson (1966) and Viessman et al., (1977) should be consulted.

The X parameter of the Muskingum method weights the relative importance of inflow and outflow on channel storage. According to equation 5-42 or 5-43, when the value of X is zero, the channel storage volume depends only on the outflow from channel storage. This is identical to the case of routing a flood through a reservoir, and will yield the maximum peak flow attenuation which is realistic. When the value of X is 0.5, the routed outflow hydrograph is almost identical to the inflow hydrograph except that it is lagged by the storage time constant K.

Therefore, the real world extremes for X range between 0.0 and 0.5. The effects of these extremes in X on the outflow hydrographs from a channel are illustrated on Figure 5-22.

Cunge (1969) demonstrated that the values of K and X for a channel segment can be determined mathematically as follows:

$$K = \frac{\Delta L}{v} \qquad (5\text{-}44)$$

$$X = 0.5\ 1 - \frac{Q}{BS_0 v \Delta L} \qquad (5\text{-}45)$$

where

- K = Muskingum channel routing time constant for a particular channel segment
- ΔL = channel routing segment length
- v = velocity or "celerity" of a small kinematic wave in the channel
- Q = flow rate in the channel
- B = top width of the channel water surface
- S_0 = slope of the channel bottom

A finite difference approximation for determining the "celerity" of a small kinematic wave in the channel is given as follows:

$$v = \frac{1}{B} \quad \frac{Q(y + \Delta y) - Q(y)}{\Delta y} \qquad (5\text{-}46)$$

where

$Q(y)$ = a representative flow rate for channel routing at representative depth y

Δy = a small increase in the representative depth of flow in the channel

$Q(y + \Delta y)$ = flow rate at the new depth $y + \Delta y$

Equations 5-44 and 5-45 indicate that the values of K and X are dependent on the channel flow rate, Q. In practice, however, it is generally acceptable to assign constant values to K and X throughout the routing procedure by determining their values at a representative flow rate. The selection of a representative flow rate should be based on engineering judgment. Experience indicates that about 75 percent of the inflow hydrograph peak flow is a good guideline for estimating a representative flow. If the flow depth for that representative flow is greater than the channel capacity, the channel capacity should be considered representative.

A lower limit on the channel length, ΔL, can be estimated from equation 5-45. When X equals 0, then ΔL_{min} can be calculated as follows:

$$\Delta L_{min} = \frac{Q}{BS_0 v} \qquad (5\text{-}47)$$

Other guidelines for estimating parameters in the Muskingum method are given as follows:

$$\frac{K}{3} \leq \Delta t \leq K \qquad (5\text{-}48)$$

and

$$K < T_r \qquad (5\text{-}49)$$

where

Δt = channel routing time interval

T_r = time of rise for the inflow hydrograph

Having established an inflow hydrograph (Section 5.0) and the values of K (equation 5-44), X (equation 5-45), and Δt, the Muskingum channel routing is performed using the following equation:

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1 \qquad (5\text{-}50)$$

where

$$C_0 = \frac{-KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad (5\text{-}51)$$

$$C_1 = \frac{KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad (5\text{-}52)$$

$$C_2 = \frac{K - KX - 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad (5\text{-}53)$$

O_2 = outflow rate at the end of time interval, Δt, in cfs

I_2 = inflow rate at the end of time interval, Δt, in cfs

I_1 = inflow rate at the beginning of time interval, Δt, in cfs

O_1 = outflow rate at the beginning of time interval, Δt, in cfs

Note: $C_0 + C_1 + C_2 = 1.0$ (5-54)

The Muskingum channel routing is performed by calculating the constants C_0, C_1, and C_2, identifying I_1, and I_2 from the inflow hydrograph, assigning the current value of O_2 to O_1, and solving equation 5-50 for a new O_2 value. The initial value of O_1 can be zero unless known to be another value. This sequence of calculations continues until the entire inflow hydrograph is routed through the channel. It is important to note that K and Δt must have the same time units and that the constants C_0, C_1, and C_2 sum to a value of 1.0.

A step-by-step summary of the Muskingum method is given as follows:

1. Select a representative flow rate for evaluating the parameters K and X. Guideline: 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 equation 5-46.

3. Estimate the minimum channel length allowable for the routing using equation 5-47. Check to ensure that ΔL is greater than ΔL min.

4. Estimate a value of K using equation 5-44. Check to ensure that K is less than the time of rise for the inflow hydrograph.

5. Estimate the value of X using equation 5-45.

6. Select a reasonable channel routing time interval. Guidelines of inequality 5-48.

7. Determine C_0, C_1, and C_2 using equations 5-51, 5-52, and 5-53, respectively. Check to ensure that $C_0 + C_1 + C_2 = 1.0$.

8. Determine an initial outflow, O_1 and calculate O_2 using equation 5-50. The routing is then performed by repetitively solving equation 5-50 by assigning the current value of O_2 to be O_1 and determining a new value of O_2. This sequence of calculations continues until the entire inflow hydrograph is routed through the channel.

This 8-step Muskingum method channel routing procedure is demonstrated in example 5-10 at the end of this section.

6.3 SCS Tabular Channel Routing

A procedure similar to the SCS tabular hydrograph method can be utilized to approximate the impact of channel routing on a runoff hydrograph. This procedure is the compilation of data obtained from numerous channel routing calculations obtained by applying the SCS computer program TR-20 to a wide range of watersheds. The tabular channel routing data were developed using TR-20 to compute hydrographs from 1 square mile of drainage area for watershed concentration times and channel travel times ranging from 0.10 hours to 2.0 hours and 0.0 to 4.0 hours, respectively. A 24-hour, type II design storm with a constant curve number of 75 and a rainfall volume large enough to produce 3 inches of excess rainfall were utilized for these computer simulations.

The information required for the SCS tabular channel routing method includes each of the items identified for the SCS tabular hydrograph method in subsection 5.5.3 of this chapter. In addition, the travel time for bank full flow conditions must be estimated for each channel segment in which routing is to take place. The SCS tabular channel routing data are presented in Table 5-20. To utilize this tabular channel routing data the following information is required:

1. Watershed drainage area, in square miles.

2. Watershed time of concentration, in hours.

3. Channel segment travel time for bank full conditions, in hours.

4. Composite runoff curve number for the watershed.

5. Twenty-four-hour rainfall depth, in inches, for the desired design storm return period.

6. Excess rainfall or direct runoff from the watershed, in inches.

Tabular runoff hydrograph data can be located in Table 5-20 for the problem in question given the watershed time of concentration and the travel time for the appropriate channel segment under bank full conditions. The time of concentration is used to identify the correct subtable in Table 5-20, and the channel travel time is used to identify the correct row in that subtable. Having identified the appropriate hydrograph flow rates in Table 5-20, the routed hydrograph flow rates are obtained according to equation 5-55 below.

$$Q_i(T_t) = U_i(t_c, T_t)AQ_{24}(T) \qquad (5\text{-}55)$$

where

$Q_i(T_t)$ = routed hydrograph flow rate, in cfs, for time period i and reach travel time T_t.

$U_i(t_c, T_t)$ = tabular runoff hydrograph ordinate for the desired time of concentration, t_c, reach travel time, T_t, time period i, in csm/in (i.e., cfs per square mile per inch of runoff obtained from Table 5-20)

A = watershed drainage area, in square miles

$Q_{24}(T)$ = watershed runoff volume, in inches, resulting from a 24-hour storm of return period T, (determined using equation 5-1)

The limitations of the SCS tabular method are very clearly stated in SCS TR-55 (1975) as follows:

> "The tabular method should not be used when large changes in the curve number occur among subareas within a watershed and when runoff volumes are less than about 1.5 inches for curve numbers less than 60. For most watershed conditions, however, this procedure is adequate to determine the effects of urbanization on peak rates of discharge for subareas up to approximately 20 square miles in size."

6.4 Example Problems

Example 5-10. Muskingum Method Channel Routing Computations

Route the synthetic runoff hydrograph developed for subbasin 2 in West End Ditch by example 5-8 through a channel with the following characteristics:

$\Delta L = 5{,}000$ feet

$B = 26$ feet

$S_0 = 0.0025$ feet/foot = 0.25%

1. Select a representative flow rate for evaluating K and X.

$$Q(Y) = (0.75)(536) = 402 \text{ cfs}$$

2. Find the velocity of a small kinematic wave using equation 5-46.

From Figure 7-30, Y = 5 feet @ 402 cfs

Assume $\Delta Y = 0.2$ feet, $y + \Delta Y = 5.2$ feet
Therefore, $Q(y + \Delta Y) = 430$ cfs

$$V = \frac{(430 - 402)}{(26)(0.2)} = 5.4 \text{ fps}$$

3. Estimate the minimum channel length for channel routing using equation 5-47.

$$\Delta L_{min} = \frac{402 \text{ cfs}}{(26)(0.0025)(5.4)} = 1{,}145 \text{ feet}$$

$\Delta L = 5{,}000$; therefore, the length is satisfactory for channel routing.

4. Estimate a value of K using equation 5-44.

$$K = \frac{5{,}000}{5.4} = 926 \text{ seconds}$$

$$K = 15.4 \text{ minutes}$$

From Figure 5-20, the time of rise for the inflow hydrograph is 40 minutes; since this is less than 15.4 minutes, K is satisfactory for routing.

5. Estimate the value of X using equation 5-45.

$$X = 0.5 \quad 1 - \frac{402}{(26)(0.0025)(5.4)(5{,}000)}$$

$$X = 0.39$$

6. Select a reasonable channel routing time interval using inequality 5-48.

$$5.1_{min} \leq \Delta t \leq 15.4_{min}$$

Select °t = 10 minutes

7. Determine C_0, C_1, and C_2 using equations 5-51, 5-52, and 5-53, respectively.

$$C_0 = \frac{-(15.4)(0.39) + (0.5)(10)}{(15.4) - (15.4)(0.39) + (0.5)(10)} = -0.07$$

$$C_1 = \frac{(15.4)(0.39) + (0.5)(10)}{(15.4) - (15.4)(0.39) + (0.5)(10)} = 0.76$$

$$C_2 = \frac{(15.4) - (15.4)(0.39) - (0.5)(10)}{(15.4) - (15.4)(0.39) + (0.5)(10)} = \frac{0.31}{1.00}$$

Satisfactory

8. Route the inflow hydrograph using equation 5-50. The calculations are tabulated below using the following sequence of calculations:

a. The previous O_2 becomes O_1; find C_2O_1.

b. Find C_0I_2.

c. Find C, I, and add C_0I_2

d. Add C_2O_1 to step c, thereby determining O_2.

Time (minutes)	Inflow Hydrograph (from Table 5-17) (cfs)	C_0I_2 $C_0 = -0.07$	C_1I_1 $C_1 = 0.76$	C_2O_1 $C_2 = 0.31$	Outflow Hydrograph O_2 (cfs)
0	0	0	0	0	0
10	13	-0.9	9.9	0.	3.2
20	96	-6.7	73.0	7.0	52
30	321	-22.5	244.0	16.0	223
40	525	-36.8	399.0	69.2	433
50	498	-34.9	378.5	134.2	488
60	353	-24.7	268.3	151.3	403
70	237	-16.6	180.1	124.9	294
90	111	-7.8	84.4	63.9	143
100	72	-5.0	54.7	44.4	96
110	40	-2.8	30.4	30.0	59
120	21	-1.5	16.0	18.3	34
130	12	-0.8	9.1	10.4	19
140	3	-0.2	2.3	6.0	8
150	1	-0.1	1.0	2.5	4
160	0	0	0	1.1	1
170	0	0	0	0.3	0.3
180	0	0	0	0.1	0

<u>Example 5-11</u>. SCS Tabular Channel Routing Computations

Route the synthetic runoff hydrograph developed for subbasin 2 in West End Ditch by example 5-8 through the same channel considered for example 5-10.

The SCS tabular channel routing method requires the following data:

1. Drainage area = 0.268 square miles.

2. t_c = 0.6 hours (see example 5-5).

3. Channel segment travel time for bankfull conditions. Assume bankfull conditions occur at 75 percent of the 25-year peak discharge. Therefore, as developed for example 5-10, v = 5.4 fps and

$$T_t = \frac{5,000}{5.4} = 926 \text{ seconds} = 15.4 \text{ minutes}$$

$T_t \cong 0.25$ hours

4. Ultimate land use CN = 84 (see example 5-10).

5. Twenty-four-hour, 25-year storm depth = 7.53 inches (see Figure 4-2).

6. $S = \frac{1,000}{84} - 10 = 1.90$ inches (equation 5-2)

$$Q = \frac{[7.53 - (.2)(1.90)]^2}{7.53 + (.8)(1.90)} = 5.65 \text{ inches}$$

(equation 5-1)

7. Computations are presented in the table below.

Hydrograph Time (hours)	$U_i(t_c, T_t)$ $t_c = 0.5$ $T_t = 0.25$ (csm/in)	$t_c = 0.75$ $T_t = 0.25$ (csm/in)	Interpolated $t_c = 0.60$ $T_t = 0.25$ (csm/in)	$Q_i(T_t)$ Calculated Using Equation 5-55 (cfs)
11.0	15	12	14	21
11.5	26	21	24	36
11.7	37	29	34	51
11.8	52	39	47	71
11.9	94	61	81	123
12.0	172	100	143	217
12.1	277	158	229	347
12.2	372	227	314	475
12.3	425	291	371	562
12.4	424	336	389	589
12.5	383	355	372	363
12.6	326	348	335	507
12.7	270	321	290	439
12.8	221	285	247	374
12.9	182	247	208	315
13.0	150	212	175	265
13.2	107	156	127	192
13.5	73	103	85	129
14.0	49	62	54	82
14.5	39	44	41	62
15.0	33	36	34	52
16.0	26	27	26	39
18.0	19	19	19	29
20.0	15	15	15	23

Note: $Q_i(T_t) = U_i(t_c, T_t)(0.268)(5.65)$

$Q_i(T_t) = U_i(t_c, T_t)(1.514)$

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LIST OF EXAMPLE PROBLEMS--CHAPTER 5

LIST OF EQUATIONS--CHAPTER 5

LIST OF EQUATIONS--CHAPTER 5
(continued)

A = watershed drainage area, in acres

A = watershed drainage area which contributes direct runoff to the conveyance system, in square miles

A_i = drainage area of an area i with uniform characteristics

a = a constant which reflects the stage-discharge characteristics of the channel segment

AMC I = antecedent moisture condition I is the lower limit of antecedent rainfall

AMC II = antecedent moisture condition II, represents average antecedent rainfall conditions

AMC III = antecedent moisture condition III is the upper limit of antecedent rainfall

A_x = cross sectional area of the waterway section where the highwater mark was observed, in ft^2

B = top width of the channel water surface

b = a constant which reflects the stage-storage characteristics of the channel segment

C = rational method runoff coefficient, dimensionless

C_0 = Muskingum channel routing constant given by equation 5-51

C_1 = Muskingum channel routing constant given by equation 5-52

C_2 = Muskingum channel routing constant given by equation 5-53

C_{10} = runoff coefficient for the design storm return period, T

CN = SCS curve number which reflects watershed cover conditions, hydrologic soil group, land use, and antecedent moisture conditions

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

CN_i = curve number for an area i with uniform characteristics

csm/in = cfs per square mile of drainage area per inch of runoff

C_T = rational method runoff coefficient, expressed as a dimensionless ratio, for the design storm return period, T

C_T = runoff coefficient for the design storm return period, T

D = distance of overland flow, in feet

D = duration of runoff producing rainfall, in hours

ΔL = channel routing segment length

$\frac{\Delta s}{\Delta t}$ = the rate of change to storage in a drainage system

Δt = channel routing time interval

Δy = a small increase in the representative depth of flow in a channel

fps = feet per second

I = inflow rate to the drainage system, in cfs

I = average rainfall intensity for the design storm, in inches/hour

I_1 = inflow rate at the beginning of time interval, Δt, in cfs

I_2 = inflow rate at the end of time interval, Δt, in cfs

$I_T(t_c)$ = average rainfall intensity, in inches/hour, during a period of time equal to t_c for the design storm return period, T

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

L = length of travel, in feet

L = hydraulic length of the watershed, in feet

L_1 = length of overland flow path, in feet

L_3 = reach length, in feet

L_i = length of the ith inlet flow segment, in feet

m = a constant which reflects the stage storage characteristics of the channel segment

N = Kerby equation roughness coefficient determined using Table 5-10

n = Manning's roughness coefficient

n = number of inlet flow segments

n = a constant which reflects the stage-discharge characteristics of the channel segment

n = total number of areas with uniform characteristics

NEH = National Engineering Handbook

O = outflow rate, in cfs

O = outflow rate from the drainage system, in cfs

O_1 = outflow rate at the beginning of time interval, Δt, in cfs

O_2 = outflow rate at the end of time interval, Δt, in cfs

P = accumulated rainfall, in inches

P_{24} = 24-hour rainfall depth for the selected design storm return period

P_T = total rainfall depth for a design storm return period, T

P_x = cumulative rainfall depth at time period x

Q = peak runoff rate, in cfs

Q = estimate of observed peak flow, in cfs

Q = accumulated rainfall excess or runoff, in inches

$Q_{24}(T)$ = watershed runoff volume, in inches, resulting from a 24-hour storm of return period T (determined using equation 5-1)

Q_i = runoff hydrograph ordinate, in cfs for time period i

Q_i = synthetic runoff hydrograph ordinate, in cfs, for time period i

$Q_i(T_t)$ = routed hydrograph flow rate, in cfs, for time period i and reach travel time T_t

q_p = unit hydrograph peak discharge in cfs

Q_T = peak runoff rate, in cfs, for the design storm return period, T

Q_T = rainfall excess for a design storm return period, T

$Q(t)$ = hydrograph flow, in cfs, at the time, t

$Q(y)$ = a representative flow rate for channel routing at representative depth y

R = hydraulic radius for a channel cross section, calculated as the cross sectional area divided by wetted perimeter

S = channel storage volume

S = slope in feet per foot

S = overland flow slope, in percent

S = overland flow slope, in feet per foot

S = maximum watershed rainfall retention factor

S = slope of the hydraulic gradient (water surface), in feet per foot

S_0 = slope of the channel bottom

SCS = Soil Conservation Service

Σqi = sum of the runoff hydrograph ordinates, in cfs for each time increment i

T = design storm return period

t_1 = overland flow travel time

t_2 = ditch and street gutter travel time

t_3 = conveyance system travel time

T_B = unit hydrograph regression equations time base, in minutes

T_b = hydrograph time base

t_c = watershed time of concentration, in minutes, defined as the time required for runoff to travel from the hydraulically most distant point of a watershed to the design point

t_I = conveyance system inlet time

T_L = watershed lag time, in hours

T_P = time to peak or time of rise of the runoff hydrograph, in hours

T_R = unit hydrograph regression equations time of rise, in minutes

T_r = hydrograph recession time

T_r = time of rise for the inflow hydrograph

t = time, in appropriate units

$U_i(t_c)$ = SCS TR-55 tabular runoff hydrograph ordinate from Table 5-19 for time of concentration t_c and time period i in csm/inch

$U_i(t_c, T_t)$ = SCS TR-55 tabular runoff hydrograph ordinate for the desired time of concentration, t_c, reach travel time, T_t, at time period i, in csm/inch

USDA = U.S. Department of Agriculture

$U(t_c)$ = unit peak runoff rate in csm/inch for a given watershed t_c (from Figure 5-12)

V = volume under the runoff hydrograph, in inches

v = average flow velocity, in fps

v = estimate of the mean velocity using Manning's equation, in fps (see equation 5-17)

V_1 = overland flow velocity estimated using Figure 5-8, in feet/second

V_3 = average velocity of flow in the reach, in fps (from equation 5-17)

V_i = average flow velocity for the ith segment of inlet flow, in fps

W_{50} = the time, in minutes, between two points on the unit hydrograph at which the discharge is half the peak discharge, Q_{50}

W_{75} = the time, in minutes, between two points on the unit hydrograph at which the discharge is three fourths of the peak discharge, Q_{75}

X = dimensionless factor which determines the relative weights of I and O on the channel storage volume

X_T = design storm frequency factor for the return period, T

Y = average watershed land slope, in percent

Table 5-1
CRITERIA FOR SELECTING A DESIGN STORM
RETURN PERIOD IN MONTGOMERY, ALABAMA

Type of Drainage System	Design Storm Return Period[a]
1. Property systems for areas less than 15,000 square feet	10-year storm for confined conveyance capacity, except for sump conditions where a 25-year storm is required
2. Minor system, onsite or on-to-site drainage for areas greater than or equal to 15,000 square feet, but less than 1 square mile	25-year storm for confined conveyance capacity, with the 100-year storm for designating the path of flow when the confined conveyance capacity is exceeded
3. Major system, through site drainage for areas greater than or equal to 1 square mile	100-year storm for confined conveyance capacity

[a]Since there is always the possibility that a storm or local flooding condition will exceed the capacity of a drainage system, open systems should be designed with freeboard, and closed systems should be designed to convey overland flow or provisions should be made to prevent damage from occurring when the system's capacity is exceeded.

Table 5-2
CALCULATIONS FOR DEVELOPING A 25-YEAR
24-HOUR DESIGN STORM HYETOGRAPH FOR
MONTGOMERY, ALABAMA
(P_{24} = 7.53 in.)

1 Duration (Hours)	2 Depth (Inches)	3 Depth Increment (Inches)	4 Rearranged Depth Increment (Inches)	5 Balanced Storm Cumulative Depth (Inches)	6 SCS Type II Px/P_{24}	7 SCS Type II Storm Cumulative Depth (Inches)
0	0			0	0	0
		2.38	0.05			
0.5	2.38			0.05	0.005	0.04
		0.62	0.05			
1.0	3.00			0.10	0.010	0.08
		0.52	0.05			
1.5	3.52			0.15	0.018	0.14
		0.32	0.05			
2.0	3.84			0.20	0.022	0.17
		0.31	0.05			
2.5	4.15			0.25	0.029	0.22
		0.24	0.05			
3.0	4.39			0.30	0.035	0.26
		0.20	0.06			
3.5	4.59			0.36	0.042	0.32
		0.19	0.06			
4.0	4.78			0.42	0.048	0.36
		0.12	0.06			
4.5	4.90			0.48	0.056	0.42
		0.12	0.06			
5.0	5.02			0.54	0.062	0.47
		0.12	0.06			
5.5	5.14			0.60	0.071	0.53
		0.12	0.06			
6.0	5.26			0.66	0.080	0.60
		0.12	0.06			
6.5	5.38			0.72	0.090	0.68
		0.12	0.06			
7.0	5.50			0.78	0.100	0.75
		0.10	0.06			
7.5	5.60			0.84	0.110	0.83
		0.10	0.10			
8.0	5.70			0.94	0.120	0.90
		0.10	0.10			
8.5	5.80			1.04	0.136	1.02
		0.06	0.12			
9.0	5.86			1.16	0.147	1.11
		0.06	0.12			
9.5	5.92			1.28	0.163	1.23
		0.06	0.12			

Table 5-2--Continued

1 Duration (Hours)	2 Depth (Inches)	3 Depth Increment (Inches)	4 Rearranged Depth Increment (Inches)	5 Balanced Storm Cumulative Depth (Inches)	6 SCS Type II Px/P_{24}	7 SCS Type II Storm Cumulative Depth (Inches)
10.0	5.98			1.40	0.181	1.36
		0.06	0.20			
10.5	6.04			1.60	0.204	1.54
		0.06	0.31			
11.0	6.10			1.91	0.235	1.77
		0.06	0.52			
11.5	6.16			2.43	0.283	2.13
		0.06	2.38			
12.0	6.22			4.81	0.663	4.99
		0.06	0.62			
12.5	6.28			5.43	0.737	5.55
		0.06	0.32			
13.0	6.34			5.75	0.772	5.81
		0.06	0.24			
13.5	6.40			5.99	0.799	6.02
		0.06	0.19			
14.0	6.46			6.18	0.820	6.17
		0.06	0.12			
14.5	6.52			6.30	0.840	6.33
		0.06	0.12			
15.0	6.58			6.42	0.852	6.42
		0.06	0.12			
15.5	6.64			6.54	0.868	6.54
		0.06	0.10			
16.0	6.70			6.64	0.880	6.63
		0.06	0.06			
16.5	6.76			6.70	0.892	6.72
		0.06	0.06			
17.0	6.82			6.76	0.902	6.79
		0.06	0.06			
17.5	6.88			6.82	0.912	6.87
		0.05	0.06			
18.0	6.93			6.88	0.920	6.93
		0.05	0.06			
18.5	6.98			6.94	0.931	7.01
		0.05	0.06			
19.0	7.03			7.00	0.940	7.08
		0.05	0.06			
19.5	7.08			7.06	0.948	7.14
		0.05	0.06			
20.0	7.13			7.12	0.952	7.17
		0.05	0.06			
20.5	7.18			7.18	0.960	7.23
		0.05	0.05			

Table 5-2--Continued

1 Duration (Hours)	2 Depth (Inches)	3 Depth Increment (Inches)	4 Rearranged Depth Increment (Inches)	5 Balanced Storm Cumulative Depth (Inches)	6 SCS Type II P_x/P_{24}	7 SCS Type II Storm Cumulative Depth (Inches)
21.0	7.23			7.23	0.968	7.29
		0.05	0.05			
21.5	7.28			7.28	0.974	7.33
		0.05	0.05			
22.0	7.33			7.33	0.978	7.36
		0.05	0.05			
22.5	7.38			7.38	0.985	7.42
		0.05	0.05			
23.0	7.43			7.43	0.990	7.45
		0.05	0.05			
23.5	7.48			7.48	0.995	7.49
		0.05	0.05			
24.0	7.53			7.53	1.000	7.53

Table 5-3
DEFINITIONS OF FOUR SCS
HYDROLOGIC SOIL GROUPS

Hydrologic Soil Group	Definition
A	Low Runoff Potential Soils having high infiltration rates even when thoroughly wetted consisting chiefly of deep, well to excessivley drained sands or gravels. These soils have a high rate of water transmission.
B	Moderately Low Runoff Potential Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
C	Moderately High Runoff Potential Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, soils with moderate fine to fine texture, or soils with moderate water tables. Those soils have a slow rate of water transmission.
D	High Runoff Potential Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

Source: USDA, SCS NEH-4, (1972)

Table 5-4
RUNOFF CURVE NUMBERS FOR SELECTED AGRICULTURAL, SUBURBAN, AND URBAN LAND USE
(ANTECEDENT MOISTURE CONDITION II, AND $I_a=0.2S$)

Land Use Description		Hydrologic Soil Group A	B	C	D
Cultivated Land[a]:					
without conservation treatment		72	81	88	91
with conservation treatment		62	71	78	81
Pasture or range land:					
poor condition		68	79	86	89
good condition		39	61	74	80
Meadow: good condition		30	58	71	78
Wood or Forest land:					
thin stand, poor cover, no mulch		45	66	77	83
good cover[b]		25	55	70	77
Open Spaces, lawns, parks, golf courses, cemeteries, etc.					
good condition: grass cover on 75% or more of the area		39	61	74	80
fair condition: grass cover on 50% to 75% of the area		49	69	79	84
Commercial and business areas (85% impervious)		89	92	94	95
Industrial districts (72% impervious)		81	88	91	93
Residential[c]:					
Average lot size	Average % Impervious[d]				
1/8 acre or less	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
Paved parking lots, roofs, driveways, etc.[e]		98	98	98	98
Streets and roads:					
paved with curbs and storm sewers[e]		98	98	98	98
gravel		76	85	89	91
dirt		72	82	87	89

Source: USDA, SCS, TR-55 (1975)

[a] For a more detailed description of agricultural land use curve numbers refer to Table 5-5.
[b] Good cover is protected from grazing and litter and brush cover soil.
[c] Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.
[d] The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.
[e] In some warmer climates of the country a curve number of 95 may be used.

Table 5-5
RUNOFF CURVE NUMBERS FOR
AGRICULTURAL LAND USES
(ANTECEDENT MOISTURE CONDITION II, AND $I_a=0.2S$)

Land Use	Cover Treatment or Practice	Hydrologic Condition	Hydrologic Soil Group A	B	C	D
Fallow	Straight row	----	77	86	91	94
Row crops	Straight row	Poor	72	81	88	91
	Straight row	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	and terraced	Poor	66	74	80	82
	and terraced	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
	Straight row	Good	63	75	83	87
	Contoured	Poor	63	74	82	85
	Contoured	Good	61	73	81	84
	and terraced	Poor	61	72	79	82
	and terraced	Good	59	70	78	81
Close seeded legumes[a] or rotation meadow	Straight row	Poor	66	77	85	89
	Straight row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
	and terraced	Poor	63	73	80	83
	and terraced	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		----	59	74	82	86
Roads (dirt)[b]		----	72	82	87	89
(hard surface)[b]		----	74	84	90	92

Source: USDA, SCS NEH-4, (1972)

[a] Close-drilled or broadcast.
[b] Including right-of-way.

Table 5-6
CLASSIFICATION OF VEGETATIVE COVERS
BY THEIR HYDROLOGIC PROPERTIES

Vegetative Cover	Hydrologic Condition
Crop rotation	Poor: Contain a high proportion of row crops, small grains, and fallow. Good: Contain a high proportion of alfalfa and grasses.
Native pasture or range	Poor: Heavily grazed or having plant cover on less than 50% of the area. Fair: Moderately grazed; 50-75% plant cover. Good: Lightly grazed; more than 75% plant cover. Permanent Meadow: 100% grass cover.
Woodlands	Poor: Heavily grazed or regularly burned so that litter, small trees, and brush are destroyed. Fair: Grazed but not burned; there may be some litter. Good: Protected from grazing so that litter and shrubs cover the soil.

Source: USDA, SCS NEH-4, (1972)

Table 5-7
RECOMMENDED APPLICABILITY OF FOUR PEAK FLOW ESTIMATING PROCEDURES FOR MONTGOMERY, ALABAMA

Method[a]	Application
1. Rational Method	Storm sewer design and overland flow estimates for tributary basins less than 50 acres.
2. Unit Hydrograph Theory or SCS Graphical Procedures	Watershed areas greater than 50 acres, major through-site open channels large closed conduits, and stormwater detention/retention basins. Unit hydrograph theory is particularly applicable for areas which will undergo significant urbanization.
3. Flood Frequency Regression Equation	An independent check on peak flows determined by other methods. Regression equations are generally best suited to medium or large watersheds not subject to urbanization.
4. Historical Highway Marks and Manning's Equation	An independent check on pre-development peak flows determined by other methods.

[a]For storm sewer design, the peak flow should be routed using the inlet hydrograph method in Subsection 5.2.3 of Chapter 7.

Table 5-8
RUNOFF COEFFICIENTS[a] FOR A DESIGN STORM RETURN
PERIOD OF 10 YEARS OR LESS

Slope	Land-Use	Sandy Soils Min.	Sandy Soils Max.	Clay Soils Min.	Clay Soils Max.
Flat (0-2%)	Woodlands	0.10	0.15	0.15	0.20
	Pasture, grass & farmland[b]	0.15	0.20	0.20	0.25
	Rooftops and pavement	0.95		0.95	
	Single family residential:				
	½-acre lots & larger	0.30	0.35	0.35	0.45
	Smaller lots	0.35	0.45	0.40	0.50
	Multi-family residential:				
	Duplexes	0.35	0.45	0.40	0.50
	Apartments, townhouses, and condominums	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 & farmland[a]	0.20	0.25	0.25	0.30
	Rooftops and pavement	0.95		0.95	
	Single family residential:				
	½-acre lots & larger	0.35	0.50	0.40	0.55
	Smaller lots	0.40	0.55	0.45	0.60
	Multi-family residential:				
	Duplexes	0.40	0.55	0.45	0.60
	Apartments, townhouses, and condominums	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 & farmland[a]	0.25	0.35	0.30	0.40
	Rooftops and pavement	0.95		0.95	
	Single family residential:				
	½-acre lots & larger	0.40	0.55	0.50	0.65
	Smaller lots	0.45	0.60	0.55	0.70
	Multi-family residential:				
	Duplexes	0.45	0.60	0.55	0.70
	Apartments, townhouses, and condominiums	0.60	0.75	0.65	0.85
	Commercial and Industrial	0.60	0.95	0.65	0.95

Source: DeKalb County Drainage Procedures Manual, December 1976.

[a] Weighted 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.

Table 5-9
DESIGN STORM FREQUENCY FACTORS
FOR 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

Source: Wright-McLaughlin (1969).

Table 5-10
N VALUES FOR THE KERBY
OVERLAND FLOW EQUATION

N	Type of Surface
0.02	smooth impervious surfaces
0.10	smooth bare packed soil, free of stones
0.20	poor grass, cultivated row crops or moderately rough bare surfaces
0.40	pasture or average grass cover
0.60	deciduous timberland
0.80	conifer timberland, deciduous timberland with deep forest litter or dense grass cover

Table 5-11
FLOOD FREQUENCY REGRESSION EQUATIONS
FOR SMALL STREAMS[a] IN ALABAMA

Equation[b]	Standard Error of Estimate, Percent[c]
$Q_2 = 92.8\ A^{.646}S^{.237}$	43.9
$Q_5 = 166.2\ A^{.683}S^{.193}$	30.6
$Q_{10} = 229.8\ A^{.700}S^{.166}$	30.3
$Q_{25} = 332.0\ A^{.716}S^{.132}$	31.5
$Q_{50} = 421.7\ A^{.727}S^{.110}$	32.6
$Q_{100} = 532.1\ A^{.731}S^{.087}$	33.9

where

Q_T = the estimated discharge, in cubic feet per second (ft^3/s), for the indicated recurrence interval, T.

A = the contributing drainage area, in square miles (mi^2),

S = channel slope in feet per mile, determined from the difference in elevations at points 10 and 85 percent of the distance along the main channel from the discharge site to the drainage basin divide. The difference in elevation is divided by the main channel length in miles between the two points.

Source: Olin and Bingham (1977)

[a] Drainage areas used to derive the equations ranged from 0.54 to 15.9 mi^2 and channel slopes ranged from 12.7 to 286.2 ft/mi.

[b] All regression coefficients are statistically significant at the 5 percent level.

[c] Standard error computations include three gauging stations deleted from the analysis. Therefore, reported errors are larger than the average error in the regression analysis.

Table 5-12
FLOOD FREQUENCY REGRESSION EQUATIONS FOR URBAN WATERSHEDS IN THE UNITED STATES

Equation[a,b]	Correlation Coefficient (logs)	Average Absolute Error, as a Percentage
$Q_{2.33} = 169\ A^{0.77} I^{0.29} S^{0.42} R_{2.33}^{1.80} \phi^{-1.17}$	0.97	30
$Q_5 = 172\ A^{0.80} I^{0.27} S^{0.43} R_5^{1.73} \phi^{-1.21}$	0.97	31
$Q_{10} = 178\ A^{0.82} I^{0.26} S^{0.44} R_{10}^{1.71} \phi^{-1.32}$	0.96	31
$Q_{20} = 243\ A^{0.84} I^{0.24} S^{0.48} R_{20}^{1.62} \phi^{-1.38}$	0.96	32
$Q_{50} = 297\ A^{0.85} I^{0.22} S^{0.50} R_{50}^{1.57} \phi^{-1.61}$	0.96	34
$Q_5 = 1.13\ Q_{2.33}^{1.03}$	0.99	8
$Q_{10} = 1.24\ Q_{2.33}^{1.05}$	0.99	16
$Q_{20} = 1.34\ Q_{2.33}^{1.06}$	0.99	22
$Q_{50} = 1.47\ Q_{2.33}^{1.08}$	0.97	28

where

A = drainage area, in square miles,

I = impervious cover expressed as a percentage,

Q_T = peak flow for specified return perorid, T, in cfs, (e.g., Q_5 is the peak flow for a 5-year storm)

R_T = rainfall, in inches, of the 6-hour duration storm for the corresponding return period, T.

S = slope of the main channel in feet per foot,

ϕ = urbanization factor. (See Table 5-13)

Source: Espey and Winslow (1974)

[a] See subsection 4.4.2 for a discussion of these equations.
[b] The 50-year flood determined by regression equations should be reviewed in light of historical rainfall or streamflow data.

Table 5-13
WATERSHED CONVEYANCE FACTORS FOR
URBAN DRAINAGE SYSTEMS

Φ_1	Classification
0.6	Extensive channel improvement and storm sewer system, closed conduit channel system.
0.8	Some channel improvement and storm sewers; mainly cleaning and enlargement of existing channel.
1.0	Natural channel conditions.
Φ_2	
0.0	No channel vegetation.
0.1	Light channel vegetation.
0.2	Moderate channel vegetation.
0.3	Heavy channel vegetation.

$$\Phi = \Phi_1 + \Phi_2 \qquad (5\text{-}21)$$

Φ = Urbanization factor

Φ_1 = Drainage system improvement factor

Φ_2 = Channel vegetation factor

Source: Espey and Winslow (1974)

Table 5-14
SCS DIMENSIONLESS UNIT HYDROGRAPH
RATIOS AND MASS DATA

Time Ratios (t/T_p)	Discharge Ratios (q/q_p)	Mass Curve Ratios (Q_a/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	.021	.993
3.8	.015	.995
4.0	.011	.997
4.5	.005	.999
5.0	.000	1.000

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

Table 5-15
TEN-MINUTE UNIT HYDROGRAPH REGRESSION EQUATIONS

Equations	Total Explained Variation
$T_R = 3.1\ L^{0.23} S^{-0.25} I^{-0.18} \phi^{1.57}$	0.802
$Q = 31.62 \times 10^3\ A^{0.96} T_R^{-1.07}$	0.936
$T_B = 125.89 \times 10^3\ A\ Q^{-0.95}$	0.844
$W_{50} = 16.22 \times 10^3\ A^{0.93}\ Q^{-0.92}$	0.943
$W_{75} = 3.24 \times 10^3\ A^{0.79}\ Q^{-0.78}$	0.834

where

Watershed Characteristics

L = total distance (in feet) along the main channel from the point being considered to the upstream watershed boundary.

S = main channel slope (in feet per foot) as defined by H/(0.8L), where L is the main channel length as described above and H is the difference in elevation between two points, A and B. A is a point on the channel bottom at a distance of 0.2L downstream from the upstream watershed boundary. B is a point on the channel bottom at the downstream point being considered.

I = impervious area within the watershed (in percent).

Φ = dimensionless watershed conveyance factor as determined by Table 5-15 or Figure 5-20.

A = watershed drainage area (in square miles).

Unit Hydrograph Shape Parameters

T_R = time of rise of the unit hydrograph (in minutes).

Q = peak flow of the unit hydrograph (in cfs).

T_B = time base of the unit hydrograph (in minutes).

W_{50} = width of the hydrograph at 50 percent of the Q (in minutes).

W_{75} = width of the unit hydrograph at 75 percent of Q (in minutes).

Source: Espey, et al. (1977).

Table 5-16
EQUATIONS TO PREDICT 30-MINUTE UNIT HYDROGRAPH
FOR SMALL URBAN WATERSHEDS

$I \geq 20\%$	$I < 20\%$	General
$T_R = 143\ L^{.43}\ S^{-.06}\ I^{-.32}$	$T_R = 222\ L^{.38}\ S^{-.45}$	$T_R = 91.4\ L^{.47}\ S^{-.22}\ I^{-.01}$
$Q_P = 40490\ A^{.96}\ T_R^{-1.1}$	$Q_P = 23580\ A^{.99}\ T_R^{-1.0}$	$Q_P = 27770\ A^{.96}\ T_R^{-1.01}$
$T_B = 87380\ A^{1.14}\ Q_P^{-.90}$	$T_B = 64670\ A^{1.03}\ Q_P^{-.87}$	$T_B = 61840\ A^{1.03}\ Q_P^{-.85}$
$W_{75} = 7940\ A^{.89}\ Q_P^{-.86}$	$W_{75} = 9850\ A^{.90}\ Q_P^{-.90}$	$W_{75} = 9018\ A^{.89}\ Q_P^{-.89}$
$W_{50} = 24560\ A^{.96}\ Q_P^{-.95}$	$W_{50} = 21560\ A^{.94}\ Q_P^{-.94}$	$W_{50} = 21840\ A^{.94}\ Q_P^{-.94}$

where

T_R = unit hydrograph time of rise, from the beginning of surface runoff to the peak runoff, in minutes.

Q_P = peak runoff rate in cfs.

T_B = unit hydrograph base time from the beginning to the end of surface runoff, in minutes.

W_{50} = time between the points on the hydrograph when the discharge represented by Q_{50} is half the peak discharge in minutes.

W_{75} = time between the points on the hydrograph when the discharge represented by Q_{75} is three-fourths of the peak discharge.

A = drainage basin area in square miles.

L = effective length of the main channel in miles.

S = effective slope of the main channel in feet per mile.

I = percent of impervious cover contained within the drainage basin.

Source: Hamm, et al. (1973).

Table 5-17
APPLICATION OF UNIT HYDROGRAPH THEORY
TO DEVELOP A SYNTHETIC RUNOFF HYDROGRAPH FOR SUBBASIN 2 IN WEST END DITCH

UH	0	32	100	213	301	324	301	253	181	126	91	67	48	35	25	18
Runoff Time (min)	0	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75
Rain Excess (inches)																
0.01	0	0	1	2	3	3	3	3	2	1	1	1	1	0		
0.06		0	2	6	13	18	19	18	15	11	8	5	4	3	2	2
0.15			0	5	15	32	45	49	45	38	27	19	14	10	7	5
0.28				0	9	28	60	84	91	84	71	51	35	25	19	13
0.47					0	15	47	100	141	152	141	119	85	59	43	32
0.61						0	20	61	130	184	198	184	154	110	77	56
0.20							0	6	20	43	60	65	60	51	36	25
0.10								0	3	10	21	30	32	30	25	18
0.06									0	2	6	13	18	19	18	15
0.09										0	3	9	19	27	29	27
0.06											0	2	6	13	18	19
0.05												0	2	5	11	15
0.03													0	1	3	6
0.03														0	1	3
0.04															0	1
0.02																0
0.03																
Runoff Total (cfs)	0	0	3	13	40	96	194	321	447	525	536	498	430	353	289	237

UH	13	9	7	5	4	3	2	1	0.3						
Runoff Time (min)	80	85	90	95	100	105	110	115	120	125	130	135	140	145	150
Rain Excess (inches)															
0.01															
0.06	1	1	1	0											
0.15	4	3	2	1	1	1	1	1	0						
0.28	10	7	5	4	3	2	1	1	1	1					
0.47	23	16	12	8	6	4	3	2	2	1	1	1	0		
0.61	41	29	21	15	11	8	6	4	3	2	2	1	1	0	
0.20	18	13	10	7	5	4	3	2	1	1	1	1	0		
0.10	13	9	7	5	4	3	2	1	1	1	1	0			
0.06	11	8	5	4	3	2	2	1	1	1	0				
0.09	23	16	11	8	6	4	3	2	2	1	1	1	1	0	
0.06	18	15	11	8	5	4	3	2	2	1	1	1	0		
0.05	16	15	13	9	6	5	3	2	2	1	2	1	1	0	
0.03	9	10	9	8	5	4	3	2	1	1	1	1	0		
0.03	6	9	10	9	8	5	4	3	2	1	1	1	1	0	
0.04	4	9	12	13	12	10	7	5	4	3	2	1	1	1	1
0.02	1	2	4	6	7	6	5	4	3	2	1	1	1	1	0
0.03	0	1	3	6	9	10	9	8	5	4	3	2	1	1	1
Runoff Total (cfs)	198	162	136	111	91	72	55	40	30	21	17	12	7	3	2

Check Volume Using Equation 5-38: $V = \frac{(12)(5)(60)(4{,}936)}{(172)(43{,}560)} = 2.37$ inches

Peak Flow = 536 cfs

NOTE: See example problem 5-8 for details concerning data in this table.

MG14186.AO

(1) Time		(2) Unit Hyd.	(3) Flood Hyd.
	.09		
	.19		
	.24		
	.31		
	.42		
	.36		
	.25		
	.11		
	.05		
	.01		
	.0		
	.0		
	.06		
	.12		
	.18		
	.26		
	.33		
0	.27	0	0
.3	.12	140	0
.6	.0 ←	420	17
.9		960	
1.2		1330	
1.5		1450	
1.8		1370	
2.1		1140	
2.4		860	
2.7		610	
3.0		440	
3.3		320	
3.6		230	
3.9		170	
4.2		120	
4.5		85	
4.8		70	
5.1		55	
5.4		40	
5.7		30	
6.0		20	
6.3		15	
6.6		10	
6.9		7	
7.2		4	
7.5		2	
7.8		0	
8.1			
8.4			
8.7			
9.0			
9.3			
9.6			
9.9			
10.2			
10.5			
10.8			
11.1			
11.4			
11.7			
12.0			
12.3			
12.6			
12.9			
13.2			
13.5			
13.7			

Total 9898

(1) Time		(2) Unit Hyd.	(3) Flood Hyd.
	.09		
	.19		
	.24		
	.31		
	.42		
	.36		
	.25		
	.11		
	.05		
	.01		
	.0		
	.0		
	.06		
	.12		
	.18		
	.26		
0	.33	0	0
.3	.27	140	0
.6	.12	420	17
.9	.0 ←	960	88
1.2		1330	
1.5		1450	
1.8		1370	
2.1		1140	
2.4		860	
2.7		610	
3.0		440	
3.3		320	
3.6		230	
3.9		170	
4.2		120	
4.5		85	
4.8		70	
5.1		55	
5.4		40	
5.7		30	
6.0		20	
6.3		15	
6.6		10	
6.9		7	
7.2		4	
7.5		2	
7.8		0	
8.1			
8.4			
8.7			
9.0			
9.3			
9.6			
9.9			
10.2			
10.5			
10.8			
11.1			
11.4			
11.7			
12.0			
12.3			
12.6			
12.9			
13.2			
13.5			
13.7			

(1) Time		(2) Unit Hyd.	(3) Flood Hyd.
0		0	0
.3		140	0
.6	.09	420	17
.9	.19	960	88
1.2	.24	1330	275
1.5	.31	1450	594
1.8	.42	1370	984
2.1	.36	1140	1337
2.4	.25	860	1563
2.7	.11	610	1620
3.0	.05	440	1515
3.3	.01	320	1300
3.6	.0	230	1050
3.9	.0	170	838
4.2	.06	120	726
4.5	.12	85	765
4.8	.18	70	988
5.1	.26	55	1359
5.4	.33	40	1737
5.7	.27	30	2143
6.0	.12	20	2342
6.3	.0 ←	15	2350
6.6		10	
6.9		7	
7.2		4	
7.5		2	
7.8		0	
8.1			
8.4			
8.7			
9.0			
9.3			
9.6			
9.9			
10.2			
10.5			
10.8			
11.1			
11.4			
11.7			
12.0			
12.3			
12.6			
12.9			
13.2			
13.5			
13.7			

(1) Time		(2) Unit Hyd.	(3) Flood Hyd.
0		0	0
.3		140	0
.6		420	17
.9		960	88
1.2		1330	275
1.5		1450	594
1.8		1370	984
2.1		1140	1337
2.4		860	1563
2.7		610	1620
3.0		440	1516
3.3		320	1300
3.6		230	1050
3.9		170	838
4.2		120	726
4.5		85	765
4.8		70	988
5.1		55	1359
5.4		40	1737
5.7		30	2143
6.0		20	2342
6.3		15	2350
6.6		10	2170
6.9		7	1854
7.2		4	1488
7.5		2	1138
7.8	.09	0	840
8.1	.19		605
8.4	.24		433
8.7	.31		318
9.0	.42		233
9.3	.36		172
9.6	.25		128
9.9	.11		96
10.2	.05		72
10.5	.01		53
10.8	.0		38
11.1	.0		27
11.4	.06		18
11.7	.12		12
12.0	.18		7
12.3	.26		4
12.6	.33		2
12.9	.27		1
13.2	.12		0
13.5	.0 ←		
13.7			

Total 33359

TABLE 5-18. Short-cut application of unit hydrograph theory to develop a synthetic runoff hydrograph.

Table 5-19
SCS TABULAR HYDROGRAPH METHOD FOR A TYPE II
STORM AND A ZERO TRAVEL TIME

Hydrograph Time Period, i (hours)	Tabular Hydrograph Flow Rates for Selected Watershed Times of Concentration, In Hours (Flow Rates In csm/in[a])									
	0.1	0.2	0.3	0.4	0.5	0.75	1.0	1.25	1.5	2.0
11.0	24	23	21	20	18	15	13	11	10	7
11.5	51	47	43	39	36	29	24	21	18	14
11.7	299	208	141	103	80	57	45	37	31	22
11.8	991	509	324	224	166	98	66	51	42	30
11.9	746	796	586	419	301	163	107	79	57	38
12.0	477	641	658	558	533	248	155	107	81	49
12.1	233	424	535	575	496	329	211	147	105	64
12.2	152	245	372	451	474	375	258	187	133	80
12.3	132	170	251	331	395	388	301	319	164	95
12.4	121	138	184	247	309	369	313	249	192	114
12.5	111	121	148	190	242	325	316	264	209	133
12.6	84	104	124	155	194	276	301	271	227	152
12.7	74	85	102	127	158	232	277	267	235	165
12.8	70	75	86	105	130	195	247	256	236	175
12.9	68	71	77	90	109	165	217	241	236	175
13.0	65	68	71	80	94	142	188	219	225	192
13.2	52	56	61	66	75	107	146	177	201	190
13.5	39	40	41	42	43	51	64	81	99	129
14.0	39	40	41	42	43	51	64	81	99	129
14.5	33	34	34	35	36	39	46	56	68	93
15.0	33	34	34	35	36	39	46	56	68	93
16.0	24	24	24	24	25	26	27	29	32	41
18.0	18	18	18	18	18	19	19	20	20	23
20.0	14	14	14	14	15	15	15	16	16	17

Source: USDA, SCS TR-55 (1975).

[a] csm/in = ft^3/sec - sq. mi. - inch of runoff, to obtain cfs, multiply csm/in by the watershed drainage area in sq. mi., and the hydrograph runoff volume in inches.

Table 5-20

SCS TR-55 TABULAR DISCHARGES FOR TYPE-II STORM DISTRIBUTION (CSM/IN)

TIME OF CONCENTRATION = 0.1 hours

Hydrograph Time in Hours

T_t	11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	24	51	299	991	746	477	233	152	132	121	111	85	74	70	68	65	52	48	39	33	29	24	18	14
0.25	20	38	66	140	327	626	686	546	364	236	169	137	117	97	83	75	66	52	41	35	30	24	18	14
0.50	15	27	36	43	67	133	288	482	580	543	429	310	222	168	134	110	81	63	47	38	32	26	19	16
0.75	12	20	25	29	34	42	65	125	245	392	496	515	452	360	273	206	127	80	53	42	35	27	19	15
1.00	9	15	19	21	24	28	32	41	63	115	209	328	427	470	451	389	245	121	64	47	38	29	20	16
1.50	6	10	12	13	14	16	17	19	22	25	29	38	56	92	154	236	410	360	133	66	47	33	21	16
2.00	3	6	7	8	9	10	11	12	13	14	16	18	20	23	27	34	74	244	371	142	68	38	23	17
2.50	2	4	4	5	5	6	7	7	8	9	10	11	12	13	15	16	21	41	243	343	150	48	26	19
3.00	1	2	2	3	3	4	4	4	5	5	6	7	7	8	9	10	12	17	50	239	321	74	29	20
3.50	0	1	1	1	1	2	2	2	3	3	4	4	4	5	6	6	7	10	17	59	304	159	33	21
4.00	0	0	0	0	0	1	1	1	1	2	2	2	2	3	3	4	5	6	10	18	67	290	39	23

TIME OF CONCENTRATION=0.2hours

Hydrograph Time in Hours

T_t	11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	23	47	208	509	796	641	424	245	170	138	121	104	85	75	71	68	56	49	40	34	29	24	18	14
0.25	18	34	49	91	196	419	603	627	486	341	235	173	138	114	96	83	70	55	43	36	31	25	18	15
0.50	14	24	32	37	50	87	181	341	490	545	497	397	296	219	167	133	92	67	49	39	33	26	19	15
0.75	11	18	23	26	30	36	49	84	161	284	409	491	481	422	340	263	157	89	56	43	36	27	19	15
1.00	9	14	18	20	22	25	29	35	48	79	143	240	347	426	452	427	299	147	69	49	39	29	20	16
1.50	5	9	11	12	13	14	16	18	20	23	26	32	43	67	110	176	330	399	159	72	50	33	22	17
2.00	3	6	7	7	8	9	10	11	12	13	15	16	18	21	24	29	56	192	363	168	75	40	24	18
2.50	1	3	4	5	5	6	6	7	7	8	9	10	11	12	13	15	19	33	200	337	174	51	26	19
3.00	0	2	2	2	3	3	4	4	5	5	6	6	7	8	8	9	11	15	40	203	316	82	29	20
3.50	0	0	1	1	1	2	2	2	2	3	3	4	4	5	5	6	7	9	16	46	300	180	34	22
4.00	0	0	0	0	0	1	1	1	1	1	2	2	2	3	3	3	4	6	9	16	53	286	41	24

Table 5-20—Continued
SCS TR-55 TABULAR DISCHARGES FOR TYPE-II STORM DISTRIBUTION (CSM/IN)

TIME OF CONCENTRATION = 0.3 hours
Hydrograph Time in Hours

T_t	11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	21	43	141	324	586	658	535	372	251	184	148	124	102	86	77	71	61	51	41	34	30	24	18	14
0.25	17	31	43	67	134	279	461	559	530	428	318	234	179	143	116	97	76	59	45	37	32	25	18	15
0.50	13	22	29	34	42	65	124	238	378	479	499	447	363	281	216	168	110	74	51	41	34	26	19	15
0.75	10	17	21	24	27	32	41	63	114	203	316	413	457	443	389	319	198	105	60	45	37	28	20	15
1.00	8	13	16	18	20	23	26	31	40	60	103	176	269	358	415	426	344	182	77	51	41	30	20	16
1.50	5	8	10	11	12	13	15	16	18	21	24	28	36	52	82	132	272	382	192	81	52	34	22	17
2.00	3	5	6	7	8	8	9	10	11	12	14	15	17	19	21	25	44	151	351	198	85	41	24	18
2.50	1	3	4	4	5	5	6	6	7	8	8	9	10	11	12	14	17	28	162	328	200	54	27	19
3.00	0	1	2	2	3	3	3	4	4	5	5	6	6	7	8	9	10	14	33	169	309	94	30	20
3.50	0	0	1	1	1	1	2	2	2	3	3	3	4	4	5	5	6	9	14	38	172	294	35	22
4.00	0	0	0	0	0	0	1	1	1	1	1	2	2	2	3	3	4	5	9	15	43	281	42	24

TIME OF CONCENTRATION = 0.4 hours
Hydrograph Time in Hours

T_t	11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	20	39	103	224	419	558	575	451	331	247	190	155	127	105	90	80	66	53	42	35	30	24	18	14
0.25	15	28	38	54	98	196	343	468	508	464	380	295	228	180	145	119	87	64	47	38	32	26	19	15
0.50	12	20	26	30	37	53	92	172	286	395	462	453	402	332	266	211	137	84	54	42	35	27	19	15
0.75	10	16	19	22	25	29	36	51	85	150	242	338	407	429	406	356	241	128	65	47	38	29	20	16
1.00	8	12	15	17	19	21	24	28	34	49	78	132	208	292	362	403	368	220	88	55	42	30	21	16
1.50	5	8	9	10	11	12	14	15	17	19	22	25	31	43	65	102	220	365	224	93	56	35	22	17
2.00	3	5	6	6	7	8	9	9	10	11	13	14	16	17	20	23	37	119	338	225	99	43	24	18
2.50	1	3	3	4	4	5	5	6	6	7	8	9	10	11	12	13	16	25	132	317	225	58	27	19
3.00	0	1	2	2	2	3	3	3	4	4	5	5	6	7	7	8	10	13	28	140	300	107	31	21
3.50	0	0	1	1	1	1	1	2	2	2	3	3	3	4	4	5	6	8	13	32	146	286	36	22
4.00	0	0	0	0	0	0	0	1	1	1	1	1	2	2	2	3	3	5	8	14	36	275	44	24

Table 5-20—Continued

SCS TR-55 TABULAR DISCHARGES FOR TYPE-II STORM DISTRIUBTION (CSM/IN)

TIME OF CONCENTRATION = 0.5 hours
Hydrograph Time in Hours

T_t	11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	18	36	80	166	301	433	496	474	395	309	242	194	158	130	109	94	75	57	43	36	31	25	18	15
0.25	15	26	37	52	94	172	277	372	425	424	383	326	270	221	182	150	107	73	49	39	33	26	19	15
0.50	12	20	25	30	38	58	101	169	252	327	374	385	366	329	285	241	169	103	59	44	36	27	19	15
0.75	9	15	19	22	25	30	41	63	103	162	229	292	335	354	348	325	255	157	77	50	39	29	20	16
1.00	7	12	15	17	19	21	25	31	43	66	103	153	210	264	304	327	317	231	109	61	44	31	21	16
1.50	5	8	9	10	11	12	14	15	17	20	24	31	43	63	92	129	214	295	224	115	65	36	23	17
2.00	3	5	6	6	7	8	9	10	11	12	13	14	16	19	23	30	58	143	271	216	120	46	25	18
2.50	1	3	3	4	4	5	5	6	7	7	8	9	10	11	12	14	18	39	150	253	209	71	28	19
3.00	0	1	2	2	2	3	3	4	4	4	5	5	6	7	7	8	10	15	48	154	239	126	32	21
3.50	0	0	1	1	1	1	2	2	2	2	3	3	4	4	5	5	6	8	16	56	155	227	38	23
4.00	0	0	0	0	0	1	1	1	1	1	1	2	2	2	3	3	4	5	9	19	63	217	52	25

TIME OF CONCENTRATION = 0.75 hours
Hydrograph Time in Hours

T_t	11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	15	29	57	98	163	248	329	375	388	369	325	276	232	195	165	142	107	76	51	39	33	26	19	15
0.25	12	21	29	39	61	100	158	227	291	336	355	348	321	285	247	212	156	103	62	44	36	27	19	15
0.50	10	16	21	24	29	41	63	100	150	208	263	305	327	329	314	288	226	147	79	52	40	29	20	16
0.75	8	13	16	18	20	24	30	43	65	98	142	192	239	278	303	311	286	208	107	63	45	31	21	16
1.00	6	10	13	14	15	17	20	24	31	44	65	95	134	177	220	256	294	264	149	81	53	33	21	16
1.50	4	6	8	9	10	11	12	13	14	16	19	23	31	42	60	83	147	269	248	152	85	40	23	17
2.00	2	4	5	5	6	7	7	8	9	10	11	12	14	16	18	23	39	97	251	235	153	56	26	19
2.50	1	2	3	3	4	4	4	5	5	6	7	7	8	9	10	11	15	28	107	218	236	91	29	20
3.00	0	1	1	2	2	2	2	3	3	4	4	5	5	6	6	7	8	12	33	113	225	153	34	22
3.50	0	0	1	1	1	1	1	1	2	2	2	3	3	3	4	4	5	7	13	39	117	215	44	24
4.00	0	0	0	0	0	0	0	1	1	1	1	1	1	2	2	2	3	4	7	15	45	207	63	26

Table 5-20—Continued

SCS TR-55 TABULAR DISCHARGES FOR TYPE-II STORM DISTRIBUTION (CSM/IN)

TIME OF CONCENTRATION = 1.0 hours

Hydrograph Time in Hours

T_t	11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	13	24	45	66	107	155	211	258	301	313	316	301	277	247	217	188	146	102	64	46	36	27	19	15
0.25	10	18	24	32	45	68	102	146	193	238	272	293	299	293	275	252	200	139	81	54	41	29	20	16
0.50	8	14	17	20	24	32	46	68	99	136	178	219	251	274	284	283	254	187	105	65	47	31	21	16
0.75	7	11	13	15	17	20	25	33	46	67	94	128	165	202	233	256	273	236	140	82	55	33	21	16
1.00	5	9	11	12	13	15	17	20	25	33	46	65	90	121	154	187	240	262	183	107	66	37	22	17
1.50	3	5	7	7	8	9	10	11	12	14	16	19	24	31	43	58	103	185	244	181	110	48	24	18
2.00	2	3	4	4	5	6	6	7	8	8	9	10	11	13	15	18	29	69	182	230	178	70	27	19
2.50	1	2	2	3	3	3	4	4	5	5	6	6	7	8	9	10	12	21	77	178	219	114	31	21
3.00	0	1	1	1	1	2	2	2	3	3	3	4	4	5	5	6	7	10	25	83	210	172	39	22
3.50	0	0	0	0	1	1	1	1	1	2	2	2	2	3	3	3	4	6	11	29	88	202	52	25
4.00	0	0	0	0	0	0	0	0	1	1	1	1	1	1	2	2	2	4	6	12	33	195	77	28

TIME OF CONCENTRATION = 1.25 hours

Hydrograph Time in Hours

T_t	11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	11	21	37	51	79	107	147	187	219	249	264	271	267	256	241	219	177	128	81	56	42	29	20	16
0.25	9	15	21	27	36	53	74	103	137	172	205	231	249	259	259	253	223	167	102	67	48	31	21	16
0.50	7	12	15	17	21	27	37	51	72	98	128	160	190	216	235	247	251	209	130	82	45	34	21	16
0.75	6	9	12	13	15	17	21	27	36	50	69	93	120	149	177	202	235	242	165	103	67	38	22	17
1.00	4	7	9	10	11	13	14	17	21	27	36	49	66	88	113	139	190	236	200	130	83	43	23	17
1.50	3	5	6	6	7	8	8	9	10	12	14	16	20	25	33	44	76	142	223	195	131	58	26	18
2.00	1	3	3	4	4	5	5	6	6	7	8	9	10	11	13	15	24	52	143	212	189	86	29	20
2.50	1	1	2	2	2	3	3	3	4	4	5	5	6	7	7	8	10	17	58	143	201	132	35	21
3.00	0	1	1	1	1	1	2	2	2	2	3	3	3	4	4	5	6	9	20	64	143	196	45	23
3.50	0	0	0	0	0	1	1	1	1	1	1	2	2	2	2	3	4	5	9	23	68	190	62	26
4.00	0	0	0	0	0	0	0	0	0	0	1	1	1	1	1	1	2	3	5	10	26	184	91	30

Table 5-20—Continued

SCS TR-55 TABULAR DISCHARGES FOR TYPE-II STORM DISTRIBUTION (CSM/IN)

TIME OF CONCENTRATION = 1.5 hours

Hydrograph Time in Hours

T_t	11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	10	18	31	42	57	81	105	133	164	192	209	227	235	236	236	225	201	153	99	68	50	32	20	16
0.25	8	13	17	22	30	41	57	76	99	125	153	178	199	215	225	230	224	188	122	82	58	36	21	16
0.50	6	10	13	15	18	22	30	40	54	72	94	118	143	167	188	204	224	214	152	99	68	39	22	17
0.75	5	8	10	11	13	15	18	22	29	39	52	69	89	111	134	157	194	219	182	122	82	44	23	17
1.00	4	6	8	9	10	11	12	14	17	22	29	38	50	66	84	105	148	198	214	150	100	50	24	18
1.50	2	4	5	5	6	7	7	8	9	10	12	14	17	21	26	34	58	109	191	204	149	70	28	19
2.00	1	2	3	3	4	4	4	5	5	6	7	8	8	10	11	13	19	40	112	184	197	102	33	20
2.50	0	1	1	2	2	2	3	3	3	4	4	5	5	6	6	7	9	14	45	114	190	147	40	22
3.00	0	0	1	1	1	1	1	1	2	2	2	3	3	3	4	4	5	7	16	49	115	184	53	25
3.50	0	0	0	0	0	0	1	1	1	1	1	1	2	2	2	2	3	4	8	18	53	178	74	28
4.00	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1	1	2	2	4	8	21	174	105	34

TIME OF CONCENTRATION = 2.0 hours

Hydrograph Time in Hours

T_t	11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	7	14	22	30	38	49	64	80	95	114	133	152	165	175	184	192	190	176	129	93	68	41	23	17
0.25	6	10	13	17	22	28	37	47	61	75	91	108	126	143	157	168	185	189	153	109	79	46	24	17
0.50	5	8	10	11	13	17	21	27	35	45	57	71	86	103	119	135	162	186	172	129	92	52	26	18
0.75	4	6	8	8	10	11	13	16	21	26	34	43	55	67	82	97	129	166	183	149	109	59	27	18
1.00	3	5	6	7	7	8	9	11	13	16	20	26	33	42	52	64	92	136	180	167	127	68	29	19
1.50	1	3	3	4	4	5	5	6	7	8	9	10	12	15	18	23	37	68	135	175	163	93	34	21
2.00	1	1	2	2	3	3	3	4	4	5	5	6	6	7	8	10	14	26	71	133	170	127	42	23
2.50	0	1	1	1	1	1	2	2	2	3	3	3	4	4	5	5	7	11	29	74	132	166	53	26
3.00	0	0	0	0	1	1	1	1	1	1	2	2	2	2	3	3	4	5	12	32	76	162	71	30
3.50	0	0	0	0	0	0	0	0	1	1	1	1	1	1	1	2	2	3	6	13	35	158	95	35
4.00	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1	2	3	6	14	80	155	43

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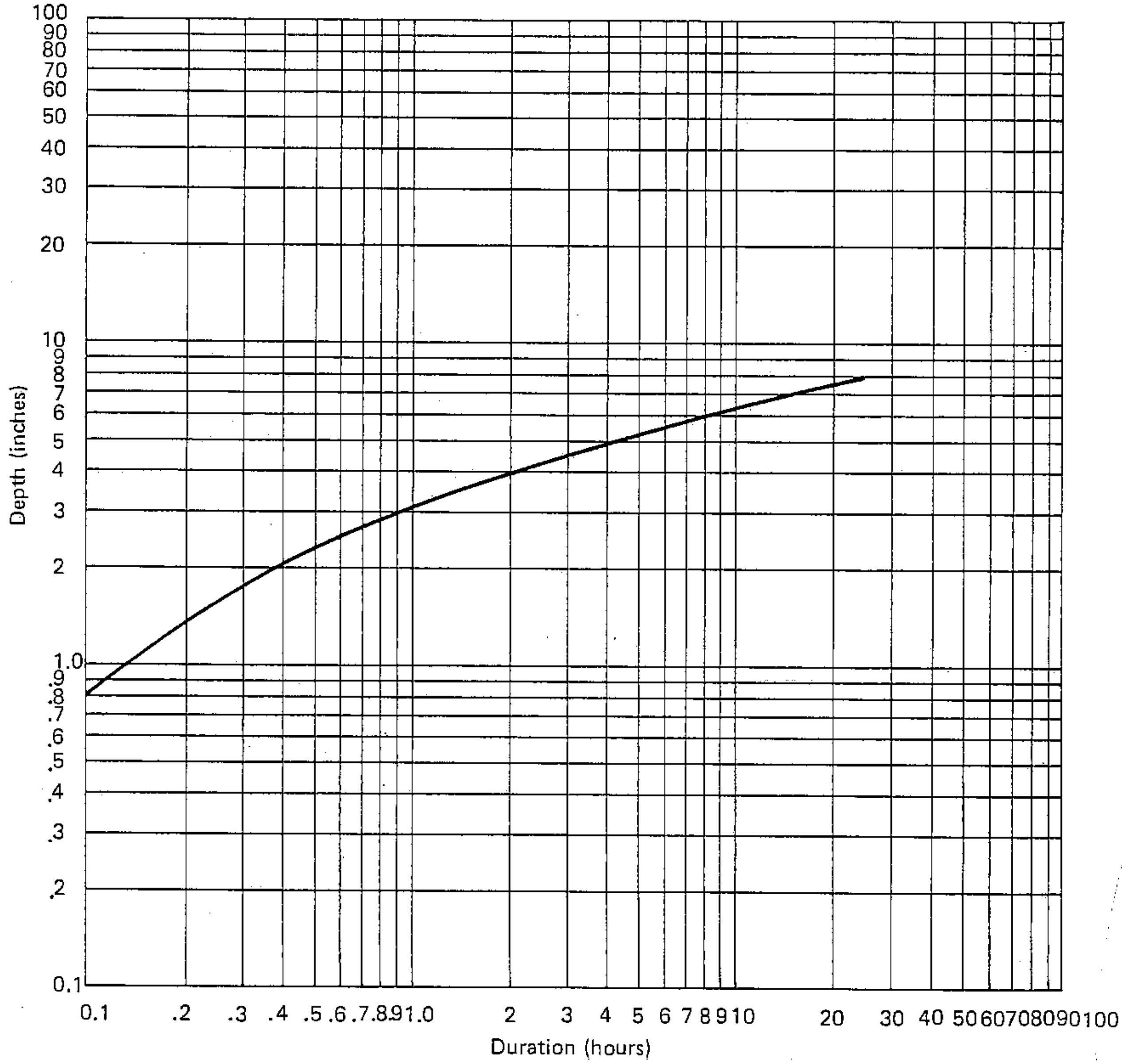
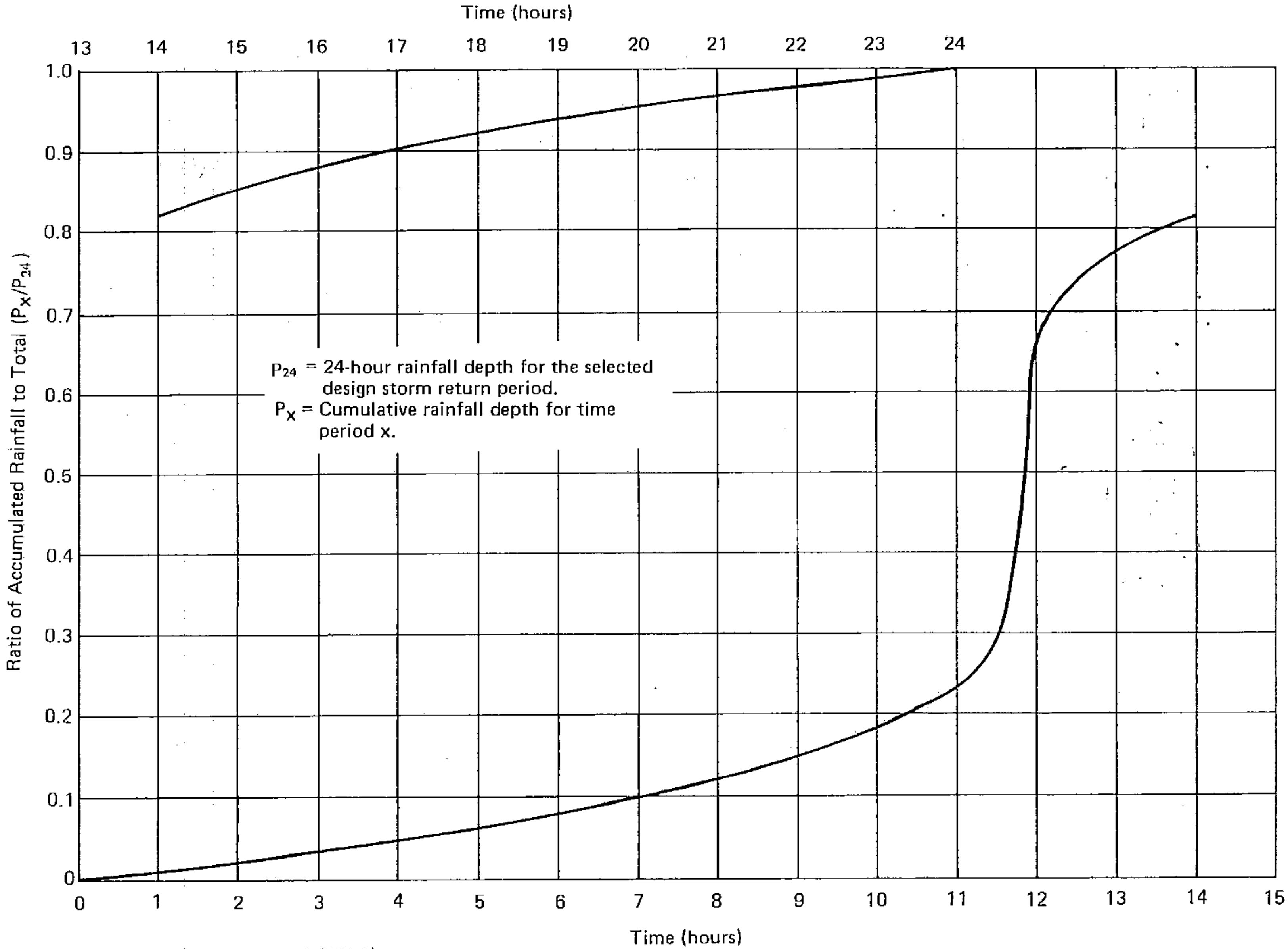


FIGURE 5-1. Montgomery 25-year depth-duration curve.



Source: USDA, SCS, TP-149 (1973)

FIGURE 5-2. SCS 24-hour type II dimensionless cumulative depth curve.

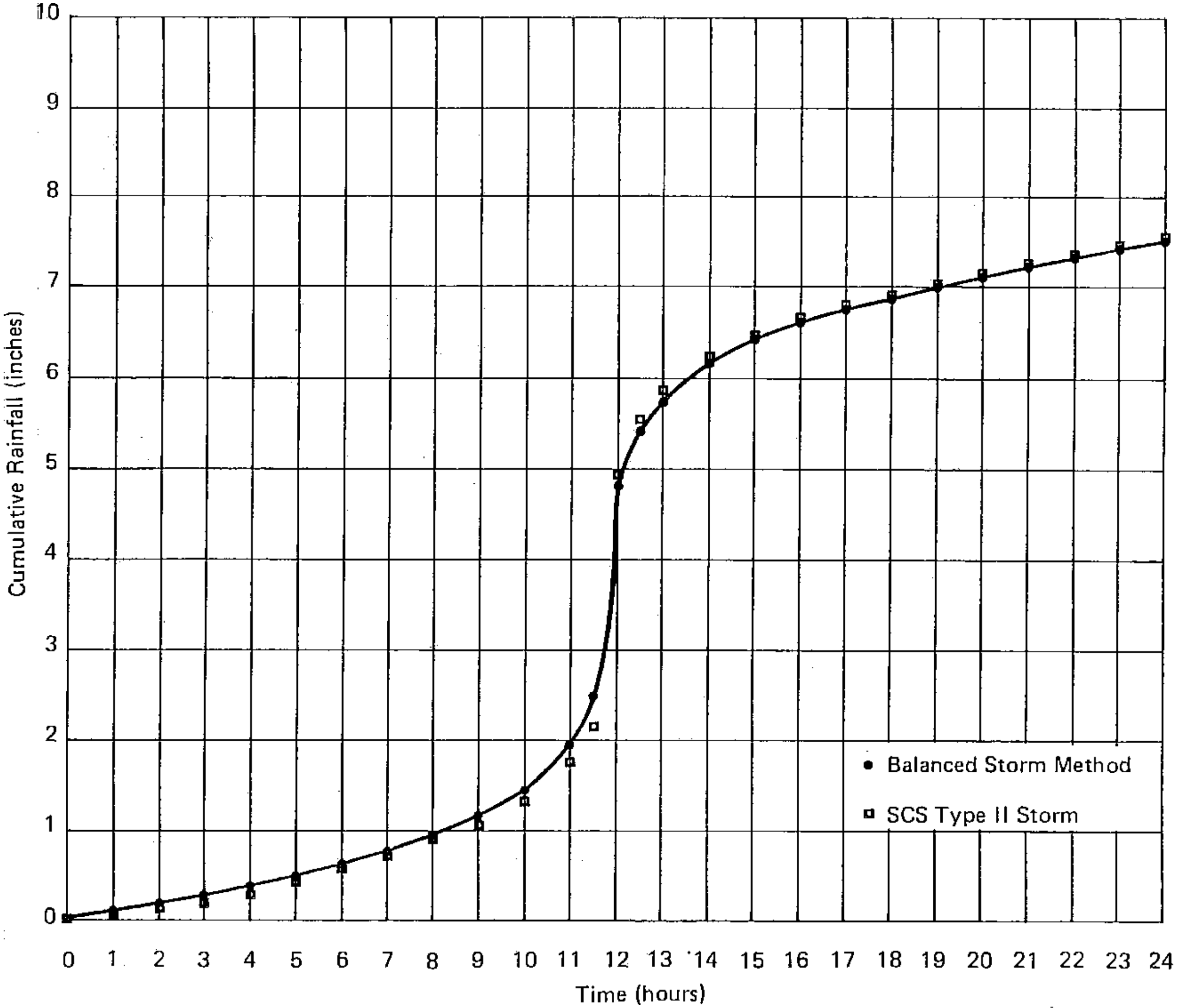
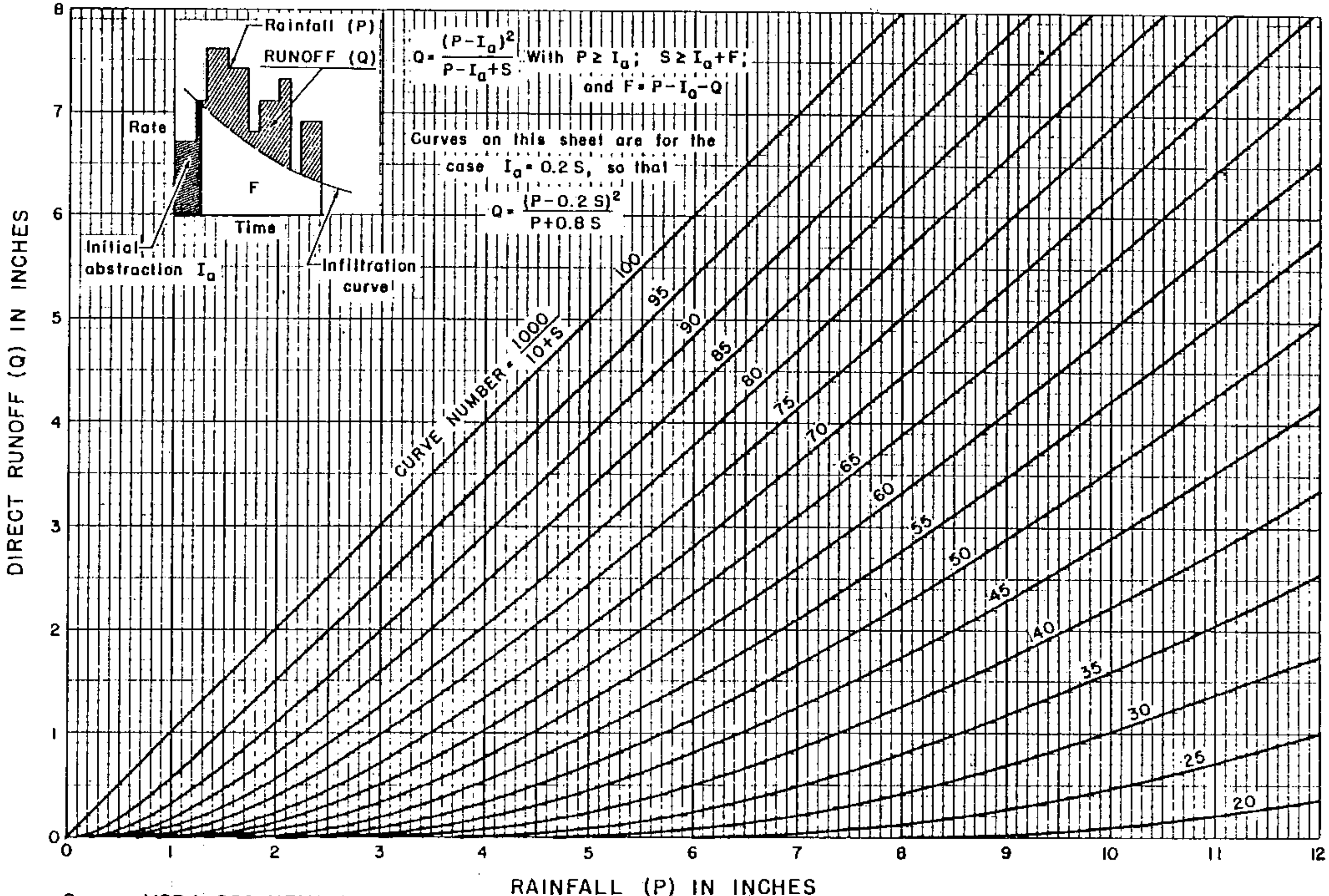


FIGURE 5-3. Comparison of methods for developing a 25-year, 24-hour design storm for Montgomery.



Source: USDA, SCS, NEH-4 (1972)

FIGURE 5-4. Graphical solution of the SCS excess rainfall model for P = 0 to 12 inches and Q = 0 to 8 inches.

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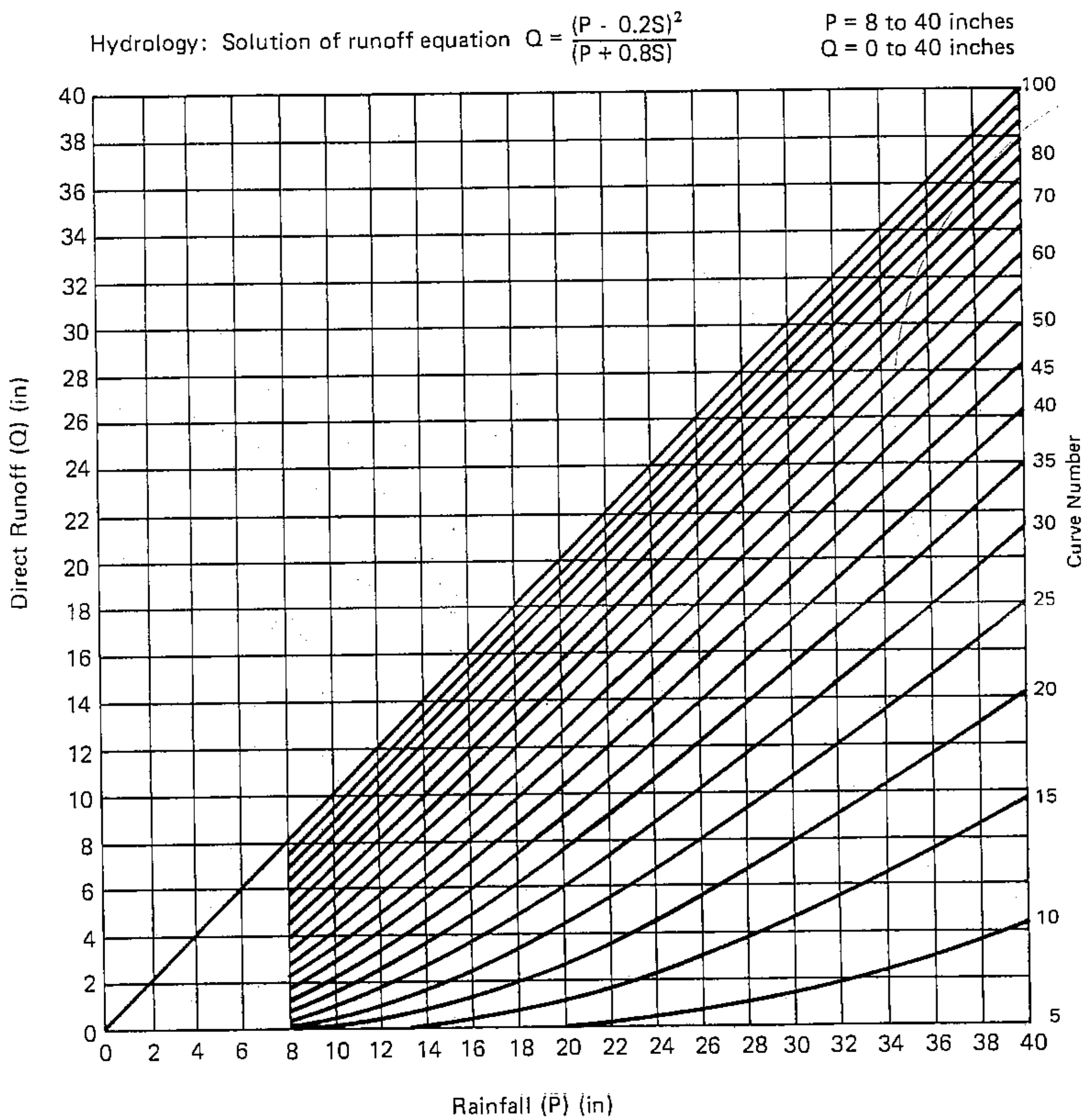

Source: USDA, SCS, NEH-4 (1972)

FIGURE 5-5. Graphical solution of the SCS excess rainfall model for P = 8 inches to 40 inches, and Q = 0 to 40 inches.

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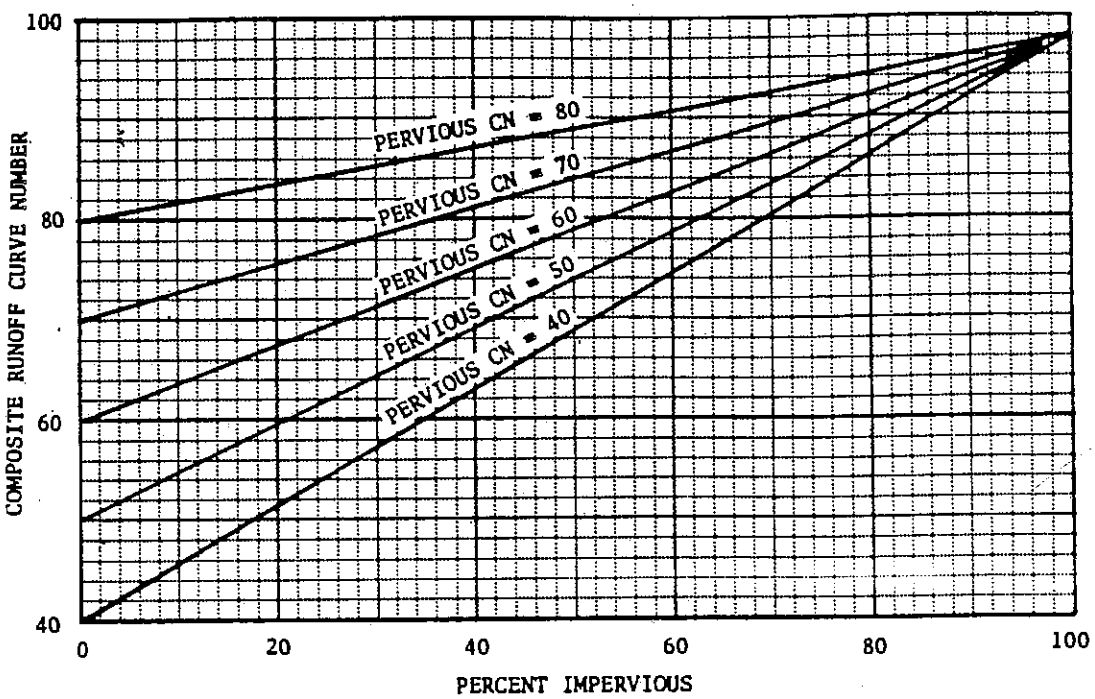

Source: USDA, SCS, TR-55 (1975)

FIGURE 5-6. Percentage of impervious area versus composite curve numbers for given pervious area curve numbers.

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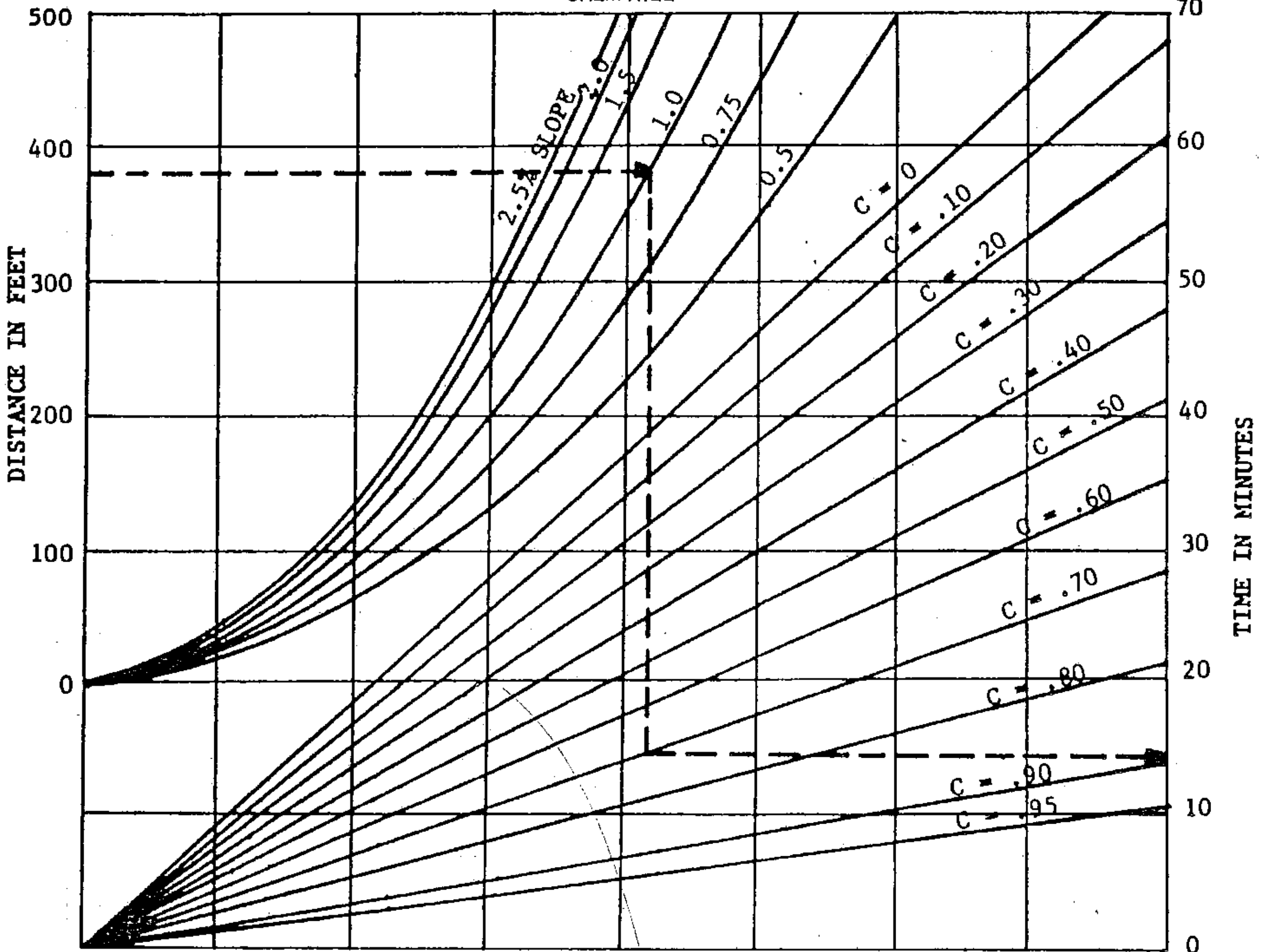

Source: Federal DOT, Advisory Circular 150-5320-5B, 1970.

FIGURE 5-7. Graphical solution of the Federal DOT overland flow travel time equation.

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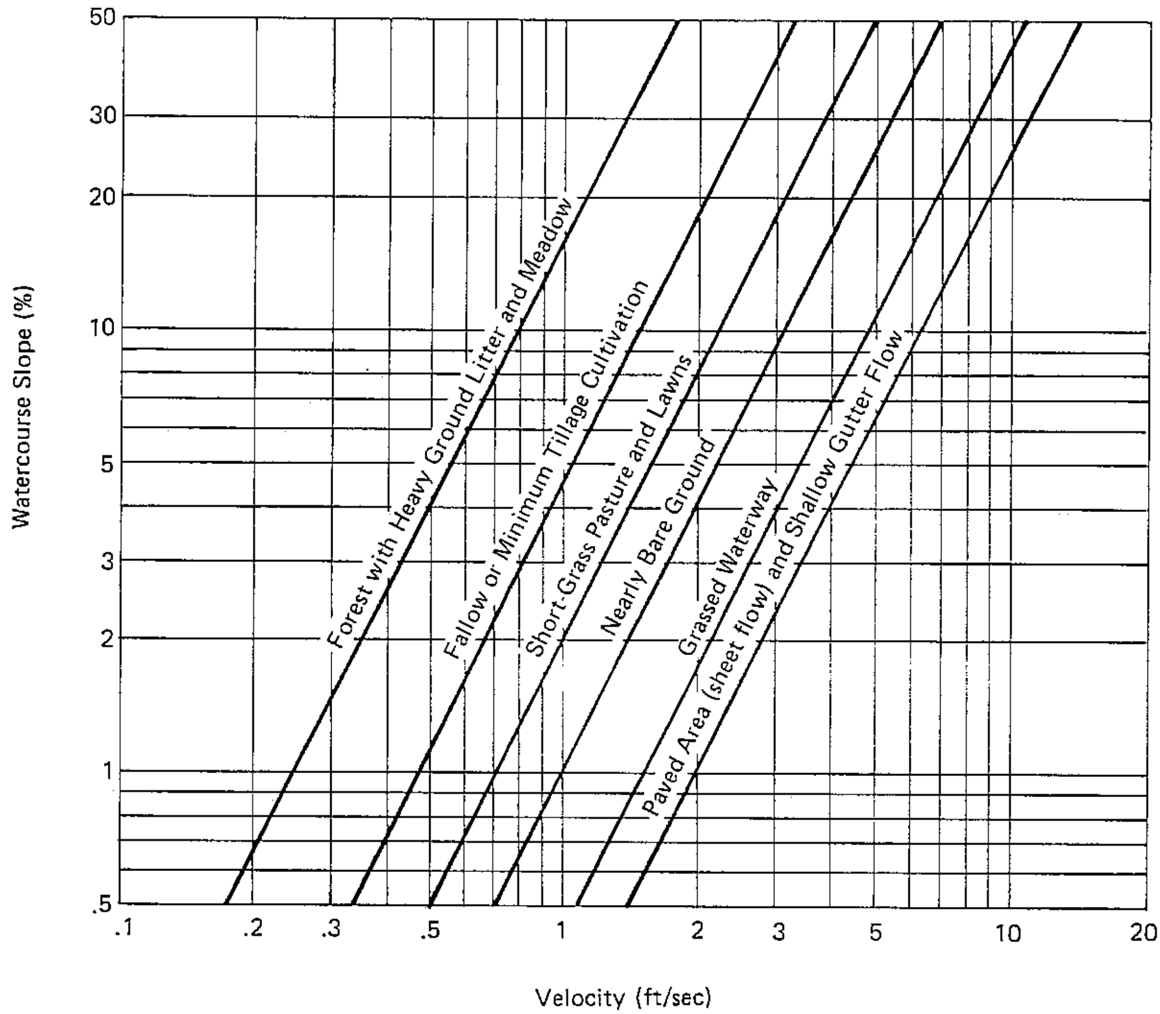

Source: USDA, SCS, TR-55 (1975)

FIGURE 5-8. Average velocities for the SCS overland or upland flow methods.

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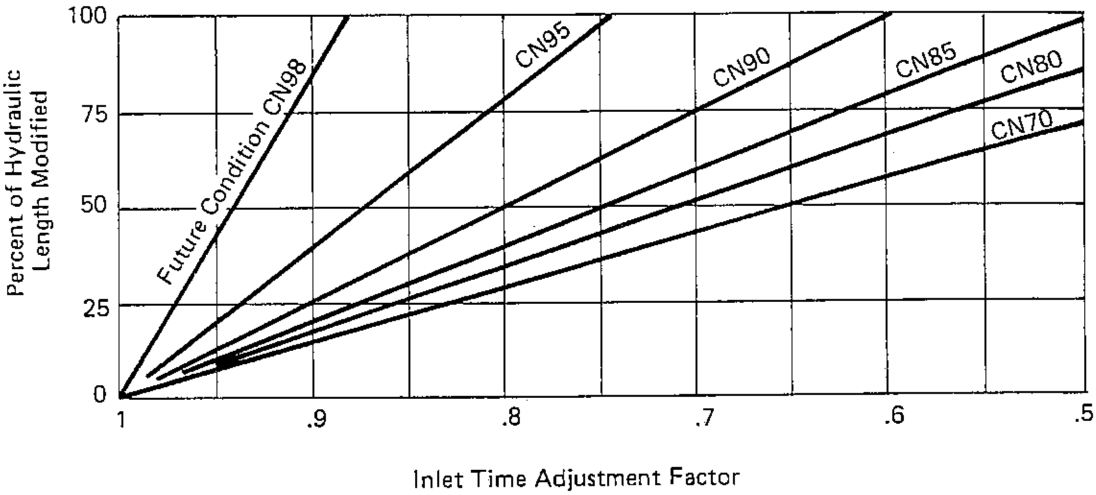
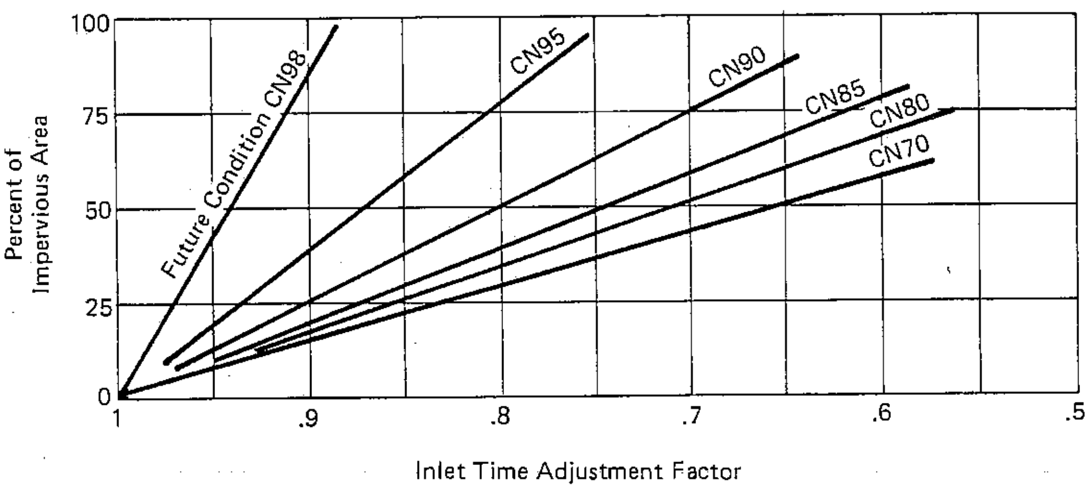


Source: USDA, SCS, TR-55 (1975)

FIGURE 5-9. SCS inlet time adjustment factors for equation 5-15 when the main channel has been hydraulically improved.



Source: USDA, SCS, TR-55 (1975)

FIGURE 5-10. SCS inlet time adjustment factors for equation 5-15 when an impervious area occurs in the watershed.

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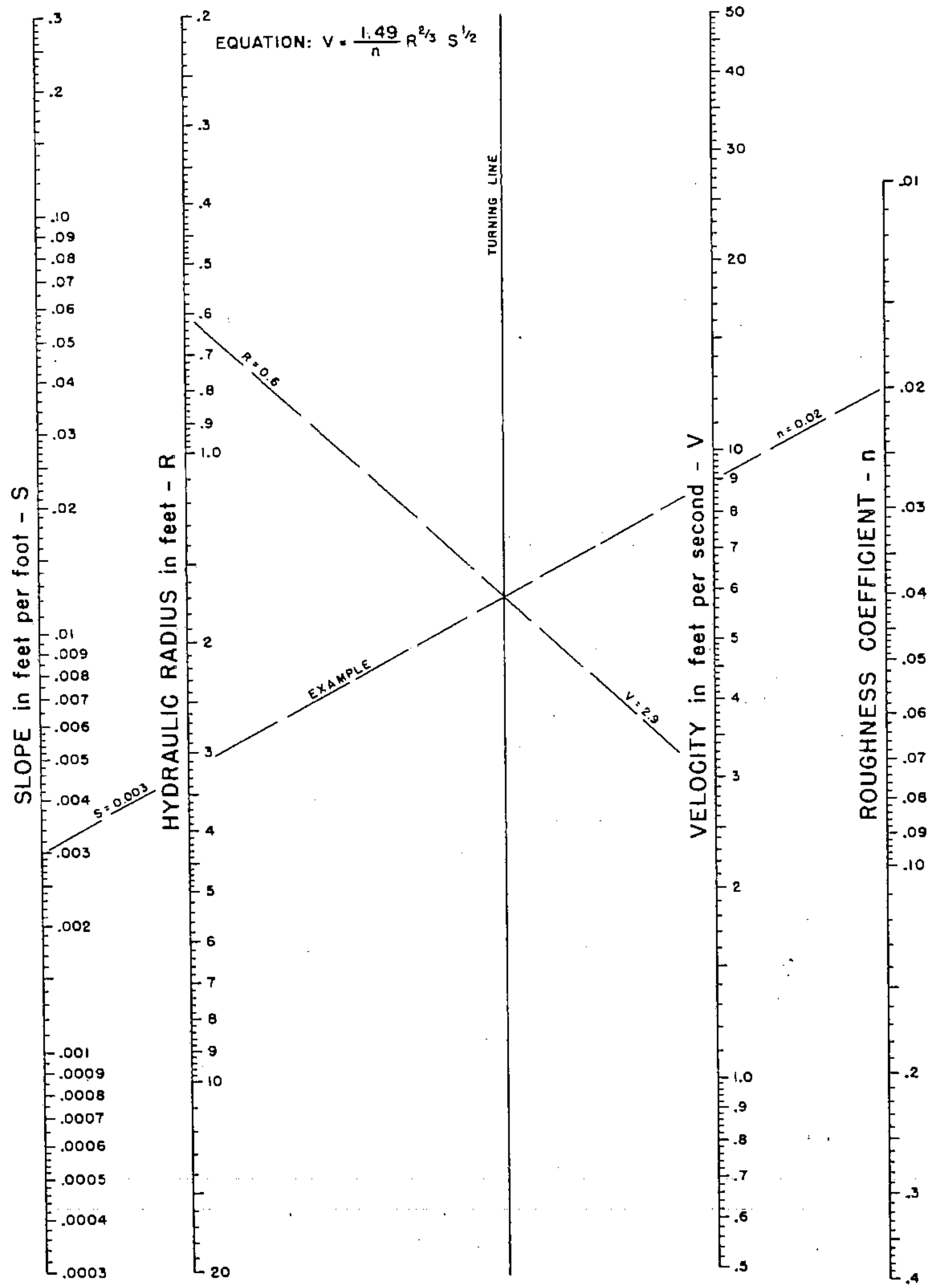

Source: U.S. DOT, FHA, HDS-3 (1961)

FIGURE 5-11. Nomograph for solving Manning's equation.

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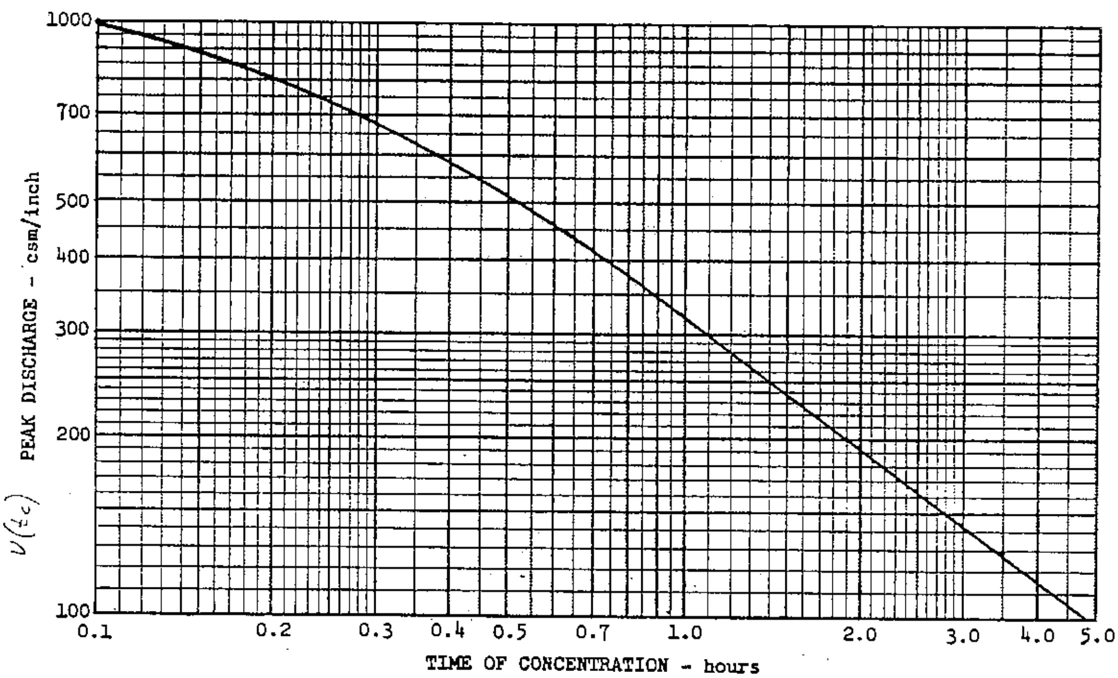

Note: CSM/inch = cubic feet per second per square mile of drainage area per inch of runoff volume.

Source: USDA, SCS, TR-55 (1975)

FIGURE 5-12. SCS TR-55 graphical peak flow method for a 24-hour type II design storm.

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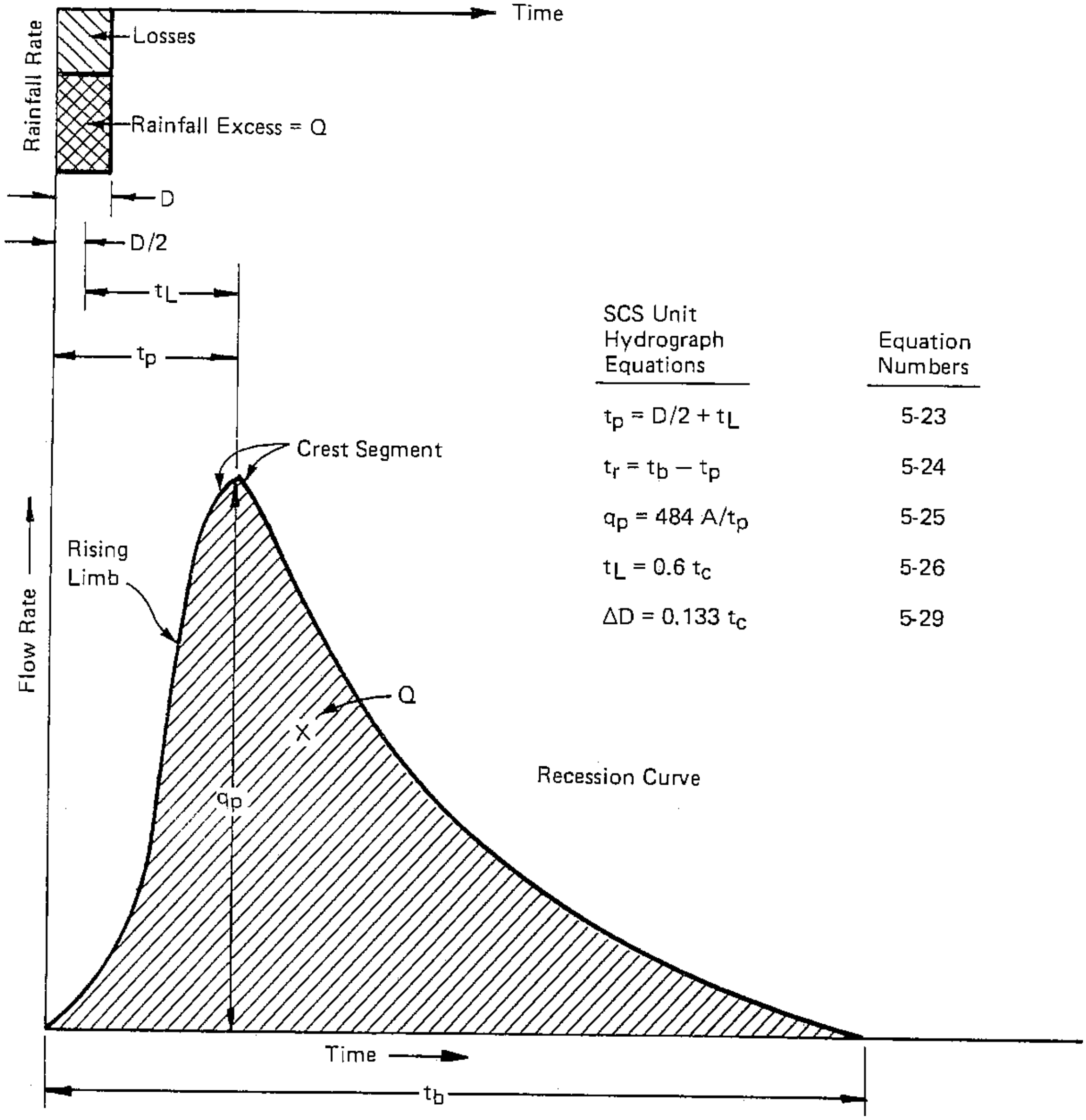


SCS Unit Hydrograph Equations	Equation Numbers
$t_p = D/2 + t_L$	5-23
$t_r = t_b - t_p$	5-24
$q_p = 484\ A/t_p$	5-25
$t_L = 0.6\ t_c$	5-26
$\Delta D = 0.133\ t_c$	5-29

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

FIGURE 5-13. General hydrograph terminology.

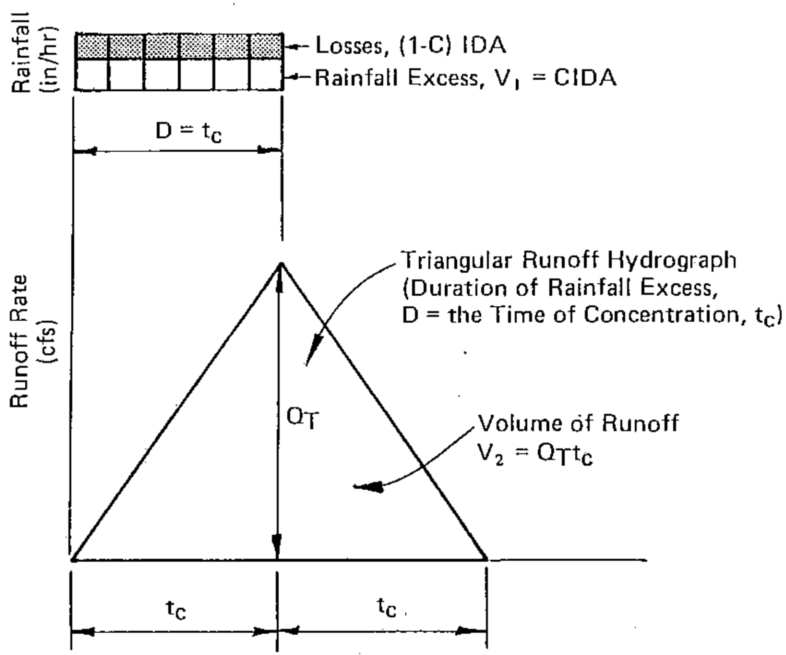


FIGURE 5-14. Rational method hydrograph for a storm duration equal to the time of concentration, equilateral triangle approximation.

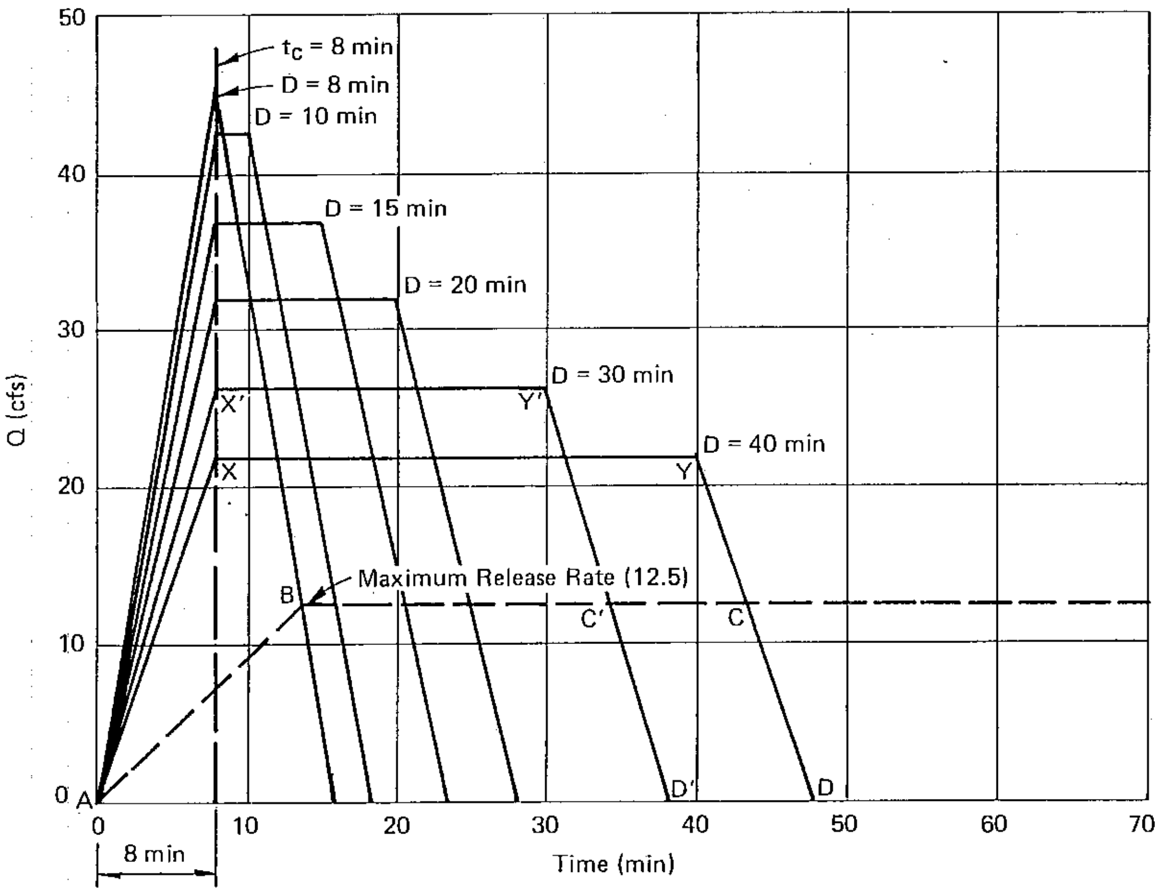
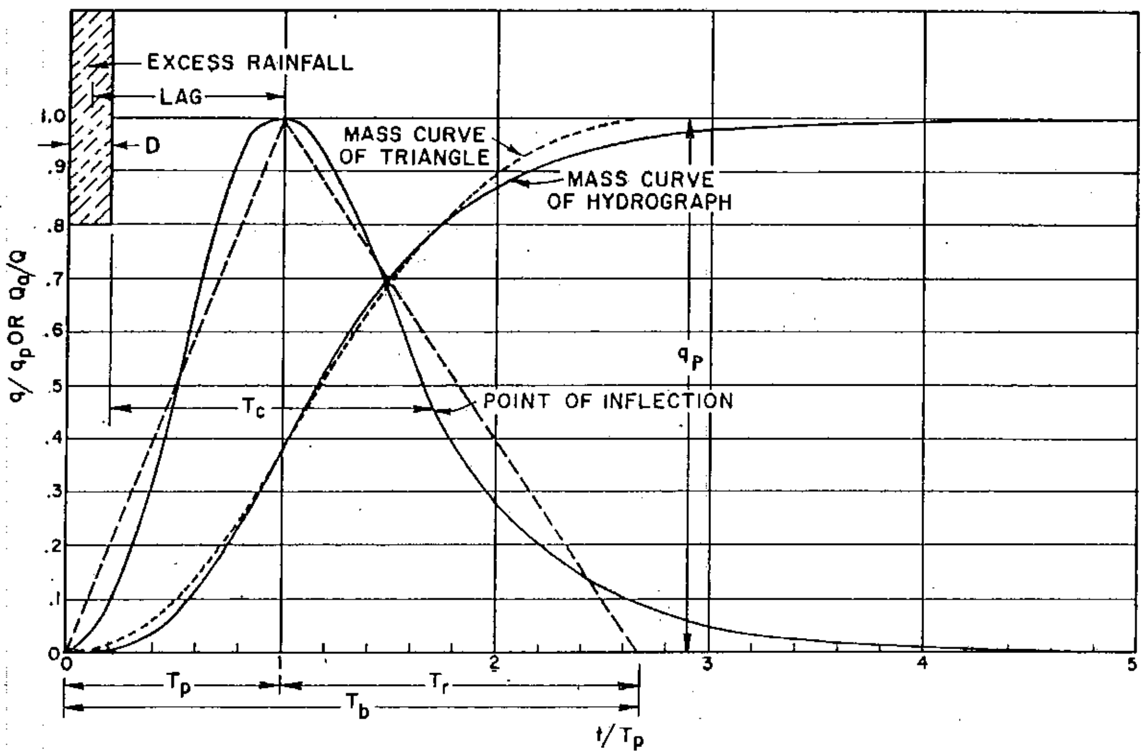


FIGURE 5-15. Rational method family of hydrographs for various storm durations.



Source: USDA, SCS, NEH-4 (1972)

FIGURE 5-16. Definition sketch of the SCS dimensionless unit hydrographs.

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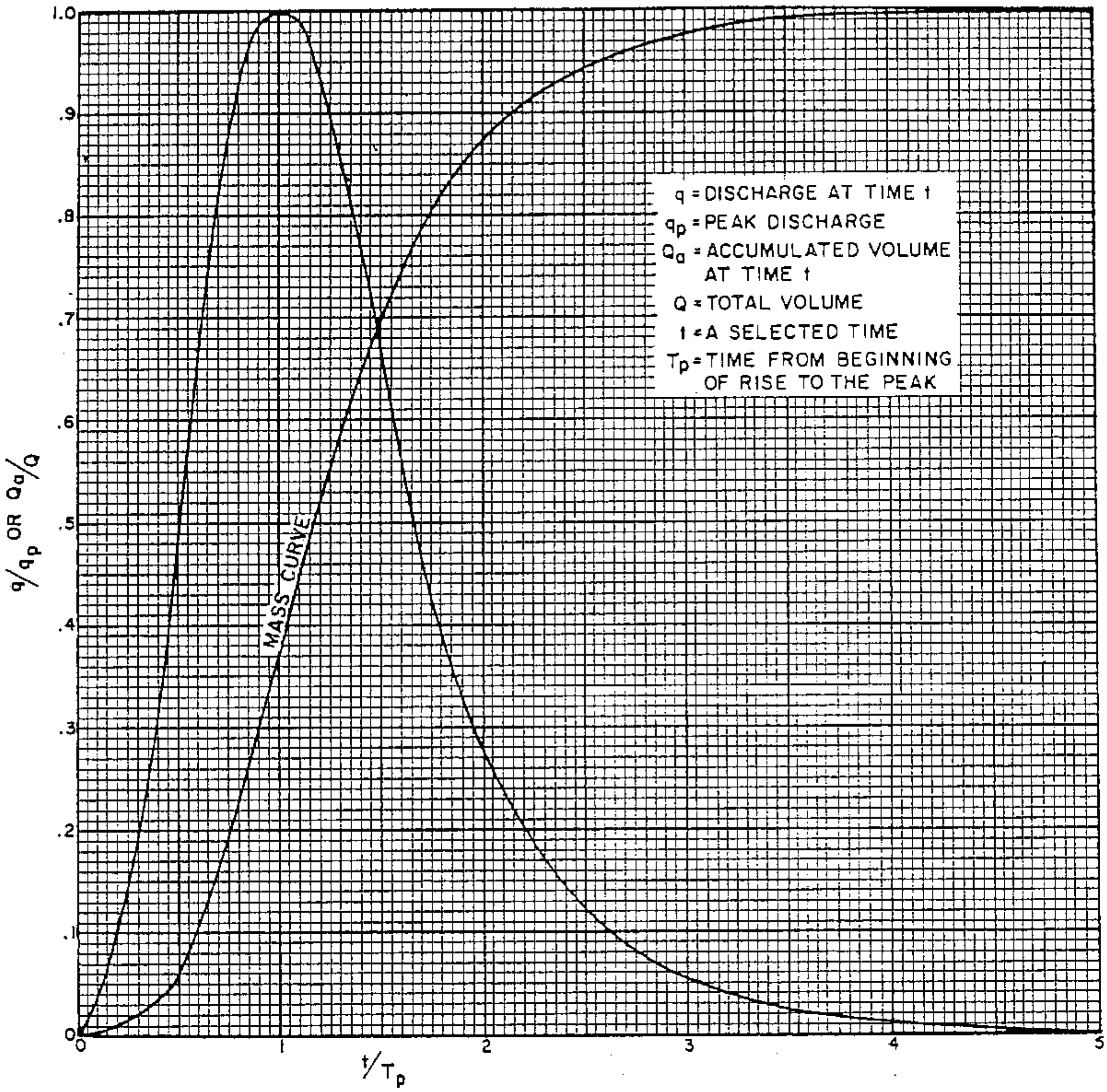

Source: USDA, SCS, NEH-4 (1972)

FIGURE 5-17. SCS dimensionless unit hydrographs and mass curves.

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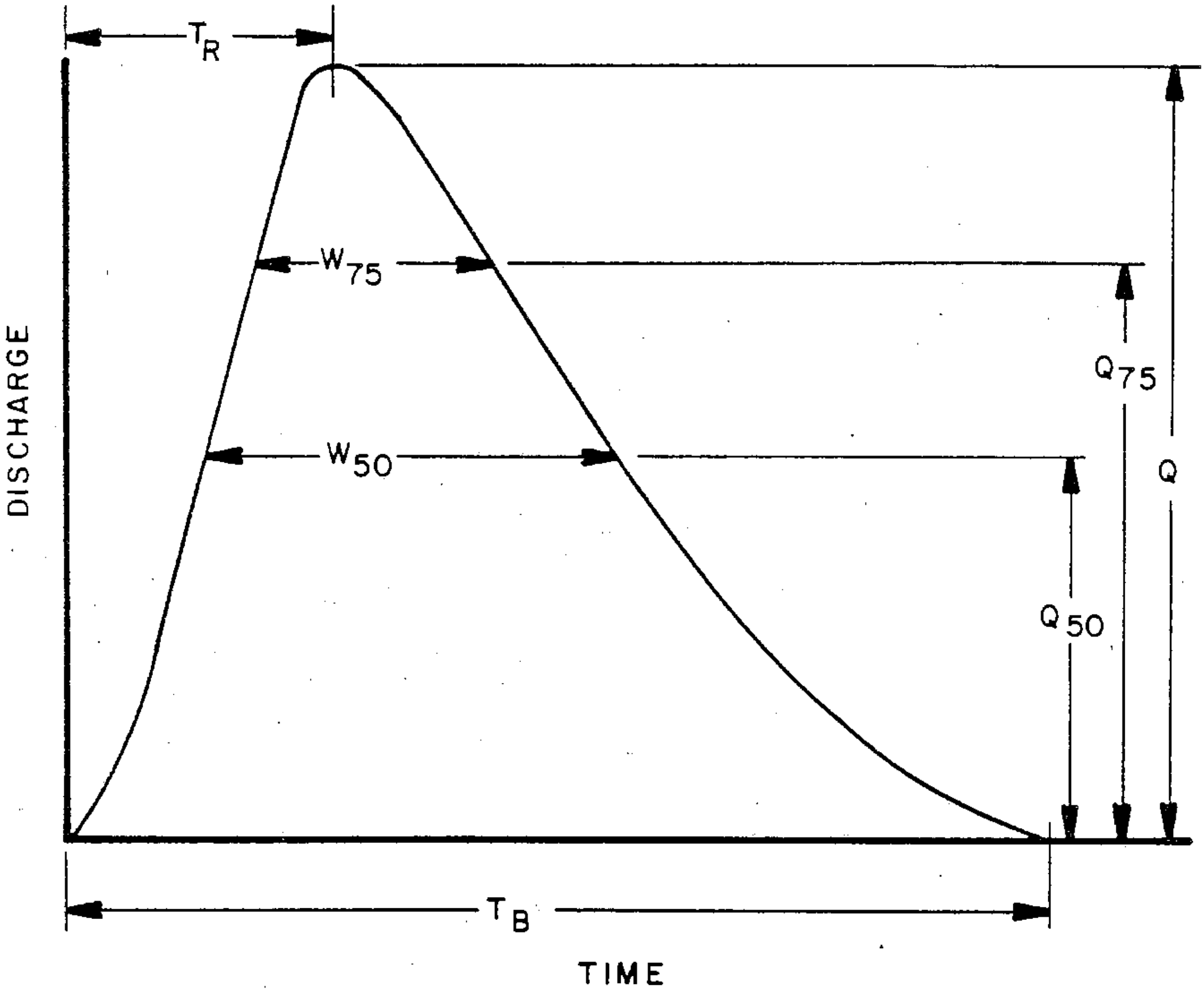
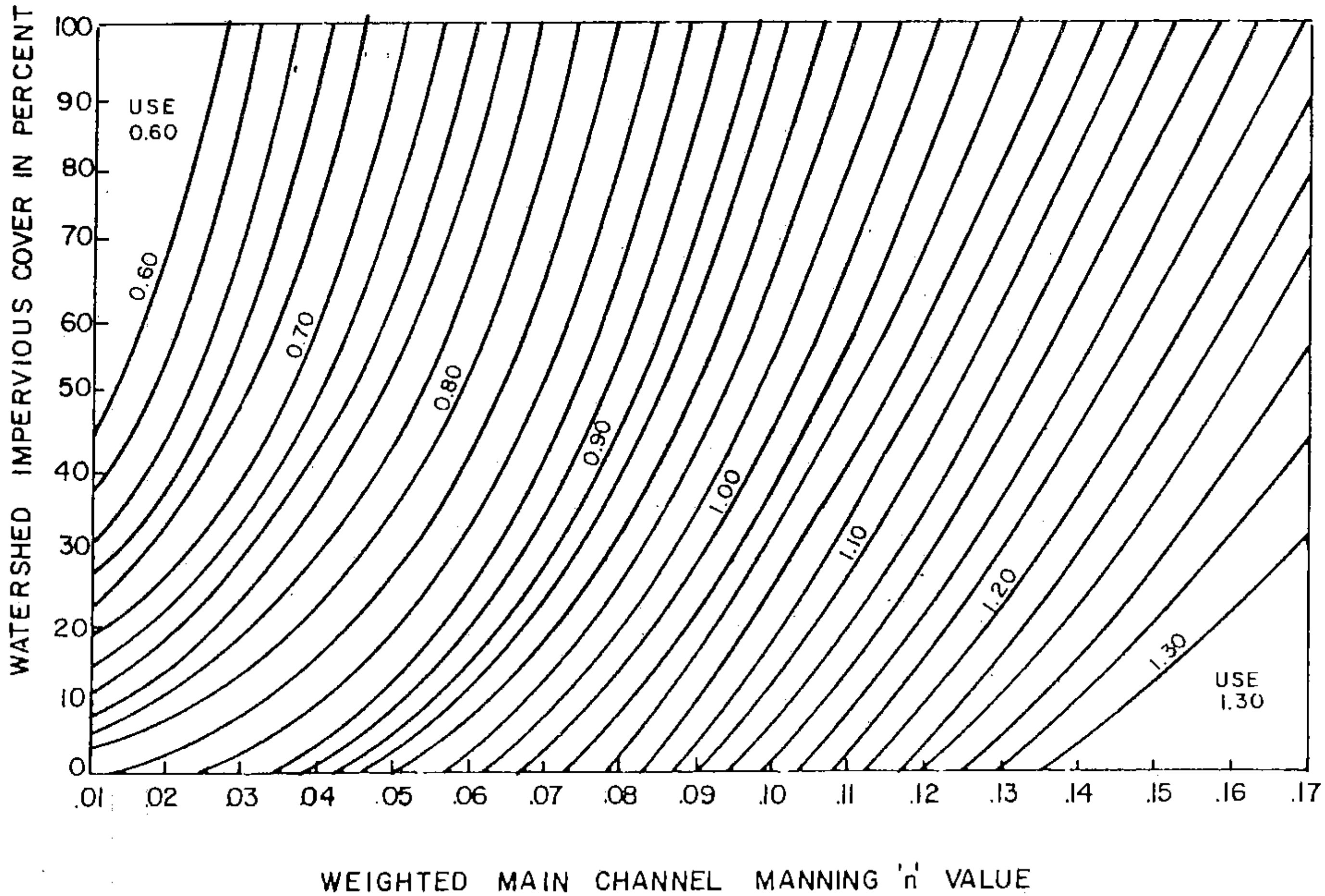


Source: Espey, et al. (1977)

FIGURE 5-18. Definition sketch of parameters determined by the unit hydrograph regression equations.



Source: Espey, et al. (1977)

FIGURE 5-19. Graphical solution for the watershed conveyance factor, Φ, used in the unit hydrograph regression equations.

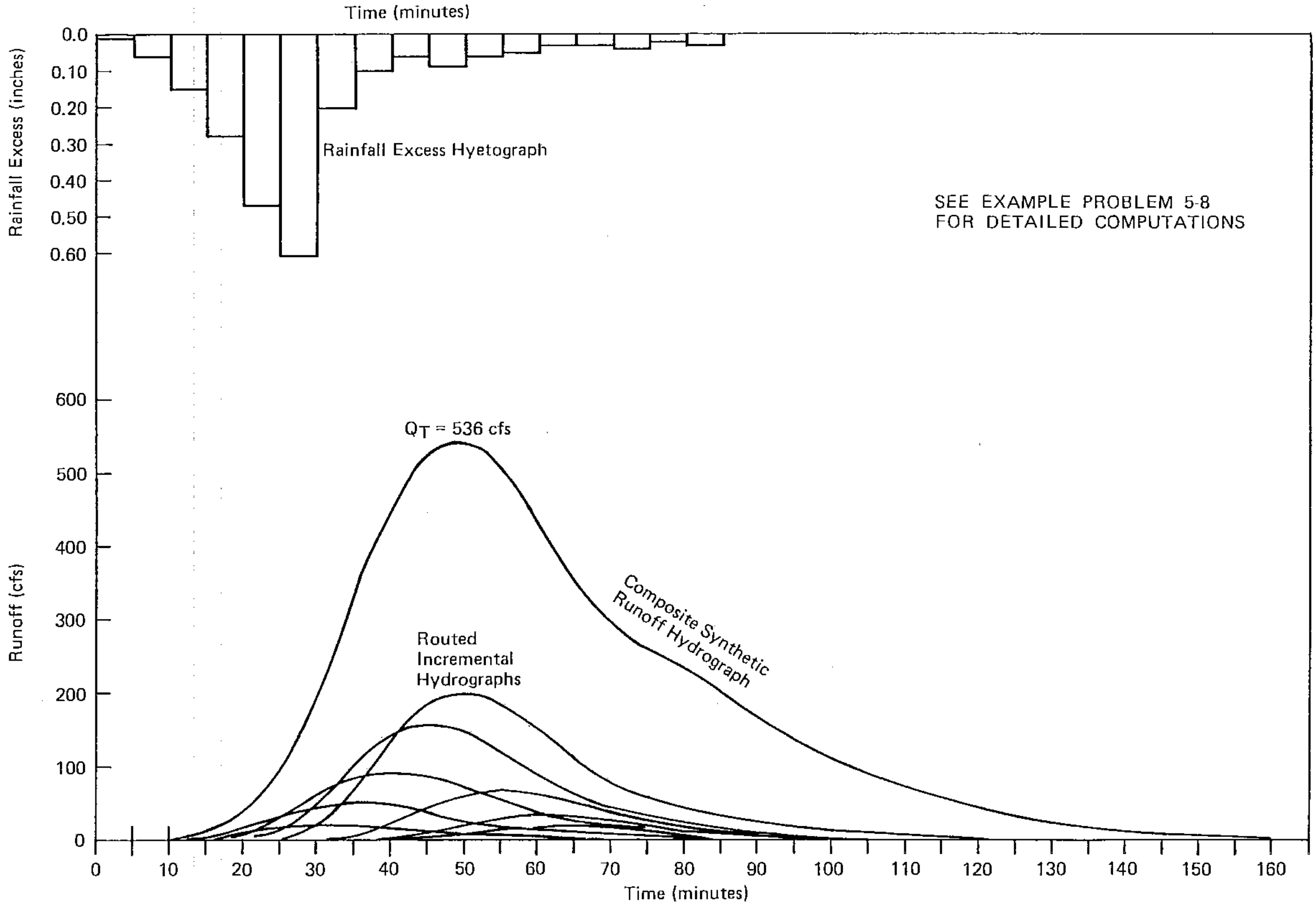


FIGURE 5-20. Application of unit hydrograph theory to develop a synthetic runoff hydrograph for subbasin 2 in West End Ditch.

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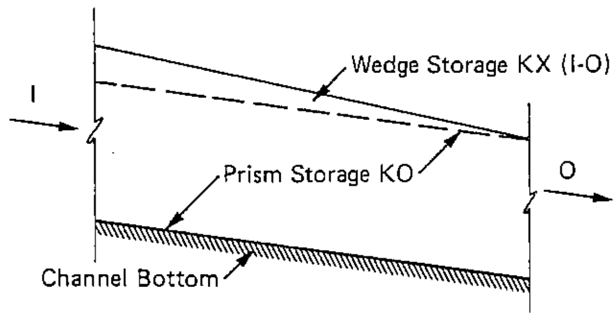

Note: Channel Storage, S = KX + KX (I-O)

FIGURE 5-21. Muskingum method two component channel storage approximation.

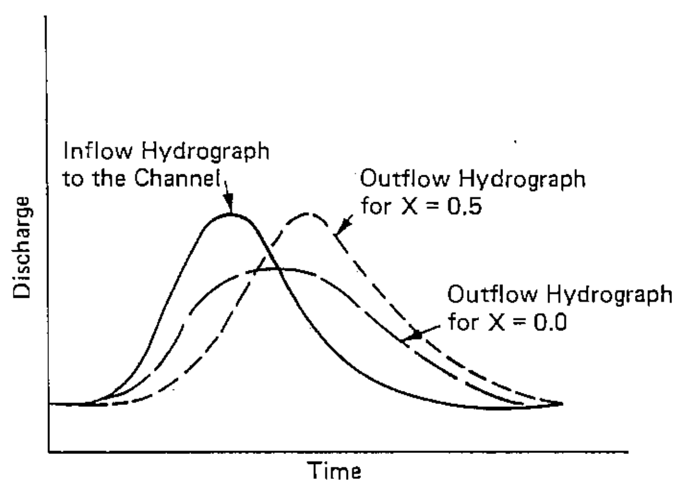

FIGURE 5-22. Impact of the Muskingum method X parameter on the outflow hydrograph from a channel.

SECTION 1.0 INTRODUCTION

The analysis of sediment yields is an important component of the hydrologic analysis when any stormwater drainage system is designed and constructed. Typical problems which have resulted in the Montgomery area due to inadequate provisions for stormwater sediment yields were discussed in Chapter 2. Although the damage caused by urban stormwater sediment does not result in a direct economic hardship onsite, as is caused by agricultural soil losses, major impacts can be experienced downstream, as culverts, open channels, storm sewers, and reservoirs become filled with sediment.

The primary tool for quantifying the loss of soil by erosion is the Universal Soil Loss Equation (USLE). Although the soil loss computed by the USLE may not be the actual loss, it does provide a practical approach for planning site changes such that the predicted soil loss is minimized. Since the USLE does not account for sediment deposition and erosion from gullies and stream banks, a sediment delivery ratio is required if the design point in question is a significant distance from the area where the USLE is applied.

The material presented in this chapter focuses on application of the USLE. Details related to the performance of erosion and sediment control systems are presented in Chapter 9. In all cases the average annual soil loss from a construction site shall be less than 15 tons/acre/year. In addition, the standard specifications of the Alabama Highway Department for temporary erosion control measures shall apply to the City of Montgomery.

SECTION 2.0 SOIL EROSION PROCESS

The sediment yield at any particular point in a watershed is affected by three processes: soil detachment, soil transport, and deposition of the detached soil. Soil detachment and transport processes can be further classified into the following subprocesses:

1. Detachment by rainfall
2. Detachment by runoff
3. Transport by rainfall
4. Transport by runoff

A conceptual diagram (see Figure 6-1) for considering the interrelationships of these subprocesses was presented by Meyer and Wischmeier (1969). According to this conceptual diagram, the soil available for transport is that detached on a particular segment by rainfall and runoff plus that carried to it by the upslope segment. A comparison of this total detached soil with the total transport capacity determines the soil carried downslope. For example, if the total detached soil is less than the transport capacity available, all of the detached soil will be carried downslope, and the potential for additional soil detachment exists. Conversely, if the total transport capacity is less than the total detached soil, the sediment load carried downslope will equal the transport capacity and deposition will occur. In other words, the sediment yield from a particular site will be determined by the limiting factor (i.e., detached soil or transport capacity).

The mechanisms which control the sediment yield from a particular site can also provide valuable information for developing an erosion and sediment control plan. For example, if the sediment yield from a site were controlled by the total detached soil, onsite erosion control practices which reduce the effect of raindrop impacts and the shear forces of flowing stormwater runoff would be appropriate. Conversely, if the sediment yield from a site were controlled by the total transport capacity, erosion control practices which decrease the stormwater transport capacity would be appropriate. In practice, a combination of soil detachment and transport capacity reduction practices are employed since the conditions that define which mechanism controls the sediment yield are highly variable throughout the lifetime of a stormwater drainage system.

SECTION 3.0 UNIVERSAL SOIL LOSS EQUATION

The USLE is an established procedure for estimating soil losses from upland slopes. Although it was developed for agricultural purposes, the USLE has been successfully adapted to construction sites. The equation contains factors which relate to the rainfall, soils, runoff, and erosion control practices. The gross erosion produced by rill and interill erosion from a field-sized upland area can be estimated using the USLE expressed as follows:

$$A = RKLSCP \tag{6-1}$$

where

A = soil loss, in tons per acre, for the time period selected for R

R = rainfall factor

K = soil erodibility factor, in tons per acre per R unit

LS = length-slope factor, dimensionless

C = cropping management factor, dimensionless

P = erosion control practice factor, dimensionless

To determine a sediment yield at some point beyond the field-sized upland area, additional erosion from gullies and stream banks must be added and deposition subtracted. The gross erosion from an upland area can be estimated for average annual, average monthly, return period annual, or return period single-storm time scales. In addition, it can be converted from weight in tons to volume in cubic yards by the appropriate conversion factors in Table 6-1.

Numerical values for each of the parameters in the USLE must be determined for each problem considered. Guiding principles and data for determining these parameters in the Montgomery area are presented in the following discussion.

3.1 Rainfall Factor

The numerical value used for the rainfall factor, R, in the USLE must quantify the effect of raindrop impact and provide relative information on the amount and rate of runoff likely to be associated with the rainfall. Research has indicated that the value of R can be determined directly as the product of the total storm energy, E, and the maximum 30-minute rainfall intensity, I. The relation of this product parameter, EI, to soil loss has been found to be linear such that values determined from individual storms are additive. Thus, the quantitative measure of the erosion potential for a given period of rainfall is the sum of the product EI values within that period of rainfall.

Wischmeier and Smith (1978) have analyzed 22 years of rainfall data at various locations across the United States to determine appropriate rainfall factors for various time scales. Storm events of less than 1/2 inch and separated from other storm events by more than 6 hours were omitted from the rainfall factor calculations unless as much as 0.25 inches fell in 15 minutes. The results of this analysis are published for average annual, average monthly, return period annual, and return period single-storm time scales.

3.1.1 Annual Factors. Annual rainfall factors reported by Wischmeier and Smith (1978) for Montgomery are presented in Table 6.2. The observed range in the rainfall factor for the 22-year period was found to be from 164 to 780. The average annual rainfall factor was found to be 350, while for return periods of 2-, 5-, and 20-years, it was found to be 359, 482, and 638, respectively.

3.1.2 Monthly Distribution. The monthly distribution or cumulative percentage of the average annual rainfall erosion index for Montgomery is presented on Figure 6-2. This distribution can be utilized to estimate soil losses for periods less than 1 year. For example, according to Figure 6-2, 41 percent of the annual rainfall erosion index occurs between the first of May and the first of August. Therefore, the percentage of the average annual erosion expected to occur during this time period can be estimated by multiplying 350 by 0.41.

3.1.3 Single-Storm Factors. Expected average values of the rainfall index for single storms are presented in Table 6-3. Although the USLE is not recommended for predicting the actual soil loss for specific design storms, it will estimate the average soil loss for a large number of storms of a given magnitude. However, the soil loss from any one of these events may differ widely from the predicted average. This is due to the random fluctuations of other variables which affect the soil loss from any single-storm event. For example, when rain falls on relatively dry soil, most of the water may infiltrate before runoff begins, resulting in a soil loss lower than the average determined by the USLE. Conversely, when rain falls on presaturated soil, runoff begins quickly, and most of the rain becomes runoff, resulting in a soil loss higher than the average determined by the USLE.

To determine the expected average value of soil loss for a single storm, the R values reported in Table 6-3 are utilized directly in the USLE (Equation 6-1). For example, if the expected average soil loss for a 5-year design storm is desired, an R value of 118 is used in the USLE.

3.2 Soil Erodibility Factor

The soil erodibility factor, K, in the USLE is a quantitative parameter which must be determined experimentally. It is defined as the rate of soil loss per erosion index unit, measured on a unit plot for a given type of soil. The unit plot which is used to determine K values has been defined arbitrarily to match those field conditions under which past erosion measurements have been made experimentally. A unit plot is 72.6 feet long, with a uniform lengthwise slope of 9 percent in continuous fallow, tilled up and down the slope. Continuous fallow, for this purpose, is land that has been tilled and kept free of vegetation for more than 2 years.

More than 25 characteristics of a soil affect its response to water erosion. A few of the most important characteristics include the texture and organic matter of the surface layer, size and stability of structural aggregates in the surface layer, permeability of the subsoil, and depth to slowly permeable layers. Several K factors may be determined for a soil series depending on the profile characteristics. Published K factors for the Montgomery area are reported in Table 4-4.

3.3 Length-Slope Factor

Theoretically, the effect of slope length and steepness on soil loss are considered separately. However, in practice these two factors are combined in the single length-slope topographic factor, LS. The LS value is defined as the ratio of soil loss per unit area from a given site to that from a site with a 72.6-foot length and a uniform slope of 9 percent under otherwise identical conditions. A site-specific value of LS can be obtained directly from Figure 6-3, which is a graphical presentation of the following equation.

$$LS = \left(\frac{\lambda}{72.6}\right)^m (65.41 \sin^2\theta + 4.56 \sin\theta + 0.065) \quad (6\text{-}2)$$

where

LS = length-slope factor, dimensionless

λ = slope length, in feet

θ = angle of slope

m = 0.5 if the percent slope is 5 or more, 0.4 on slopes of 3.5 to 4.5 percent, 0.3 on slopes of 1 to 3 percent, and 0.2 on slopes less than 1 percent

To utilize Figure 6-3, enter the abscissa with the actual slope length of the site in question. Move vertically to the appropriate percent-slope curve, and read the LS value on the ordinate scale.

The relationship expressed by Equation 6-2 was derived from data obtained on slopes ranging from 3 to 18 percent and about 30 to 300 feet in length. How far beyond these ranges in length and slope the equation is valid has not been determined by direct soil loss measurements. In addition, it is important to note that Equation 6-2 and Figure 6-3 are derived for uniform slopes. If the actual slope is irregular special considerations may be required as discussed below.

The soil loss to the foot of a convex slope will be underestimated using Figure 6-3 or Equation 6-2. Conversely, the soil loss from a concave slope will be overestimated when the LS factor is determined using this procedure. Typical concave and convex slopes are illustrated on Figure 6-4. Concave and convex slopes may have the same average slope and length; however, the soil loss will be different. Other factors being equal, the convex slope will have the highest sediment production because the steepest slope is nearest the receiving water.

Irregular slopes can be analyzed using Figure 6-3 or Equation 6-2 by dividing the irregular slope into a small number of equal-length and uniform segments. If this is done, two simplifying assumptions must be valid:

1. The changes in gradient are not sufficient to cause upslope deposition.

2. The irregular slope can be divided into a small number of equal-length segments in such a manner that the gradient within each segment is uniform.

After dividing the convex, concave, or complex (comprised of both concave and convex components) slope into equal-length segments, the LS factor is determined as follows:

1. List the segment gradients in the order in which they occur on the slope, beginning at the upper end.

2. Enter the abscissa of Figure 6-3 with the total slope length and read the LS factor for each of the gradients listed in step 1.

3. Multiply these LS factors by the appropriate factors from Table 6-4.

4. Sum the products obtained from Step 3 to obtain the LS factor for the entire slope.

Research has not defined just how much change in slope is required to induce the deposition of eroded soil. However, in practice, areas of deposition should be identified by observation. When the slope breaks are sharp enough to cause deposition, the four-step procedure described above can be used to estimate the LS factor for slope segments above and below the point of deposition. This procedure will not provide an estimate of the total soil loss from such an irregular slope, because the USLE does not account for deposition.

3.4 Control Practice Factor

As originally proposed by Wischmeier and Smith (1978), the control-practice factor, CP, consisted of two separate factors (i.e., C and P factors). The cover factor, C, was defined as the ratio of soil loss from an area with specified cover and management conditions to the soil loss from an identical area in tilled continuous fallow. The support practice factor, P, was defined as the ratio of soil loss with a support practice such as contouring, stripcropping, or terracing to that with straight-row farming up and down the slope. Since conditions of a cleared construction site are similar to those of a tilled field in continuous fallow, a similar procedure can be utilized to evaluate construction erosion control practices.

For construction sites, Chen (1974) proposed that the individual C and P factors of the USLE be evaluated with a single control-practice factor, CP, which is defined as follows:

$$CP = C_s \, C_r \, C_o \qquad (6\text{-}3)$$

where

CP = control-practice factor

C_s = control due to surface stabilization such as seeding, mulching, and netting

C_r = control due to runoff-reduction practices such as diversion berms, interceptor dikes, terraces, sodded ditches, level spreader, and sectional downdrains

C_o = control due to any other erosion control practice not noted above

Chen (1974) also proposed control factors related to the exposure scheduling of a cleared site and sediment trapping measures. Due to theoretical difficulties in determining a site-specific value of the CP factor for exposure scheduling, a procedure which utilizes the monthly distribution of the rainfall factor, R (see Subsection 3.2 of Chapter 9), is presented in Chapter 9, Subsection 3.2. In addition, for the purposes of this design manual, the USLE is utilized to evaluate onsite erosion control measures. Since a sediment trapping device acts to control erosion away from the site of erosion, it will be accounted for when the sediment delivery ratio is evaluated.

Detailed information for determining quantitative values of the CP factor for selected erosion control systems for various types of land use and cover conditions is presented in Chapter 9.

Any temporary erosion control system shall conform to the requirements of the Alabama Highway Department Standard Specifications. According to Section 665 of these specifications, a contractor shall not expose more than 17.5 acres of erodible material for any separate major operation without prior approval of the engineer. Requirements of the following temporary onsite erosion control measures shall apply:

3.5 Soil Loss Tolerance

Extensive field data relating crop productivity and onsite soil loss has indicated that there is a maximum soil loss that will permit a high level of crop productivity to be sustained economically and indefinitely. This maximum level of onsite soil loss has been termed the "soil loss tolerance" (Wischmeier and Smith, 1978). Soil-loss tolerances ranging from 5 to 2 tons per acre per year for soils of the U.S. were derived by soil scientists, agronomists, geologists, soil conservationists, and Federal and State research leaders at six regional workshops in 1961 and 1962.

Soil-loss tolerances for soils common in the Montgomery area are listed in Table 4-4 along with soil erodibility factors. These soil-loss tolerances provide a planning guideline for evaluating an onsite erosion control program. For example, once the USLE has been applied to a particular site, onsite erosion control measures can be selected which reduce the uncontrolled soil loss to the appropriate soil-loss tolerance. Since the soil-loss tolerance values reported in Table 4-4 were developed for agricultural purposes, they may not be appropriate for construction sites; however, they do establish a basis for comparison. To put the soil loss tolerances in perspective, the loss of 1 inch of soil per year would result in a soil loss of 145 tons per acre per year. In all cases, the average annual soil loss from a construction site in Montgomery shall be less than 15 tons per acre per year.

Factors to consider when evaluating soil-loss tolerances include the distance of the field from a stormwater conveyance system, the sediment transport characteristics of the intervening area, sediment size distribution, and the probable magnitude of fluctuations in the soil loss.

3.6 Example Problems

Example 6-1. Length-Slope Factors on Irregular Slopes

A 400-foot convex slope has a 1 percent uniform slope on the upper third, a 5 percent uniform slope on the middle third, and a 10 percent uniform slope on the bottom third. Determine a length-slope (LS) factor for the entire slope.

1. List the segment gradients as illustrated below in columns 1 and 2:

(1) Segment	(2) Percent Slope	(3) LS Factor from Figure 6-3	(4) Soil Loss Fraction from Table 6-4	Product of Columns 3 and 4
1	1	0.195	0.24	0.047
2	5	1.08	0.35	0.378
3	10	2.74	0.46	1.260
				Σ = 1.685

2. Enter the abscissa of Figure 6-3 with the total slope length of 400 feet and read the LS factor for each uniform segment of the convex slope and list it in column 3 above.

3. Multiply these LS factors by the appropriate factors from Table 6-4.

   For segment 1, $m = 0.3$ and the soil loss fraction is 0.24. For segment 2, $m = 0.5$ and the soil loss fraction is 0.35. For segment 3, $m = 0.5$ and the soil loss fraction is 0.46. Each of these soil loss fractions is listed in column 4 of the above table.

4. Find the product of each LS factor and the associated soil loss fraction (columns 3 and 4). Sum these products to obtain a composite LS factor for the convex slope. For this example, LS is found to be 1.685.

Example 6-2. Average Annual Soil Loss From a Construction Site

A residential development is planned for construction in an area of Montgomery covered exclusively by the Pheba Soil Series. If the cleared land is left unprotected during construction, what is the average annual soil loss in tons per acre per year from an individual lot? Assume that the cleared sites are cut 12 inches on the average, are graded to have a uniform slope of 1 percent, and that the average slope length is 200 feet.

1. From Table 6-2, the average annual rainfall factor, R, for the USLE is 350.

2. From Table 4-5, the soil erodibility factor, K, for Pheba exposed at 12 inches of depth, is 0.49.

3. From Figure 6-3, the length-slope, LS, factor for a 200-foot slope of 1 percent, is 0.60.

4. Since the sites are unprotected, CP = 1.0.

5. According to the USLE Equation 6-1, the average annual soil loss is:

$$A = (350)(0.49)(0.60)(1.0) = 102.9 \text{ tons/acre-year}$$

Example 6-3. Soil Losses for Conditions Other Than Average Annual

Using the soil loss data for a residential development as presented in example problem 6-2, determine the soil loss from March 1 through July 1, from a 10-year design storm, and for 20-year return period annual conditions.

1. Using Figure 6-2, the cumulative percentages of the average annual rainfall erosion index for March 1 and July 1 in Montgomery are 12 percent and 50 percent, respectively. This means that 38 percent of average annual soil loss may be expected during this 4-month period. Therefore, the soil loss during this period is estimated as follows:

   $A = (102.9)(0.38) = 39.1$ tons/acre
   between March 1 and July 1

2. Using Table 6-3, the USLE rainfall factor for a 10-year design storm is 145. Therefore, the soil loss from a 10-year design storm is estimated as follows:

   $A = (145)(0.49)(0.60)(1.0) = 42.6$ tons/acre
   for a 10-year design storm

3. Using Table 6-2, the annual USLE rainfall factor expected for 20-year return period conditions is 638. Therefore, the soil loss for 20-year return period annual conditions is calculated as follows:

$$A = (638)(0.49)(0.60)(1.0) = 187.6 \text{ tons/acre-year}$$
for 20-year return period conditions

Example 6-4. Soil Loss Reduction Due to Control Practices

An undisturbed woodland is to be cleared for construction between April 1 and June 1. Using the data from example problem 6-2, what combination of on-site erosion control practices will limit the average annual soil loss from the site to 15 tons per acre per year?

1. Assuming that the site is undisturbed from January 1 to April 1, cleared from April 1 to June 1, covered with temporary seeding from June 1 to August 1, and permanently seeded from August 1 to December 31, a weighted CP factor can be determined using Figure 6-1 and Tables 9-1, 9-3, and 9-5 as follows:

Time Period	Surface Cover	C_s Factor	Fraction of Annual R	Weighted C_s
1/01 to 4/01	Undisturbed Woodland	0.003	0.20	0.0006
4/01 to 6/01	Cleared Site	1.0	0.18	0.18
6/01 to 8/01	Temporary Seeding	0.40	0.31	0.12
8/01 to 12/31	Permanent Seeding	0.05	0.31	0.02
			$\Sigma =$	0.32

Therefore, according to Equation 6-3, CP = 0.32.

2. According to the USLE Equation 6-1, the average annual soil loss is:

$$A = (350)(0.49)(0.60)(0.32) = 32.9 \text{ tons/acre-year}$$

3. If the cleared site is mulched with straw at a rate of 1 ton/acre, the C_s factor is 0.20. Therefore, according to Equation 6-1:

$CP = (0.32)(0.20) = 0.064$

and the soil loss is

$A = (350)(0.49)(0.60)(0.064) = 6.6$ tons/acre-year

Therefore, straw mulch is adequate protection to meet the 15-ton-per-acre-per-year limit.

SECTION 4.0 SEDIMENT DELIVERY RATIOS

The USLE provides a practical method for estimating the gross soil loss from a field-sized upland area. Numerous natural or manmade opportunities for sediment deposition can exist from the point of origin to the design point in question. It is also possible that additional soil may be eroded from the gullies or stream banks which transport stormwater from the point of origin to the design point in question. Therefore, it is possible for the sediment yield at a particular design point in a stormwater conveyance system to be greater or smaller than the gross soil loss estimated using the USLE. A sediment delivery ratio can be utilized to quantify the sediment transported to a particular design point. The sediment delivery ratio is defined as follows:

$$D = \frac{Y}{A_T} \qquad (6\text{-}4)$$

where

D = sediment delivery ratio without manmade controls

Y = sediment yield from a watershed, without manmade controls in tons/acre for the specified time period.

A_T = total gross erosion from the watershed, which includes upland sheet and rill erosion, gully erosion, and stream erosion, in tons/acre for the specified time period.

The sediment yield, Y, accounts for the total deposition which occurs in the watershed, while the total gross erosion, A_T, includes additional erosion which is not predicted by the USLE. Very limited information is available to quantify sediment delivery ratios without actual field data. If site-specific field data are not available, computer simulation may be a valid alternative for quantifying the sediment delivery ratio parameters. Additional information on sediment delivery ratios may be obtained from Williams (1975), USDA, SCS (1971), and Graf (1971).

In Montgomery, the sediment delivery ratio of a cleared construction site is likely to be close to 1.0 since the maximum allowable area of exposure is 17.5 acres (see Subsection 3.4 of this chapter). Therefore, according to Equation 6-4, the gross erosion, A_T, is the same as the sediment yield from a watershed without manmade controls, Y. For the purposes of planning sediment and erosion control systems, the sediment delivery ratio with controls can be defined as follows:

$$D_C = \frac{Y - X - O}{A_T} \qquad (6-5)$$

where

D_C = sediment delivery ratio with controls

X = sediment trapped offsite by appropriate sediment removal controls, in tons/acre for the specified time period

O = onsite soil loss reduction by appropriate erosion control techniques, in tons/acre for the specific time period

Y and A_T are defined above

If the sediment delivery ratio, D, is 1.0, then $Y = A_T$, and the product of A_T and D_C should be less than 15 tons/acre/year. Details related to determining T, the sediment trapped offsite by appropriate sediment removal controls, and O, the onsite soil loss reduction by appropriate erosion control techniques, are presented in Chapter 9.

4.1 Example Problem

Example 6-5. Sediment Delivery Ratio Computations

The undisturbed woodland considered in example 6-4 is to be cleared for construction bewteen April 1 and September 1. What sediment delivery ratio with controls will cause the product of A_T and D_C to be less than 15 tons per acre/year? Assume that the sediment delivery ratio without controls is equal to 1.0.

1. Using the following data, compute the average annual soil loss without controls using the USLE, Equation 6-1.

Time Period	Surface Cover	C_s Factor	Fraction of Annual R (from Figure 6-2)	Weighted C_s
1/01 to 4/01	Undisturbed Woodland	0.003	0.20	0.0006
4/01 to 9/01	Cleared Site	1.0	0.60	0.60
9/01 to 11/01	Temporary Seeding	0.40	0.13	0.05
11/01 to 12/31	Permanent Seeding	0.05	0.07	0.0035
			$\Sigma =$	0.65

$A = (350)(0.49)(0.60)(0.65) = 66.9$ tons/acre/year

2. Assuming that gully erosion and stream erosion are 0, $A = A_T$. Using Equation 6-5 and the performance criterion of 15 tons/acre-year, the following relationship is obtained:

$$(A_T)(D_C) < T$$

$$(A_T)\left(\frac{Y - X - O}{A_T}\right) < T$$

where

T is the soil-loss tolerance of 15 tons/acre/year

Since D is assumed to be 1.0, $Y = A_T$, therefore:

$$(A_T)\left(\frac{A_T - X - O}{A_T}\right) < 15$$

which becomes

$$A_T - X - O < 15$$

$$66.9 - X - O < 15$$

3. Find O if the cleared site is mulched at a rate of 1 ton/acre. From Table 9-4, $C_s = 0.20$.

$$CP = (.65)(.20) = 0.13$$

$$A = (350)(0.49)(0.60)(0.13) = 13.4 \text{ tons/acre/year}$$

$$O = 66.9 - 13.4 = 53.5 \text{ tons/acre/year}$$

4. $66.9 - X - 53.5 < 15$

   $13.4 - X < 15$

   Therefore, X = 0.0 and straw mulch is likely to keep the soil loss during construction below 15 tons/acre/year. If onsite controls had not been adequate to meet the desired tolerance, offsite sediment trapping would be required.

SECTION 5.0 REFERENCES

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LIST OF EXAMPLE PROBLEMS--CHAPTER 6

LIST OF EQUATIONS--CHAPTER 6

LIST OF SYMBOLS--CHAPTER 6

A = soil loss, in tons/acre, for the time period selected for R

A_T = total gross erosion from the watershed, which includes upland sheet and rill erosion, gully erosion, and stream erosion, in tons/acre for the specified time period

C = cropping management factor, dimensionless

CP = control practice factor

C_o = control due to any erosion control practice not considered by C_r or C_s

C_r = control due to runoff-reduction practices such as diversion berms, interceptor dikes, terraces, sodded ditches, level spreader, and sectional downdrains

C_s = control due to surface stabilization such as seeding, mulching, and netting

D = sediment delivery ratio without manmade controls

D_c = sediment delivery ratio with controls

K = soil erodibility factor, in tons per acre per R unit

LS = length-slope factor, dimensionless

m = exponent for the LS Equation 6-2

O = onsite soil loss reduction by appropriate erosion control techniques in tons per acre for the specified time period

P = erosion control practice factor, dimensionless

R = rainfall factor

T = desired soil loss tolerance, in tons/acre/year

USLE = Universal Soil Loss Equation

X = sediment trapped offsite by appropriate sediment removal controls, in tons/acre for the specified time period

Y = sediment yield from a watershed, without manmade controls in tons per acre for the specified time period

λ = slope length, in feet

θ = angle of slope

Table 6-1
VOLUME WEIGHTS OF SOILS AND FACTORS FOR CONVERTING SOIL LOSSES (AIR-DRY) FROM TONS TO CUBIC YARDS

Soils	Volume Weight (lb/cubic feet)	Tons to Cubic Yards
Sands and loamy sands	110	0.67
Sandy loam	105	0.71
Fine sandy loam	100	0.74
Loam	90	0.82
Silt loam	85	0.87
Silty clay loam	80	0.93
Clay loam	75	0.99
Silty, sandy clay, and clay	70	1.06
Aerated Sediment	80[a]	0.93
Saturated Sediment	60[a]	1.24

Source: USDA-SCS-Florida (1978)

[a]These are the approximate aerated and saturated weights to be used at damage sites (streams or reservoirs).

Table 6-2
ANNUAL RAINFALL FACTOR, R, OBSERVED RANGE AND VALUES FOR 2, 5, AND 20 YEAR RETURN PERIODS IN MONTGOMERY, ALABAMA

Observed R Value Annual Range (22 Years)	Average Annual R Value	2-Year Return Period Annual R Value	5-Year Return Period Annual R Value	20-Year Return Period Annual R Value
164-780	350	359	482	638

Source: USDA--AH537 (1978).

Table 6-3
EXPECTED VALUE OF SINGLE STORM RAINFALL FACTORS IN MONTGOMERY, ALABAMA

1-Year Single-Storm R Value	2-Year Single-Storm R Value	5-Year Single-Storm R Value	10-Year Single-Storm R Value	20-Year Single-Storm R Value
62	86	118	145	172

Source: USDA--AH537 (1978).

Table 6-4
ESTIMATED RELATIVE SOIL LOSSES FROM SUCCESSIVE EQUAL-LENGTH SEGMENTS OF A UNIFORM SLOPE

Number of Segments	Sequence Number of Segment	Fraction of Soil Loss[a]		
		m = 0.5	m = 0.4	m = 0.3
2	1	0.35	0.38	0.41
	2	.65	.62	.59
3	1	.19	.22	.24
	2	.35	.35	.35
	3	.46	.43	.41
4	1	.12	.14	.17
	2	.23	.24	.24
	3	.30	.29	.28
	4	.35	.33	.31
5	1	.09	.11	.12
	2	.16	.17	.18
	3	.21	.21	.21
	4	.25	.24	.23
	5	.28	.27	.25

Source: USDA--AH 537 (1978).

[a]Derived by the formula:

$$\text{Soil loss fraction} = \frac{i^{m+1} - (i-1)^{m+1}}{N^{m+1}}$$

where

i = segment sequence number

m = slope-length exponent (0.5 for slopes $\geq$ 5 percent, 0.4 for 4 percent slopes, and 0.3 for $\leq$ 3 percent or less)

N = number of equal-length segments into which the slope was divided

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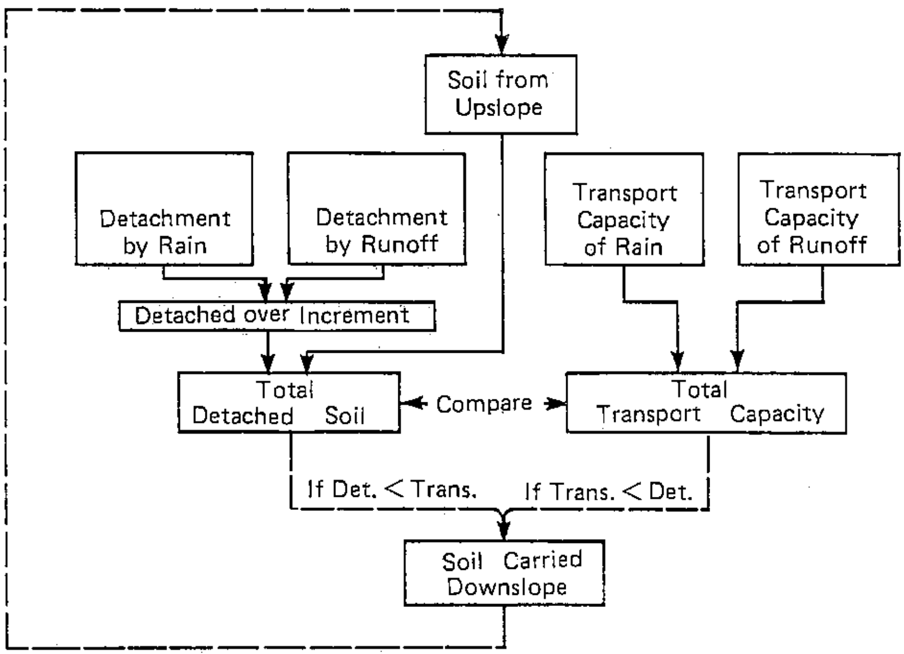
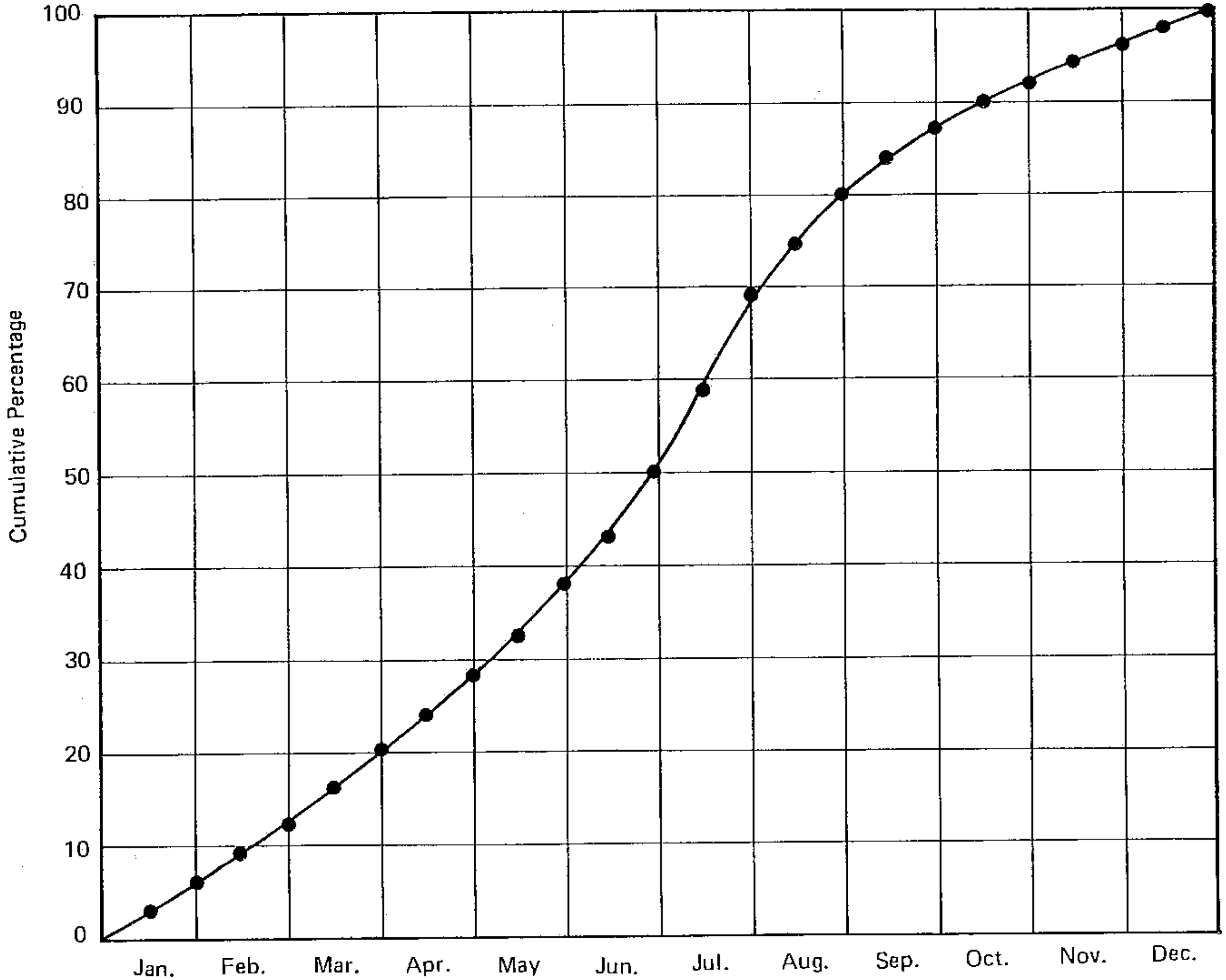
SOIL EROSION PROCESS



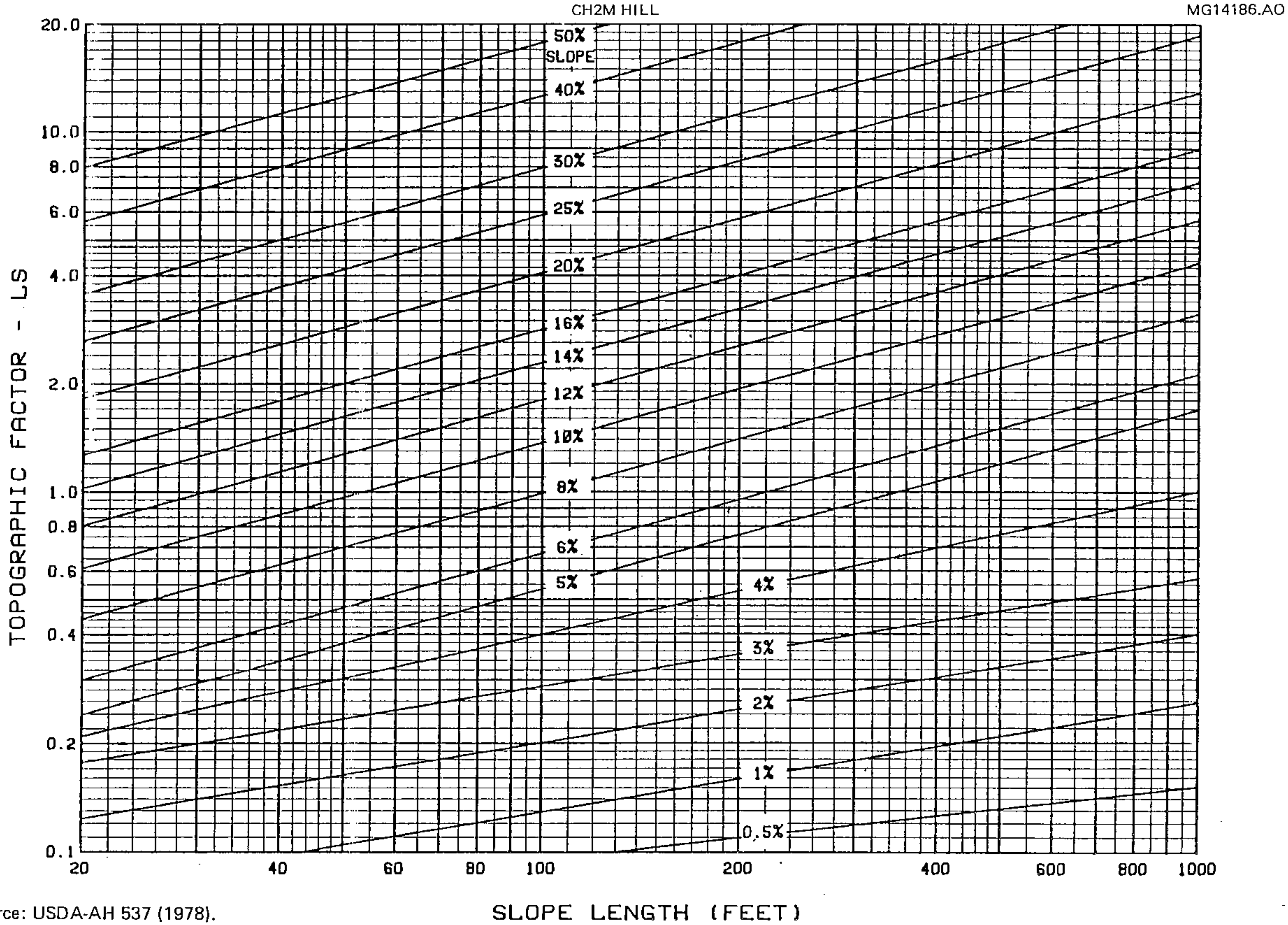
Source: Meyer & Wischmeirer (1969)

FIGURE 6-1. Conceptual diagram of the soil erosion process.



Source: USDA, AH-537 (1978)

FIGURE 6-2. Cumulative percentage of the average annual rainfall erosion index for Montgomery, Alabama.



Source: USDA-AH 537 (1978).

FIGURE 6-3. Length-slope factor chart, for the USLE.

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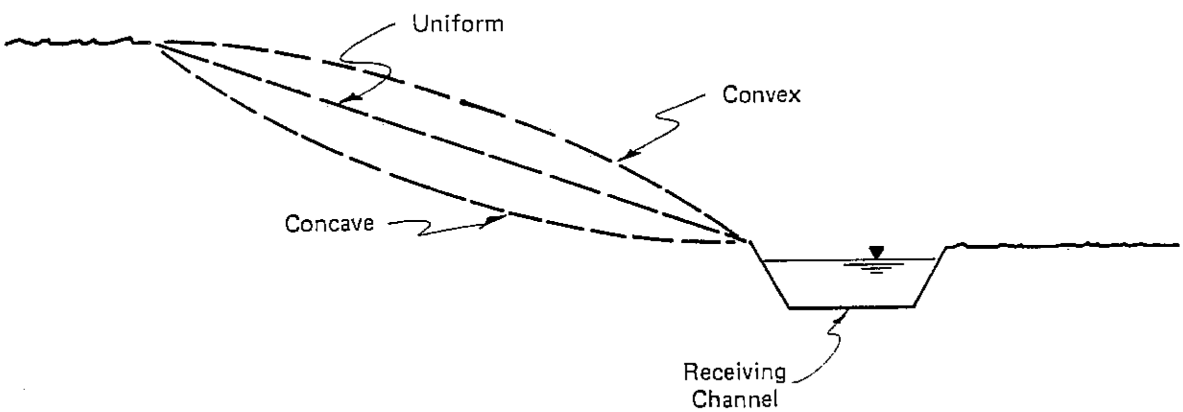

FIGURE 6-4. Typical concave and convex slopes.

SECTION 1.0 INTRODUCTION

The hydraulic design of a stormwater conveyance system requires that water surface or flood elevation profiles be determined for a given design storm return period. Having established the flood elevation profiles for a given design storm, the potential for flood damage can be determined. A stormwater conveyance system should then be designed to mitigate that flood damage to the extent possible with the financial resources available. In addition to flood-damage mitigation, factors concerning erosion and sedimentation within the conveyance system must be considered during the hydraulic design process. Thus there are three primary areas of concern regarding stormwater conveyance systems:

1. Flood-damage mitigation

2. Erosion and sediment control within the conveyance system

3. Economic feasibility

It is the purpose of Chapter 7 to identify desk-top procedures which are applicable in Montgomery for the hydraulic design of open channels, culverts, stormwater inlets, and storm sewers. Hydraulic design considerations of this manual include flood-damage mitigation and the control of erosion and sedimentation within the conveyance system. Economic feasibility considerations are not dealt with quantitatively in this manual.

The procedures identified in Chapter 7 require appropriate hydrologic data as an input to the hydraulic design process. The appropriate hydrologic data should be developed according to the procedures presented in Chapter 5, "Stormwater Runoff Estimation." Generally, an estimate of the peak stormwater flow rate expected to occur during a design storm is all that will be required. However, if channel routing is required then a complete runoff hydrograph should be developed.

SECTION 2.0 OPEN CHANNELS

Open channels can be classified as either natural or man-made. This section identifies hydraulic design procedures for only man-made open channels. These procedures may be applicable to natural channels when a channel can be approximated using an appropriate man-made cross section extending for a distance sufficiently long to establish uniform flow. However, a natural channel is usually not designed, merely analyzed.

The discussion which follows begins with a presentation of open channel flow fundamentals. Specific hydraulic design procedures for lined and unlined open channels are then presented. The section concludes with general considerations for the construction and maintenance of open channels.

2.1 Fundamentals of Open Channel Flow

An open channel is defined as any conduit which conveys a fluid with the liquid surface open to the atmosphere. Thus, flow within a closed conduit is considered to be open channel flow when the closed conduit is only partly full. Common terms which describe flow in open channels include: steady flow, unsteady flow, uniform flow, nonuniform flow, hydraulic grade line, energy grade line, specific energy head, normal depth of flow, critical depth of flow, supercritical flow, and subcritical flow. Working definitions of each of these terms and hydraulic design considerations using Manning's equation are presented below.

2.1.1 Types of Flow. Steady flow occurs in an open channel when the discharge or rate of flow at any location on the channel remains constant with respect to time. The maintenance of steady flow in any reach requires that the rates of inflow and outflow be constant and equal. Conversely, open channel flow is unsteady when the discharge at any location in the channel changes with respect to time. During periods of stormwater runoff, the inflow hydrograph to an open channel is usually unsteady. However, in this manual, open channel flow will be assumed to be steady at the discharge rate for which the channel is being designed (e.g., peak discharge of the inflow hydrograph).

Steady flow can be further classified as uniform or nonuniform. Uniform flow can occur only in a channel of constant cross section, slope, and roughness (i.e., a uniform open channel). If a given channel segment is uniform, the mean velocity and depth of flow will be constant with respect to distance. When the requirements for uniform flow are met, the depth of flow for a given discharge is defined as the normal depth of flow. In practice, minor undulations in the channel bottom or deviations from the average cross section can be ignored as long as the average values are representative of actual channel conditions.

True uniform flow rarely exists in either natural or man-made channels. Any change in the channel cross section, slope, or roughness with distance causes the depths and average velocities to change with distance. Flow that varies in depth and velocity when the discharge is constant (i.e., steady) is defined as steady nonuniform flow.

2.1.2 Energy Considerations. In channel flow problems it is often desirable to consider the energy content of flowing water with respect to the channel bottom. The energy of open channel flow with respect to the channel bottom is defined as the specific energy head and can be determined mathematically as the sum of the water depth and velocity head as given in the following equation:

$$E = d + \frac{v^2}{2g} \qquad (7\text{-}1)$$

where

E = specific energy head, in feet

d = depth of open channel flow, in feet

v = average channel velocity, in ft/sec, defined as the discharge divided by the area of flow, derived from equation 7-2 below

$$Q = Av \qquad (7\text{-}2)$$

therefore

$$v = \frac{Q}{A} \qquad (7\text{-}3)$$

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

Q = discharge, in cfs

A = cross sectional area of open channel, in ft^2

At times it is desirable to consider the total energy head of an open channel, which is the specific energy head plus the elevation of the channel bottom above a selected datum. Since energy input must equal output, the energy equation between two points in a channel reach is written as follows:

$$d_1 + \frac{v_1^{\,2}}{2g} + z_1 = d_2 + \frac{v_2^{\,2}}{2g} + z_2 + h_{loss} \qquad (7\text{-}4)$$

where

d_1 and d_2 = depth of open channel flow at channel sections 1 and 2, respectively, in feet

v_1 and v_2 = average channel velocities at channel sections 1 and 2, respectively, in ft/sec

z_1 and z_2 = channel elevations above an arbitrary datum at channel sections 1 and 2, respectively, in ft

h_{loss} = head or energy loss between channel sections 1 and 2

(g is defined above.)

A longitudinal profile of specific or total energy head elevations is called the <u>energy grade line</u>, or <u>gradient</u>. The longitudinal profile of water surface elevations is called the <u>hydraulic grade line</u>, or <u>gradient</u>. The energy and hydraulic gradients for uniform open channel flow are illustrated on Figure 7-1. For flow to occur in an open channel, the energy gradient must have a negative slope in the direction of flow. A drop in the energy gradient for a given length of channel represents the loss of energy caused by friction. When considered together, the hydraulic and energy gradients reflect not only the loss of energy by friction, but also the conversion between potential and kinetic forms of energy.

Under uniform flow, the energy gradient is parallel to the hydraulic gradient or water surface, which is parallel to the channel bottom (see Figure 7-1). Thus, for uniform flow the slope of the channel bottom becomes an adequate basis for the determination of friction losses. It is also important to note that during uniform flow there are no conversions between kinetic and potential forms of energy. If the flow is accelerating, the hydraulic gradient would be steeper than the energy gradient, while retarding flow would produce an energy gradient which is steeper than the hydraulic gradient.

<u>2.1.3 Critical Flow Conditions</u>. Considering the relative values of potential energy (depth) and kinetic energy (velocity head) in an open channel can greatly aid the hydraulic analysis of open channel flow problems. These analyses are usually performed using a <u>specific energy head curve</u>, which is the relationship between the specific energy head and depth of flow for a given discharge in a given channel that can be placed on various slopes. The specific energy head curve is generally plotted with depth of flow on the ordinate and the specific energy head on the abscissa. However, these axes are reversed in this discussion to facilitate the definition of supercritical and subcritical flow conditions as shown on Figure 7-2.

A typical specific energy head curve is illustrated on Figure 7-2B. The straight diagonal line on this figure represents points where the depth of flow and specific energy head are equal. Since the depth and specific energy are equal on this line the kinetic energy is zero and, therefore, this diagonal line is a plot of the potential energy, or energy due to depth. The ordinate interval between the diagonal line of potential energy and the specific energy curve for the desired discharge is the velocity head or kinetic energy for the depth in question. The lowest point on the specific energy curve represents flow with the minimum content of energy. The depth of flow at this point is known as the critical depth. The general equation for determining the critical depth is as follows:

$$\frac{Q^2}{g} = \frac{A^3}{T} \qquad (7\text{-}5)$$

where

T = top width of water surface, in feet

(Q, g, and A are defined above.)

Critical depth for a given channel can be calculated by trial and error using equation 7-5. However, in practice it may be computed using the equations presented in Table 7-1 for various channel cross sections or design charts which are presented later in this chapter. It is important to note that the determination of critical depth is independent of the channel slope and roughness, since critical depth simply represents a depth for which the specific energy head is a minimum. According to equation 7-5, the magnitude of critical depth depends only on the discharge and the shape of the channel. Thus, for any given size and shape of channel, there is only one critical depth for the given discharge, which is independent of the channel slope or roughness. For a given value of specific energy, the critical depth results in the greatest discharge, or conversely, for a given discharge, the specific energy is a minimum for the critical depth.

The velocity at critical depth is called the critical velocity. An equation for determining the critical velocity in an open channel of any cross section is as follows:

$$v_c = \sqrt{g\ d_m} \qquad (7\text{-}6)$$

where

v_c = critical velocity, in ft/sec

d_m = mean depth of flow, in feet, calculated according to equation 7-7

$$d_m = \frac{A}{T} \qquad (7\text{-}7)$$

(g, A, and T are defined above.)

For conditions of uniform flow, the critical depth or point of minimum specific energy occurs when the channel slope equals the critical slope (i.e., the normal depth of flow in the channel is critical depth). When channel slopes are steeper than the critical slope and uniform flow exists, the specific energy head is higher than the critical value due to higher values of the velocity head or kinetic energy. This characteristic of open channel flow is illustrated by the specific head curve segment to the left of critical depth on Figure 7-2B and is known as supercritical flow. Supercritical flow is characterized by shallow depths and high velocities as shown on Figure 7-2A. If the natural depth of flow in an open channel is supercritical, the depth of flow at any point in the channel may be influenced by an upstream control section. The relationship of supercritical flow to the specific energy curve is shown on Figures 7-2A and 7-2B.

When channel slopes are flatter than the critical slope and uniform flow exists, the specific energy head is higher than the critical value due to higher values of the normal depth of flow or potential energy. This characteristic of open channel flow is illustrated by the specific head curve segment to the right of critical depth on Figure 7-2B and is known as subcritical flow. Subcritical flow is the most common type of flow to be expected in Montgomery and is characterized by relatively large depths with low velocities as shown on Figure 7-2C. If the natural depth of flow in an open channel is subcritical, the depth of flow at any point in the channel may be influenced by a downstream control section. The relationship of subcritical flow to the specific energy curve is shown on Figures 7-2C and 7-2B.

Several points concerning Figure 7-2 should be noted. First, at depths of flow near the critical depth for any discharge, a minor change in specific energy will cause a much greater change in depth. Second, the velocity head for any discharge in the subcritical portion of the specific energy curve is relatively small when compared to specific energy. For this subcritical portion of the specific energy curve, changes in depth of flow are approximately equal to changes in specific energy. Finally, the velocity head for any discharge in the supercritical portion of the specific energy curve increases rapidly as depth decreases. For this supercritical portion of the specific energy curve, changes in depth are associated with much greater changes in specific energy.

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

1. If the velocity head is less than one-half the mean depth of flow (equation 7-7) the flow is subcritical.

2. If the velocity head is equal to one-half the mean depth of flow (equation 7-7) the flow is critical.

3. If the velocity head is greater than one-half the mean depth of flow (equation 7-7) 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.

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

2.1.4 Manning's Equation. The hydraulic design of an open channel is generally performed using Manning's equation to compute the average velocity, given the depth of flow in a uniform channel cross section. Given the velocity, the discharge is calculated as the product of velocity and cross sectional area (see equation 7-2). Manning's equation is an empirical equation in which the values of constants and exponents have been derived from experimental data. According to Manning's equation the mean velocity of flow is a function of the channel roughness, the hydraulic radius (i.e., shape), and slope of the energy gradient. As noted previously, for uniform flow the slope of the energy gradient is equal to the channel bottom slope. Manning's equation is expresed mathematically as follows:

$$v = \frac{1.49}{n} R^{2/3} S^{1/2} \qquad (7\text{-}8)$$

or

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \qquad (7\text{-}9)$$

where

v = average channel velocity, in ft/sec

n = Manning's roughness coefficient

R = hydraulic radius of the channel, in feet, calculated using equation 7-10

S = slope of the energy gradient in ft/ft

A detailed discussion of procedures for solving Manning's equation is presented in Subsection 2.2.2.

2.1.4.1 Manning's Roughness Coefficient--Manning's n values for conveyance system design in Montgomery are presented in Table 7-2. For concrete open channels with troweled bottoms and broom-finished side slopes, a Manning's n value of 0.015 is required. For natural or excavated open channels, the value of Manning's n should be determined using Cowan's equation as presented in Table 7-2, and the coefficients presented in Table 7-3.

Typical ranges of Manning's roughness coefficient for various conveyance systems are presented in Tables 7-4 to 7-9. Values of Manning's n for closed conduits are presented in Table 7-4, values for nonvegetative lining of open channels are presented in Table 7-5, values for street and expressway gutters are presented in Table 7-6, values for excavated channels (unlined) are presented in Table 7-7, and for natural streams are presented in Table 7-8. Manning's n values for vegetated channels are determined for various retardance classes using Table 7-9 and Figure 7-3, while n values of riprap channels are determined using equation 7-14.

Factors to consider when selecting the value of Manning's n include the following:

1. The physical roughness of the bottom and sides of the channel should be taken into account. Soils made up of fine particles on smooth, uniform surfaces result in relatively low values of n. Coarse materials such as gravel or boulders and pronounced surface irregularity cause higher values of n.

2. The value of n will be affected by height, density, and type of vegetation. Consideration should be given to density and distribution of the vegetation along the reach and the wetted perimeter; the degree to which the vegetation occupies or blocks the cross section of flow at different depths; and the degree to which the vegetation may be bent or "shingled" by flows of different depths.

3. Channel shape variations such as abrupt changes in channel cross sections or alternating small and large cross sections will require somewhat larger n values than normal. These variations in channel cross section become particularly important if they cause the flow to meander from side to side.

4. A significant increase in the value of n is possible if severe meandering occurs in the alignment of a channel. Meandering becomes particularly important when frequent changes in the direction of curvature occur with relatively small radii of curvature.

5. Active channel erosion or sedimentation will tend to increase the value of n since these processes may cause variations in the shape of a channel. It is also important to consider the potential for future erosion or sedimentation in the channel.

6. Obstructions such as log jams or deposits of debris will increase the value of n. The level of this increase will depend on the number, type, and size of obstructions.

The value of n, in either natural or man-made earthen channels, varies with the season and from year to year; it is not a fixed value. Each year n increases in the spring and summer, as vegetation grows and foliage develops, and diminishes in the fall as the dormant season approaches. The annual growth of vegetation, uneven accumulation of sediment in the channel, lodgment of debris, erosion and sloughing of banks, and other factors all tend to increase the value of n from year to year until the hydraulic efficiency of the channel is improved by clearing, clean-out, or reconstruction.

All of these factors should be studied and evaluated with respect to kind of channel, degree of maintenance, seasonal requirements, and other considerations as a basis for making a determination of an appropriate design n value. As a general rule, conditions tending to induce turbulence will increase retardance, and those tending to reduce turbulence will reduce retardance.

2.1.4.2 Hydraulic Radius--The hydraulic radius, R, is a shape factor that depends only on the dimensions of the channel cross section and depth of flow. It is calculated by dividing the channel cross sectional area by the channel wetted perimeter. This calculation is expressed mathematically as follows:

$$R = \frac{A}{P} \qquad (7\text{-}10)$$

where

R = hydraulic radius of the channel, in feet

P = wetted perimeter of the channel cross section, in feet,

(A is defined above.)

Mathematical expressions for calculating the area, wetted perimeter, hydraulic radius, and channel top width for selected open channel cross sections are presented in

Table 7-10. These cross sections include trapezoidal, rectangular, triangular, parabolic, and circular. Channel cross sections which are irregular, such as those having 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 for the overbank channel. This same procedure of subdividing the channel may be used when different parts of the channel cross section have different roughness coefficients. When computing the hydraulic radius of the subsections, the water depth common to two adjacent subsections is not counted as wetted perimeter.

2.1.5 Nonuniform Flow Considerations. In the vicinity of changes in the channel section or slope which will result in nonuniform flow profiles, the direct solution of Manning's equation is not possible, since the energy gradient no longer equals the channel slope. Three typical examples of nonuniform flow are illustrated on Figures 7-4 to 7-6. The following paragraphs describe these nonuniform flow profiles and briefly explain how the total head line is used for approximating these water surface profiles in a qualitative manner.

A channel on a mild slope (subcritical) discharging into a reservoir or pool is illustrated on Figure 7-4. The vertical scale is exaggerated to illustrate the case more clearly.

Cross section 1 is located at the end of uniform channel flow in the channel, and cross section 2 is located at the beginning of the pool. The depth of flow between sections 1 and 2 is changing, and the flow is nonuniform. The water surface profile between the sections is known as a "backwater curve" and is characteristically very long. The computation of backwater curves in a quantitative manner can be quite complex. If a detailed analysis of backwater curves is required, the user should consider applying the HEC 2 computer program developed by the U.S. Army Corps of Engineers. A brief summary of computer programs for the hydraulic design of stormwater conveyance systems is presented in Appendix B. In addition, textbooks by Chow (1959), Henderson (1966), or Streeter (1971) and handbooks by Brater and King (1976) or the SCS (1956) may be useful.

A channel in which the slope changes from subcritical (mild) to supercritical (steep) is illustrated on Figure 7-5. The flow profile passes through critical depth near the break in slope (section 1). This is true whether the upstream slope is mild, as in the sketch, or whether the water above section 1 is ponded, as would be the case if section 1 were the crest of a spillway of a dam. If, at section 2, the total head were computed, assuming normal depth on the steep slope, it would plot (point a on Figure 7-5) above the elevation of total head at section 1. This is physically impossible, because the total head line must slope downward in the direction of flow. The actual total head line will take the position shown and have a slope approximately equal to S_0 at section 1 and approaching slope S_0 farther downstream. The drop in the total head line h_{loss} between sections 1 and 2 represents the loss in energy due to friction. At section 2 the actual depth, d_2, is greater than d_n (normal depth) because sufficient acceleration has not occurred, and the assumption of normal depth at this point would clearly be in error. As section 2 is moved downstream, so that total head for normal depth drops below the pool elevation above section 1, the actual depth quickly approaches the normal depth for the steep channel. This type of water surface curve (section 1 to section 2) is characteristically much shorter than the backwater curve discussed in the previous paragraph.

Another common type of nonuniform flow is the drawdown curve to critical depth which occurs upstream from section 1 (Figure 7-5) where the water surface passes through critical depth. The depth gradually increases upstream from critical depth to normal depth, provided that the channel remains uniform over a sufficient distance. The length of the drawdown curve is much longer than the curve from critical depth to normal depth in the steep channel.

A <u>hydraulic jump</u> occurs when a steep (supercritical) channel discharges into a reservoir or pool. This special case condition is illustrated on Figure 7-6. A hydraulic jump makes a dynamic transition from the supercritical flow in the steep channel to the subcritical flow in the pool. This situation differs from that shown on Figure 7-4 in that the flow approaching the pool on Figure 7-6 is supercritical and the total head in the approach channel is large relative to the pool depth. In general, supercritical flow can be changed to subcritical flow only by passing through a hydraulic jump. The violent turbulence in the jump dissipates energy rapidly, causing a sharp drop in the total head line between the supercritical and subcritical states of flow. A jump will occur whenever the ratio of the depth, d_1, in the approach channel to the depth, d_2, in the downstream channel reaches a specific value. Note on Figure 7-6 that normal depth in the approach channel persists well beyond the point where the projected pool level would intersect the water surface of the channel at normal depth. Normal depth can be assumed to exist on the steep slope upstream from section 1, which is located about at the toe of the jump. Detailed information related to the quantitative evaluation of hydraulic jump conditions in an open channel is not presented in this manual. If the user requires such detailed information, textbooks by Chow (1959), Henderson (1966), or Streeter (1971) should be consulted. In addition, handbooks by King and Brater (1976) or the SCS (1956) may be useful.

2.2 Hydraulic Design of Open Channels

The primary consideration in the hydraulic design of any open channel is to provide the most economical cross section which will carry the desired design discharge without overtopping. The designer's general aim with respect to hydraulics is, therefore, to minimize the channel cross sectional area given the design discharge, channel slope, and Manning's roughness coefficient for the proposed channel bottom (i.e., lined or unlined). The type of proposed channel bottom may place an additional constraint on the hydraulic design process if the maximum permissible velocity for the channel bottom in question is less than the average velocity produced in the minimum cross sectional area. The hydraulic design process for any open channel can thus be visualized as a two-step process. First, an optimum channel cross section is determined. Second, this optimum open channel configuration is adjusted if necessary such that the average channel velocity is less than the maximum permissible velocity for the channel material in question.

Permissible velocities for channels lined with uniform stands of well-maintained grass cover are presented in Table 7-12, and permissible velocities for channels with erodible linings are presented in Table 7-13. A concrete-lined open channel should be designed to provide a minimum velocity of 3 fps at normal depth and, if possible, supercritical flow conditions should be avoided.

2.2.1 Best Hydraulic Section. The best hydraulic section of an open channel can be determined mathematically by considering the continuity equation of open channel flow (equation 7-2) and the Manning's formula (equation 7-8). According to the continuity equation, if the cross sectional area is to be a minimum, the velocity must be a maximum for any given cross sectional area. According to Manning's formula the velocity is a maximum for a given cross section when the hydraulic radius is a maximum. The hydraulic radius is a maximum when the wetted perimeter is minimized for a given cross sectional area (see equation 7-10). It can be shown mathematically that the hydraulic cross section which maximizes the velocity and thus minimizes the area required to convey a given discharge is a semicircle (see Streeter, 1971, or Chow 1959). Therefore, it can also be shown that the best trapezoidal hydraulic section is one which approximates a semicircle (see Figure 7-7a). For the special case in which the trapezoid is a rectangle, the best shape is that for which the width is twice the depth (see Figure 7-7b).

In practice, the best hydraulic section of an open channel may be altered from the mathematically ideal cross section to account for the following factors:

1. The average velocity of the best hydraulic section should not exceed the maximum permissible velocity for the channel bottom in question.

2. The best hydraulic section may not produce the minimum total excavation if a significant overburden must be removed.

3. The proportions of an open channel may vary widely without significantly changing the required cross section.

4. The cost of excavation is not solely dependent on the amount of material removed. Considerations such as the ease of access and disposal may be more important than the volume of material excavated.

2.2.2 Solutions to Manning's Equation. Manning's equation can be solved explicitly if the velocity, discharge, or slope of the energy gradient are desired. However, if the channel cross section or depth of uniform flow are desired, a trial and error solution process is required. For the typical open channel design problem, a trial and error solution process is required since the channel shape parameters are unknown. Given the design discharge and allowable channel slope, the variables under the control of the designer are the channel width, side slopes, and to some extent, the depth of flow. The maximum permissible velocity and the value of Manning's roughness coefficient are properties of the channel lining under consideration.

2.2.2.1 Explicit Solution--Given the appropriate hydraulic radius and roughness coefficient for an open channel, the velocity or slope can be determined explicitly using Manning's equation given either the velocity or the slope. Given the velocity, the discharge is calculated according to equation 7-2. Explicit solutions to Manning's equation can be obtained from one of several nomographs presented in this manual. A general nomograph solution to Manning's equation is presented on Figure 5-11, and four very detailed nomographs are presented as Figures 7-8 to 7-11.

2.2.2.2 Trial and Error Solution to Manning's Equation--As noted previously, a trial and error procedure is generally required to solve Manning's equation for the typical open channel design problem. This trial and error process can often be simplified by using published design charts for common channel cross sections. Published open channel design charts are presented in Subsection 2.2.3. When a published design chart is not available for a given channel cross section and a small number of design calculations will be required for that cross section, a manual trial and error solution process may be suitable. However, when numerous trial and error design calculations are required, design charts can be developed for any desired channel cross section according to the procedures described in Hydraulic Design Series No. 3, developed by the U.S. Department of Transportation (1961).

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

$$AR^{2/3} = \frac{Qn}{1.49\ S^{1/2}} \qquad (7\text{-}11)$$

where all variables are as defined above.

To determine the normal depth of flow in a channel by the trial and error process, trial values of depth are used to determine values of A, P, and R for the given channel cross section. Trial values of $AR^{2/3}$ are computed until the equality of equation 7-11 is satisfied such that the design flow is conveyed within the banks of the desired channel cross section.

If a large number of trial and error calculations are required, it may be convenient to introduce the concept of channel conveyance. The conveyance of a channel is defined by Henderson (1966) as follows:

$$K = \frac{1.49\ AR^{2/3}}{n} \qquad (7\text{-}12)$$

and

$$Q = KS^{1/2} \qquad (7\text{-}13)$$

where

K = conveyance of a given open channel

(Q, S, A, R, and n are as defined above.)

A table or plot of K or the product of Kn can be developed as a function of normal depth of flow for any given channel section. The tables or curves can then be used to determine the normal depth of flow for any given values of Q, S, and n.

Additional tabular design aids for determining the normal depth of uniform flow in rectangular or trapezoidal channels are presented by Brater and King (1976). Although these aids are not dealt with specifically in this design manual, they are considered accceptable engineering practice in Montgomery.

2.2.3 Open Channel Design Charts

2.2.3.1 Nonvegetative Lining--Hydraulic design charts for typical channel cross sections lined with nonvegetative material are published by the U.S. DOT Federal Highway Administration (1961). The depth of flow and velocity of uniform flow may be read directly from these charts for a given channel slope and cross section when n is 0.015 for rectangular channels or 0.03 for trapezoidal channels. Auxiliary scales on each chart allow solutions for other values of n.

A separate hydraulic design chart is provided for each rectangular cross section from 2 feet to 10 feet of width at 1-foot intervals and from 10 feet to 20 feet of width at 2-foot intervals. These rectangular hydraulic design charts for nonvegetative channel lining are presented on Figures 7-12 to 7-25.

Trapezoidal design charts are provided at 1-foot intervals for bottom widths from 2 feet to 10 feet on Figures 7-26 to 7-34 and at 2-foot intervals for bottom widths from 10 feet to 20 feet on Figures 7-35 to 7-39.

These trapezoidal design charts for nonvegetative channel lining were developed for side slopes of 2:1 (horizontal to vertical). Since the City Engineering Department of Montgomery requires that the side slopes of concrete-lined open channels be equal to or less than 1.5:1, they should be adequate for most design problems.

The nonvegetative hydraulic design charts provide a graphical solution of Manning's equation for flow in open channels of uniform slope, cross section, and roughness. It is important to note that these design charts are not applicable if the open channel flow is affected by backwater or if the channel is too short for the establishment of uniform flow in the channel. A rounding of trapezoidal cross sections such that the intersection of the side slopes with the bottom of the channel does not significantly affect the usefulness of these charts.

The charts for rectangular and trapezoidal cross section channels are similar in design and their method of use. The abscissa scale is discharge, in cfs, and the ordinate scale is velocity, in feet per second. Both of these scales are logarithmic. Superimposed on the logarithmic grid are steeply inclined lines representing depth, in feet, and slightly inclined lines representing channel slope in feet per foot.

A heavy dashed line on each chart shows the position of critical flow. Auxiliary abscissa and ordinate scales are provided for use with values of n other than those used in preparing the chart.

A chart for triangular cross sections (presented in nomograph form as Figure 7-40) may be used for street sections with a vertical curb face. The equation given on the chart does not take into account the resistance of the curb face, but this resistance is negligible in practice, provided the width of flow is at least 10 times the depth of the curb face (i.e., if $Z > 10$). The equation gives a discharge about 19 percent greater than will be obtained by the common procedure of computing discharge from the hydraulic radius of the entire section. The latter procedure is not recommended for shallow flow with continuously varying depth. The nomograph may also be used for shallow v-shaped sections. Appropriate values of Manning's n for street and expressway gutters can be determined from Table 7-6.

Channels lined with nonvegetative material should be designed to convey only subcritical flow. In practice, the normal depth of flow at the design discharge should be at least 10 percent above the critical depth.

The following steps summarize the hydraulic design process using Figures 7-12 to 7-39 for open channels lined with nonvegetative material:

1. Select the appropriate hydraulic design chart from Figures 7-12 to 7-39 for the channel cross section in question. If a design chart is not published for the channel cross section in question, one can be developed according to the procedure presented in Hydraulic Design Series No. 3 published by the U.S. DOT (1961).

2. Enter the design chart selected in Step 1 on the abscissa scale at the appropriate design discharge. If the Manning's n in question is different from the n value printed on the design chart, the design discharge is multiplied by the n value in question and the Qn scale is entered on the abscissa instead of the Q scale. From the appropriate point on the abscissa one proceeds vertically to intersect the appropriate slightly inclined slope line in feet per foot. At this intersection the normal depth of flow can be obtained directly from the steeply inclined lines representing depth, in feet.

3. Proceed horizontially from the point of intersection determined in Step 2 to read the average velocity of flow in feet per second from the ordinate scale. If the Qn scale was used in Step 2 to find the

point of intersection, then the Vn scale is used to determine the average velocity. Having determined the value of Vn, the average velocity is found by dividing Vn by the n value in question.

4. To determine the critical velocity and depth of flow, enter the chart at the appropriate discharge on the abscissa; the intersection of a vertical line through that discharge on the dashed line identifies the appropriate critical velocity and depth of flow for the channel in question. The critical velocity and depth of flow are independent of the n value in question. Their values can be obtained directly from the dashed line on each design chart.

5. Read the critical channel slope directly from the dashed line if the n value in question is the same as that printed on the design chart. If the critical slope for an n value is different from that printed on the chart a different procedure is required. Start at the intersection of the critical curve (dashed line) and a vertical line through the design discharge. At this intersection the critical depth can be found directly from the appropriate steeply inclined normal depth of flow curve. This normal depth of flow curve is then followed until it intersects a vertical line through the appropriate value of Qn on the abscissa. At this intersection the critical slope is read directly from the appropriate slightly inclined slope line.

6. Compare the average velocity from Step 3 and the critical velocity from Step 4 to ensure that flow is subcritical. If flow is supercritical, Steps 1 through 5 should be repeated to provide a channel configuration which provides subcritical flow conditions.

7. Multiply the critical depth from Step 4 by 0.10 and add this value to the critical depth. If the normal depth of flow, d_n, determined by Step 2 is greater than the critical depth plus 0.10 times the critical depth, the channel configuration is satisfactory.

8. Multiply the normal depth by 0.20 and add this amount to the normal depth as a freeboard safety factor. A minimum of 0.5 feet should be added as a freeboard.

Application of the rectangular and trapezoidal design charts for nonvegetative channel linings is illustrated by example 7.3. Instructions with example problems for the triangular design chart are given on the design chart itself.

As noted previously, if the desired channel cross section is not considered in the published hydraulic design charts, a trial and error solution to Manning's equation can be obtained manually. However, if numerous design calculations are likely to be performed for an unpublished channel cross section, a design chart can be developed by the designer for the desired channel cross section. A detailed description of the procedure for constructing hydraulic design charts for any channel cross section with a nonvegetative lining is presented in Hydraulic Design Series No. 3 published by the U.S. DOT (1961).

<u>2.2.3.2 Vegetative Lining</u>--Hydraulic design charts for typical channel cross sections lined with vegetation are also published by the U.S. DOT Federal Highway Administration (1961). These hydraulic design charts are similar in appearance and use to those for trapezoidal cross sections described previously. However, the construction of hydraulic design charts for vegetative channel lining is much more difficult because the roughness coefficient, n, varies with the type and height of grass and with the velocity and depth of flow.

The effect of velocity and depth of flow on n for vegetated channels may be evaluated by the product of velocity and hydraulic radius, vR. The variation of Manning's n with the retardance and the product, vR, is presented for observed data on Figure 7-3. Four retardance curves labeled A, B, C, and D are shown on Figure 7-3. Retardance classes vary with the height and condition of the grass used to line an open channel. The classification of vegetal covers as to the degree of retardance is presented in Table 7-9. The retardance classification depends primarily upon the type of grass, planting conditions, and maintenance practices. A guide for the selection of vegetal retardance is presented in Table 7-11. In practice, retardance classes A and B do not apply to grasses and maintenance conditions required for urban open channels. Therefore, the published design charts presented in this chapter relate only to retardance classes C and D. Grasses commonly used to line urban open channels include bermudagrass, Kentucky bluegrass, orchardgrass, redtop, Italian ryegrass, and buffalo grass.

Open channels lined with vegetation are typically designed in two steps. First, a channel cross section is determined which has the capacity to carry the design discharge on the slope which is available. Second, the average velocity for the channel cross section required to carry the design discharge is checked to ensure that the grass lining is not likely to be eroded. Since the actual retardance of a grass-lined channel at any given time cannot be determined by the designer, the channel capacity should be determined for the highest vegetal retardance expected, and the average velocity should be determined for the lowest vegetal retardance expected. These retardance classifications represent worst case conditions for each design step and are recommended as good engineering practice. Therefore, the designer should compute the channel capacity using retardance C (high retardance) and the average velocity using retardance D (low retardance).

The hydraulic design charts for typical channel cross sections lined with vegetation are presented on Figures 7-41 to 7-45. These hydraulic design charts present a graphical solution of Manning's equation for flow in open channels of uniform slope, cross section, and roughness. It is important to note that these design charts are not applicable if the open channel flow is affected by backwater or if the channel is too short for uniform flow to become established in the channel. A rounding of trapezoidal cross sections such that the intersection of the side slopes with the channel bottom does not significantly affect the usefulness of these charts.

The City Engineering Department of Montgomery requires that the side slopes of grass-lined channels be equal to or less than 3:1. The hydraulic design charts for grass-lined channels were developed for trapezoidal channel cross sections with a bottom width of 4 feet and side slopes of 2:1, 4:1, 6:1, and 8:1, respectively, and are presented as Figures 7-41 to 7-44. A single design chart is developed for a triangular cross section with a side slope of 10:1 (Figure 7-45). Each of these design charts is plotted with discharge, in cfs, as the abscissa, and slope, in feet per foot, as the ordinate. Both scales are logarithmic. Superimposed on the logarithmic grid are lines for velocity, in feet per second, and lines for depth, in feet. A dashed line shows the position of critical flow.

A special design requirement for grass-lined open channels is the provision for a low flow pilot channel to eliminate standing water. Ideally, this pilot channel should be sized using observed base flow characteristics of a given watershed. However, when these data are not available, a rule of thumb is to size the pilot channel for 0.2 to 0.5 cfs per square mile of drainage area. If above ground, the pilot channel should be lined with concrete. If below ground, the collection pipe should be placed in a bed of ballast rock lined with a filter membrane to prevent sand particles from clogging pore spaces. Design details for underground drainage systems can be obtained from the U.S. DOT (1980).

The following steps summarize the hydraulic design process using Figures 7-41 to 7-45 for open channels lined with vegetative material:

1. Select the channel cross section to be used and locate the appropriate chart from Figures 7-41 to 7-45. If a design chart is not published for the channel cross section in question and numerous design calculations for that channel section are expected, a chart can be developed according to the procedure presented in Hydraulic Design Series No. 3 published by the U.S. DOT (1961).

2. Enter the chart for retardance C (i.e., highest resistance) with the design discharge value on the abscissa; proceed vertically from that discharge to the value of the available slope on the ordinate scale. At this intersection, read the normal velocity and normal depth, and note the position of the critical curve. If this intersection point is below the critical curve, the flow is subcritical and the channel cross section has adequate capacity for the design flow. If the intersection is above the critical line, the flow is supercritical and a larger channel cross section should be selected, to avoid supercritical flow.

3. Enter the chart for retardance D (i.e., lowest resistance) with the design discharge and slope and determine average velocity as in step 2. If the computed velocity is less than the maximum permissible for the type of grass, channel slope, and soil (see Table 7-12), design is acceptable. If the maximum permissible velocity is exceeded, a larger channel cross section should be selected until the computed velocity is less than the maximum permissible velocity.

4. Size a concrete-lined pilot channel or underground infiltration pipe to carry base flow. If observed base flow characteristics for the watershed are not available, size the pilot channel for 0.2 to 0.5 cfs per square mile of drainage area.

5. Multiply the normal depth of flow by 0.20 and add this amount to the normal depth as a freeboard safety factor. A minimum of 0.5 foot should be added as a freeboard.

An example problem which illustrates how to use these hydraulic design charts for vegetated open channels is presented in example 7.4. Without the aid of these design charts a trial and error solution process is required to solve Manning's equation. If numerous design calculations are likely to be performed for unpublished channel cross sections and retardance classes, additional design charts can be developed for the channel conditions in question. A detailed description of this procedure for vegetated channel linings is presented in Hydraulic Design Series No. 3 published by the U.S. DOT (1961).

2.2.3.3 Riprap Hydraulic Design--The hydraulic design of riprap-lined channels is similar in concept to the hydraulic design of grass-lined channels, because the value of Manning's n increases with larger sizes of stone, just as it increases with higher lengths and densities of vegetative cover. A direct solution to this problem is not practical unless design charts similar to those presented for vegetal retardance classes are developed for a range of stone sizes. In practice, a graphical trial and error procedure has been developed which is adequate for most open channel design problems. This procedure supplements the hydraulic design charts for solving Manning's equation when a nonvegetal lining is used (Figures 7-41 to 7-45) with an equation for estimating n based on the median stone size and two hydraulic charts for determining a stable stone size for the channel conditions in question. The steps in this procedure are described as follows:

1. Estimate a trial size of stone. This can be accomplished by using Figure 7-46. Assuming the average velocity against the stone channel side slopes, a trial value for the stone size can be estimated.

2. Calculate a trial value of Manning's n based on the trial stone size determined in Step 1. A trial value of Manning's n is calculated from the following equation:

$$n = \frac{(12k)^{1/6}}{44.4} \qquad (7\text{-}14)$$

where

n = Manning's roughness coefficient for a given size of stone

k = median stone diameter, in feet

Equation 7-14 applies to a stone lining on both the side slopes and bottom of a channel. When only the sides of a channel are lined, the value of n might require weighting when the bottom width exceeds four times the depth of flow.

3. Given the design discharge, allowable channel slope, and Manning's n, a depth of flow (d) and average velocity in the channel can be estimated for the desired channel cross section using the appropriate hydraulic design chart presented in Figures 7-12 to 7-39.

4. Divide the trial stone size (k) from Step 1 by the depth of flow (d) estimated in Step 3 to obtain the k/d ratio.

5. Enter the ordinate of Figure 7-47 with the k/d ratio from Step 4 and read a value of the v_s/v ratio from the abscissa,

   where

   v_s = average velocity against the stone, in feet per second

   v = average velocity in the channel, in feet per second

6. Multiply the average velocity in the channel estimated in Step 3 by the v_s/v ratio determined in Step 5 to obtain an estimate of v_s, the average velocity against the stone.

7. Enter the ordinate of Figure 7-46 with the value of v_s estimated in Step 6 and read a value of the stone size from the abscissa for the appropriate channel side slope.

8. If the stone size estimated in Step 1 is smaller or larger than the stone size required according to Step 7, a new stone size is selected and Steps 2 through 7 are repeated until the estimated size agrees with the required size.

This riprap hydraulic design procedure is illustrated by example 7.5.

<u>2.2.3.4 Unlined Channels</u>--The hydraulic design of unlined channels can be performed using any appropriate procedure for determining the velocity and depth of flow, given the design discharge, channel slope, and Manning's roughness coefficient for the erodible channel lining. The channel must be sized such that it has adequate capacity to handle the design discharge without the average velocity of that channel exceeding the maximum permissible velocity for the erodible channel lining in question. Permissible velocities for channels with erodible linings based on uniform flow in continuously wet, aged channels are presented in Table 7-11.

In addition to maximum permissible velocities, appropriate Manning's roughness coefficients are reported in Table 7-5 for the selected erodible linings.

2.2.3.5 Open Channel Freeboards--A final consideration for the hydraulic design of any open channel should be the addition of adequate freeboard to ensure containment of the design discharge. This freeboard is a safety factor to protect against underestimates of flow or the roughness coefficient and to provide for containment of wave action. Generally, a freeboard of around 20 percent of the depth with a minimum of 0.5 feet should be added to the computed channel depth.

2.2.4 Construction Considerations for Open Channels. Details concerning the methods for constructing open channels are beyond the scope of this manual. However, a brief discussion to aid the designer in locating more detailed information is presented in this subsection.

The City Engineering Department of Montgomery requires that superelevation be provided on all channel bottoms along an open channel curve. Channel bottoms at curve locations must be constructed with a minimum 1/4-inch-per-foot cross slope (bottom draining toward the outside of the curve). The purpose of this cross slope is to prevent sediment deposits from accumulating along the inside edge of the channel curve. As shown on Figure 7-48, the channel bottom flow line elevation established by the channel profile should follow the outside edge of the channel bottom. Thus, the superelevation is obtained by rotating the channel bottom upward about its outside edge.

General information related to the construction of open channels concerns provisions for adequate erosion control and supervision during construction. Supervision during construction involves ensuring not only that the construction complies with the plans and specifications, but that unforeseen problems can be solved in the field. Sufficient hydraulic design data from the original calculations should be shown on the construction plans such that corrections can be made in the field. Some guidelines for placing hydraulic design information on project plans are provided by a U.S. Bureau of Public Roads (1959) report.

Provisions for erosion control should be based on the maximum permissible velocity for the channel material in question. If the mean velocity at the design flow exceeds the permissible velocity for the soil type in question, the channel should be protected from erosion. Design details for erosion control systems are presented in Chapter 9 of this manual. Information related to the hydraulic design of nonvegetative-

(e.g., concrete-), vegetative-, and riprap-lined channels was provided previously in this section. Concrete-lined channels are generally constructed for the following reasons:

1. To provide erosion control on steep slopes.

2. To provide a nonsilting velocity on very flat slopes.

3. To ensure efficient water removal from ponded areas.

4. To reduce the channel dimensions required to handle a given design discharge.

Additional information related to the construction of open channels can be found in the following reports:

1. U.S. Bureau of Reclamation, 1963. Linings for Irrigation Canals, U.S. Government Printing Office, Washington, D.C.

2. U.S. Bureau of Reclamation, 1960. Design of Small Dams, U.S. Government Printing Office, Washington, D.C.

3. The Asphalt Institute, 1961. Asphalt in Hydraulic Structures, Manual Series No. 12, College Park, Maryland.

4. U.S. Dept. of Commerce, 1965. Design of Roadside Drainage Channels, Hydraulic Design Series No. 4, Bureau of Public Roads, U.S. Government Printing Office, Washington, D.C.

5. U.S. Dept. of Agriculture, 1947. Handbook of Channel Design for Soil and Water Conservation, SCS-TP-61, Soil Conservation Service, Washington, D.C. (Revised June 1954).

6. Lane, E. W., 1955. "Design of Stable Channels," Trans. ASCE, Vol. 120, page 1248.

7. Murphy, T. E. and Grace, J. L., 1963. Riprap Requirements for Overflow Embankments, Highway Research Board Record, No. 30, Washington, D.C.

8. U.S. Dept. of Commerce, 1967. Use of Riprap for Bank Protection, Hydraulic Engineering Circular No. 11, Bureau of Public Roads, U.S. Government Printing Office, Washington, D.C.

In general, concrete-lined open channels should not be designed for supercritical flow conditions. However, if site conditions require supercritical flow, special design factors must be considered for channel construction to be stable. Design considerations for supercritical flow conditions can be found in References 2 and 4 above.

In saturated soils, empty channels with rigid linings (e.g., concrete) may float or break up due to the buoyancy of displaced water. The total upward force on an empty channel is equal to the weight of the displaced water. This uplift pressure is resisted by the total weight of the lining. If the weight of the lining is less than the uplift pressure the channel is unstable. The lining should then be increased in thickness to add additional weight or, if the flow is subcritical, weep holes may be placed at intervals in the channel bottom to relieve the upward pressure on the channel. The City Engineering Department requires that 2-inch weep holes be located on both side slopes at 10 feet on center and at the 1/4 normal depth mark. To allow adequate seepage, a minimum of 1 cubic foot of crushed gravel is required behind the weep holes. When flow is supercritical, subdrainage should be used rather than weep holes.

Grass lining can best be attained by sodding, either alone or or in combination with seeding. Sod might be used in the channel bottom with the channel side slopes being seeded. Seeding should be protected by mulch, temporary cover grasses, and jute or fiber netting. A strip of sod should be placed along the edges of all concrete, riprap, and similar linings to prevent erosion due to overland flow entering the channel. Without adequate erosion control, serious undermining of the channel lining can result.

All stone used for channel linings or bank protection should be hard, dense, and durable. Most of the igneous and metamorphic rocks, many of the limestones, and some of the sandstones make excellent linings. Shale is not suitable, nor are limestones or sandstones that have shale seams. Quarried stones, which are angular in shape, are preferred to rounder boulders or cobbles. If rounded stones are used, the size of stone determined by the methods presented previously in this section should be increased, particularly if the rounded stones are relatively uniform in size. Further details concerning the construction of riprap-lined channels can be found in References 2, 4, and 7 listed above.

<u>2.2.5 Maintenance Considerations for Open Channels</u>. Open channels rapidly lose their effectiveness unless they are adequately maintained. Maintenance includes repairing erosion damage, mowing grass, and removing any sediment or debris deposited in the channel. If the channel cross section contains brush, sediment, or debris the channel cannot carry

the design flow. In addition, the sediment and debris may kill the vegetative lining of a channel, resulting in possible erosion damage to the channel cross section during large storm events. The maintenance of vegetative linings may include the repeated application of fertilizer and reseeding or resodding to restore damaged vegetative cover before serious erosion occurs.

2.3 Example Problems

Example 7-1. Trial and Error Solution to Manning's Equation

Find the normal depth of flow and best hydraulic section for a rectangular open channel lined with concrete, if it must carry 200 cfs on a slope of 0.006 ft/ft. Determine the type of flow at normal depth.

1. From Table 7-2, n = 0.015; therefore:

$$\frac{Qn}{1.49\ S^{\frac{1}{2}}} = \frac{(200)(0.015)}{1.49(0.006)^{\frac{1}{2}}} = 25.99$$

2. Solve equation 7-11 by trial and error until $AR^{2/3}$ is greater than or equal to 25.99.

Assume B = 4.0 ft

Y_{ft}	A_{ft^2}	P_{ft}	R_{ft}	$AR^{2/3}$	Remarks
2.0	8.0	8.0	1.0	8.0	Too Low
3.0	12.0	10.0	1.2	13.6	Too Low
4.0	16.0	12.0	1.3	19.4	Too Low
5.0	20.0	14.0	1.4	25.4	Too Low
5.2	20.8	14.4	1.44	26.6	Adequate

Since B does not equal 2Y, this is not the best hydraulic section.

Try B = 8.0 ft

Y	A	P	R	$AR^{2/3}$	Remarks
2.0	16.0	12.0	1.33	19.38	Too Low
3.0	24.0	14.0	1.71	34.38	Too High
2.5	20.0	13.0	1.54	26.65	Adequate

Therefore, the best hydraulic section is approximately 8 feet wide with a normal depth of flow of approximately 2.5 feet.

3. Using equation 7-8, the average channel velocity for normal depth is:

$$v = \frac{1.49}{(0.015)} (1.54)^{2/3} (0.006)^{1/2} = 10.26 \text{ fps}$$

According to equation 7-6

$$v_c = \sqrt{(32.2)(2.5)} = 8.97 \text{ fps}$$

Therefore, flow is supercritical. In practice, supercritical flow conditions should be avoided if possible. Therefore, a trapezoidal channel cross section with riprap lining should be considered for this flow configuration.

From Table 7-5, Item C-3, use n = 0.030 for dry rubble riprap.

$$\frac{Qn}{1.49\ S^{\frac{1}{2}}} = \frac{(200)(0.030)}{1.49\ (0.006)^{\frac{1}{2}}} = 52.0$$

Try B = 8 feet, 2:1 side slopes (see Table 7-10 to calculate A, P, and R).

Y	A	P	R	$AR^{2/3}$	Remarks
2.5	32.50	19.18	1.69	46.2	Too Low
2.7	36.18	20.07	1.80	53.6	Adequate

$$v = \frac{1.49}{(0.030)} (1.80)^{2/3} (0.006)^{1/2} = 5.7 \text{ fps}$$

$$v_c = \sqrt{(32.2)(2.7)} = 9.32 \text{ fps} \qquad \underline{\text{Adequate}}$$

Add a freeboard of (0.20)(2.7) = 0.54 for a channel design depth of 3.2 ft.

Example 7-2. Open Channel Minimum Slope

A concrete-lined trapezoidal channel with 2:1 side slopes and a 6-foot bottom width must carry 350 cfs within a depth of 4 feet. What is the minimum slope required to carry this design flow without overtopping the channel? Check to ensure that the velocity is greater than 3 fps.

1. Using appropriate equations from Table 7-10, with b = 6 ft, d = 4 ft, and z = 2 feet:

$$A = (6)(4) + (2)(4)^2 = 56 \text{ ft}^2$$

$$R = \frac{(6)(4) + (2)(4)^2}{(6) + (2)(4)\sqrt{(2)^2 + 1}} = 2.34 \text{ ft}$$

2. From Table 7-2, n = 0.015. Using equation 7-12,

$$K = \frac{(1.49)(56)(2.34)^{2/3}}{(0.015)}$$

3. Rearranging equation 7-13,

$$S_{min} = \frac{350}{9{,}805}^{\;2} = 0.0013 = 0.13\%$$

4. Using equation 7-8,

$$v = \frac{1.49}{0.015}(2.34)^{2/3}(0.0013)^{1/2} = 6.25 \text{ fps}$$ Adequate

Example 7-3. Concrete Open Channel Design Using Design Charts

Perform the hydraulic design calculations for a trapezoidal concrete-lined open channel with 2:1 side slopes which must carry 250 cfs on a slope of 0.20 percent.

1. From Table 7-2, n = 0.015; try a bottom width of 10 feet and select Figure 7-34.

2. Multiply Q by n.

$$(250)(0.015) = 3.75$$

and enter the abscissa of Figure 7-34 on the Qn scale at 3.75. For S = 0.002, find d_n = 2.5 feet.

3. From the ordinate of Figure 7-34, find vn = 0.10; therefore,

$$v = \frac{0.10}{0.015} = 6.67 \text{ fps}$$

4. For 250 cfs, d_c = 2.3 ft, and v_c = 7.5 fps (found directly from Figure 7-34).

5. Find the critical channel slope by following a d_c of 2.3 feet until it intersects 3.75 on the Qn abscissa. Therefore, S_c = 0.0028 ft/ft.

6. The v_c of 7.5 fps is greater than the average v of 6.67 fps; therefore, flow is subcritical.

7. Check to ensure that flow is not too close to critical flow conditions:

(0.10)(2.3) = 0.23 ft

2.3 + 0.23 = 2.53 ft

d_n < 2.53; therefore, a different channel configuration should be considered.

For B = 8.0 ft, n = 0.030 and 2:1 side slopes; use Figure 7-32.

d_n = 4.0 ft

v = 4.0 fps

d_c = 2.5 ft

v_c = 7.8 fps

S_c = 0.013 ft/ft

(0.10)(2.5) = 0.25, 2.5 + 25 = 2.75

d_n > 2.75; therefore, channel section is adequate.

8. Add a freeboard of (0.20)(4.0) = 0.8 feet for channel design depth of 4.8 feet.

<u>Example 7-4</u>. Grass-Lined Open Channel Design Using Design Charts

Design a grass-lined open channel to carry 20 cfs on a 2 percent slope, in easily eroded soil. The channel is to be lined with good bermudagrass sod, have 4:1 side slopes, and a 4-foot bottom width. The drainage area tributary to the design point is approximately 50 acres.

1. Since the City Engineering Department of Montgomery requires that side slopes for grass-lined open channels be less than 3:1 (horizontal to vertical), the 4:1 side slope is adequate, and Figure 7-42 is selected.

2. Enter the lower chart of Figure 7-42, for retardance C with Q = 20 cfs, and move vertically to the line for S = 0.02. At this intersection, read d_n = 1 foot, and normal velocity, v, = 2.6 fps. Since these data are below the dashed line which represents critical flow, this cross section has adequate capacity.

3. Repeat Step 2 for retardance D to ensure that the permissible velocity for bermudagrass is not exceeded. From Figure 7-42, d_n = 0.85 foot, v = 3.1 fps. According to Table 7-12, the permissible velocity for bermudagrass on easily eroded soils is 6 fps; therefore, the channel is adequate.

4. Size a concrete pilot channel to carry base flow, assuming a base flow of 0.35 cfs/mi^2

   DA = (50)(0.00156) = 0.078

   Q = (0.35)(0.078) = 0.0273 cfs

   Therefore, base flow is probably not significant.

5. Add a freeboard depth equal to 20 percent of the normal depth for retardance C.

   (0.20)(1.0) = 0.20

   design depth = 1.0 + 0.20 = 1.2 feet

Example 7-5. Riprap-Lined Open Channel Design Using Design Charts

Design a riprap-lined (dumped stone) open channel to carry 75 cfs on a 2 percent slope in easily eroded soil. Use a trapezoidal channel cross section with 3:1 side slopes and a 4-foot bottom width.

1. Assuming n = 0.030 and using Table 7-11 from Brater and King (1976), D/b = 0.35; therefore, D = 1.4 ft and v = 6.5 fps. A trial stone size is then selected for a velocity of 6.5 fps using Figure 7-46. This stone size is about 0.25 foot in diameter.

2. Calculate n using equation 7-14.

$$n = \frac{[(12)(0.25)]^{1/6}}{44.4} = 0.027$$

3. Using Table 7-11 from Brater and King (1976) D/b = 0.335; therefore, D = 1.34 feet, A = 10.7 ft^2, R = 0.856, and v = 7.04 fps. A check of Q using equation 7-2:

$$Q = (10.7)(7.04) = 75.3 \text{ cfs}$$

checks with design Q.

4. Using K = 0.25 ft and d = 1.34 ft

$$\frac{K}{d} = \frac{0.25}{1.34} = 0.187$$

5. Using Figure 7-47:

$$\frac{v_s}{v} = 0.58$$

6. Find the average velocity against the stone as follows:

$$v_s = (0.58)(7.04) = 4.08 \text{ fps}$$

7. Using v_s = 4.1 fps and 3:1 side slopes, stone size K is about 0.2 ft, which is close enough to 0.25 ft (the trial stone size) for design purposes. Therefore, use 2-inch-diameter stones.

8. Add a freeboard of (0.2)(1.34) = 0.27 ft for a total design depth of 1.34 + 0.27 = 1.61 ft.

   The minimum thickness of riprap lining is 2 inches with a tolerance of about 1 inch in surface elevation. Stone of equivalent spherical diameter, equal to or larger than 2 inches with a few larger stones not exceeding 3 inches, should make up 50 percent of the rock by weight.

SECTION 3.0 CULVERTS

The primary purpose of a culvert is to convey water under roadways. Culverts can differ from bridges in one or more of the following ways (Ritter and Paquette, 1960):

1. The top of the culvert does not form a part of the traveled roadway.
2. Culverts have a shorter span than bridges, (e.g., 20 feet or less of span might be called a culvert).
3. A culvert is generally designed to flow full under certain conditions, whereas bridges are not.

In addition to roadway drainage, culverts may be used as outlet structures for stormwater storage basins. Details concerning the hydraulic design of stormwater storage systems are presented in Chapter 8.

The design factors which influence the selection of a culvert include the following:

1. Hydraulics
2. Structural strength
3. Durability
4. Economics

Hydraulic design factors are the only ones dealt with specifically in this manual.

3.1 Culvert Hydraulics

Three different flow regimes may govern flow through a culvert: weir flow, orifice flow, and pipe flow. In addition, the capacity of a culvert can be controlled at two points on a culvert. A culvert is said to be operating under inlet control if the barrel has a greater hydraulic capacity than the inlet (i.e., the inlet controls culvert capacity). Conversely, a culvert is operating under outlet control if the hydraulic capacity of the barrel is less than the hydraulic capacity of the inlet (i.e., the barrel controls culvert capacity).

In general, when a culvert is operating under inlet control, the barrel will be flowing partly full. Flow under inlet control may be described mathematically by either the weir formula or the orifice formula, depending on the headwater depth. In general, when a culvert is operating under outlet control, the barrel will be flowing full. Flow under outlet control may be described mathematically by the pipe flow formula. A culvert may operate under inlet control or outlet control or both during a given storm event.

Conceptually, a culvert may be compared to a short length of pipe equipped with two valves, the first located at the inlet and the second at the outlet. The amount of water which will pass through the system is governed by the setting of only one valve. That is, if both valves are the same type and size and if the inlet valve is partly closed and the outlet valve is completely open, then the capacity of the system is controlled by the inlet valve. Thus, the outlet valve will not operate at capacity. Installing a larger outlet valve would add to the cost of the system, but would not increase the system's capacity. In order to increase the system's capacity, adjustment must be made at the inlet valve. If the inlet valve is opened completely, then both the inlet and outlet valve will operate at maximum capacity simultaneously. This situation is termed a "balanced system." If a larger inlet valve were installed, the capacity of the system would remain unaltered, because the capacity would then be controlled by the smaller outlet valve.

The hydraulic design of a culvert is a trial and error process. A trial culvert size is assumed and then checked to determine if it will satisfy the conditions prevailing at the proposed location. A culvert system is selected by choosing an inlet structure, a barrel material, a barrel shape, a barrel size, and an outlet structure. The outlet section is usually the same as the inlet section in order to achieve a symmetrical installation. If the outlet velocity is high enough to cause erosion at the culvert outlet (see permissible velocities in Tables 7-12 or 7-13), erosion control may be required. In this situation, the designer may have the choice of lowering the allowable high water at the culvert entrance (which will increase the culvert size and lower the outlet velocity) or providing energy dissipation and erosion protection at the outlet (see Chapter 9).

It is also important to note that storm sewer systems may be analyzed using the nomograph solutions presented below for culvert hydraulic problems. These nomograph solutions provide a quick method to determine water surface elevations at critical points in the storm sewer system such as at manholes and drop inlets.

<u>3.1.1 Inlet Control</u>. When a culvert is operating under inlet control the barrel of the culvert has a greater hydraulic capacity than does the inlet. For this reason the capacity of the culvert is dependent only upon the inlet properties and is independent of barrel properties. Under inlet control the culvert functions either as a weir or as an orifice. In practice, inlet control exists for slopes of 1 percent or greater.

Three different configurations of inlet control are illustrated on Figure 7-49. When the depth of water approaching the culvert is less than the culvert height, the flow rate being passed is governed by weir control. As the depth of approaching water increases, the entrance of the culvert will submerge. Submergence occurs when the ratio of the depth of the approaching water, HW, to the height of the culvert, D, exceeds a certain limit. This limit depends on the inlet geometry and normally lies between 1.2 and 1.5. When the entrance is submerged and the control is at the inlet, the flow will be governed by orifice flow.

Nomographs have been developed which provide a headwater-discharge relation for culverts operating under inlet control. These inlet control nomographs are based on an analysis of experimental data obtained from various sources and provide a direct solution to inlet control culvert flow. Figures 7-50 to 7-56 are the inlet control nomographs to be used in the design and analysis of culverts. Given the size and the flow rate of the culvert, the submergence ratio of the culvert, i.e., headwater depth divided by the depth of the culvert (HW/D), may be read directly from these nomographs. Use of these nomographs is illustrated on each figure by means of an example.

<u>3.1.2 Outlet Control</u>. A culvert flowing under outlet control may be full or partly full. Outlet control with partly full flow will occur only when the submergene ratio (HW/D) is near or less than 1.0 and when the culvert is on a mild (sub-critical) slope. This type of flow does not occur often in small culverts. Also, partly full flow under outlet control is very complex and requires the computation of a backwater

curve. This type of flow, therefore, is not treated in detail in this manual. However, partly full outlet control flow is recognized in the computer program (see Appendix B) which is available for culvert design computations, and occasionally culvert design will be based on this flow regime.

Full flow outlet control occurs when the culvert barrel flows full throughout its length. This is the most common type of outlet control and the only type to be considered when design computations are carried out by hand. Hereafter the term "outlet control" will refer to full flow outlet control unless otherwise specified. A culvert flowing full under outlet control is illustrated on Figure 5-57.

Culverts flowing full under outlet control can be analyzed mathematically according to the following equation:

$$H = \left[1.0 + K_e + \frac{29n^2L}{R^{1.33}}\right] \frac{v^2}{2g} \qquad (7\text{-}15)$$

where

H = total head, or the elevation difference between the headwater (HW) and tailwater (TW), in feet

K_e = entrance loss coefficient for a given inlet. Design values are given in Table 7-14

n = Manning's roughness coefficient for the culvert barrel

L = length of culvert barrel, in feet

R = hydraulic radius of the culvert, in feet

v = average velocity of flow, in fps

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

The above equation accounts for energy losses in the culvert due to the development of the velocity head, entrances loss, and friction loss. Outlet control nomographs have been developed which provide a graphical solution to this equation for various culvert material, cross section, and inlet combinations (Figures 7-58 to 7-64). Use of these nomographs is illustrated on each figure by means of an example.

3.1.3 Head Loss Due to Bends. Occasionally it is necessary to build a culvert which has one or more bends in the alignment. If this culvert is operating under outlet control then these bends will reduce the capacity of the culvert. The head loss due to bends may be estimated by the following formula.

$$H_b = K_b \frac{V^2}{2g} \tag{7-16}$$

where

K_b = the bend loss coefficient

The bend loss coefficient Kb may be estimated as follows:

$$K_b = 0.05 \sqrt{\Delta/90} \tag{7-17}$$

where

Δ = angle of bend, in degrees

The above equations apply to sharp bends. If the culvert alignment is changed by means of a circular curve with a radius equal to or greater than four culvert diameters, then energy loss in the bend may be ignored.

If bend losses are encountered in design, these losses should be computed by the above equation and added to the total head obtained by application of the general outlet control equation. In this manner the total required energy head will be obtained.

The actual headwater depth at the culvert inlet depends on the total head as discussed above, the culvert outlet conditions, and the length and slope of the culvert barrel. These items are covered in detail in the discussion of outlet control computations.

3.2 Culvert Design Data

3.2.1 Discharge. The design discharge is obtained by conducting an appropriate hydrologic analysis for the subject watershed using procedures presented in Chapter 5.

3.2.2 Allowable Headwater. Headwater depth is defined as the depth of the water above the culvert flow line measured at the culvert entrance. The allowable headwater depth is a site parameter which is subject to some degree of engineering judgment on the part of the designer. Ritter and Paquette (1960) proposed that the allowable headwater be based on one of the following factors:

1. A certain permissible freeboard below the proposed height of fill.

2. The elevation of permissible flooding upstream.

3. Where 1 and 2 above will permit a large headwater depth, it may be limited by an allowable outlet velocity.

4. Some lesser headwater depth as governed by other design considerations or departmental policy.

The condition which yields the lowest headwater depth governs the hydraulic design process.

3.2.3 Type of Culvert. The type of culvert (i.e., barrel material and inlet type) is usually governed by the type of structure being designed. The designer does, however, have the option of considering both a box culvert and a circular culvert at any given location. At locations where it is questionable which type of structure will yield the most economical design, both structure types should be considered and the least costly type selected.

3.2.4 Culvert Length and Slope. The length and slope of the culvert are functions of the stream being enclosed, the geometry of the roadway embankment, and the skew angle of the culvert. Design values may be obtained by scaling from the strip map or cross sections.

3.2.5 Tailwater Depth. The tailwater depth is influenced by conditions downstream from the culvert outlet. If the culvert outlet is operating in a free outfall condition, the tailwater is taken as 0.0. If the culvert discharges into an open channel, the tailwater is equal to the normal depth of flow in that channel. If the culvert outlet is located near the inlet of a downstream culvert, the headwater elevation of the downstream culvert may define the tailwater depth for the upstream culvert. In any case the tailwater depth is defined as the depth of water measured from the flow line of the culvert (invert) at the outlet to the water surface elevation at the outlet.

3.3 Inlet Computations

All inlet control computations are carried out with the aid of the inlet control nomographs. Given the size and flow rate of the culvert, the submergence ratio, HW/D, may be read directly from these nomographs. For pipe or pipe arch culverts the submergence ratio is presented as a direct function of the culvert size and discharge. For box culverts it is necessary to first determine the flow rate per foot of width, Q/B, expressed in cfs per foot. The submergence ratio is then given as a function of the height of the box, D, and the flow rate being conveyed per foot of box, Q/B.

Knowing the submergence ratio, HW/D, the headwater depth, HW, expressed in feet is computed as follows:

$$HW = \frac{HW}{D} D \tag{7-18}$$

where

HW = headwater depth, in feet

D = height or diameter of the culvert, in feet

Each nomograph has three submergence ratio scales, each corresponding to a different inlet concentration. Care should be exercised to use the scale corresponding to the inlet type being investigated.

3.4 Outlet Computations

The computation of headwater depth based on outlet control is somewhat more complex than the inlet control computations. Several separate steps are involved.

3.4.1 Total Head. The total energy head required to pass a given flow rate through the culvert may be computed using the pipe flow, equation 7-15, or the proper outlet control nomograph, Figures 7-58 to 7-64.

On the outlet control nomographs the total head, H, is presented as a function of the entrance loss coefficient, K_e, the length of the barrel, L, the size of the culvert, and the discharge, Q. Two steps are required to use these nomographs. First, given the entrance loss coefficient, K_e, the barrel length, "L", and the culvert size, a point on the turning line of the nomograph is established. Next, given the discharge, Q, and the point on the turning line, the total head, H, is read directly from the chart. This procedure is illustrated on each of the outlet control nomographs.

These nomographs provide a graphical solution to the pipe flow equation assuming that there are no losses due to bends. If bends are encountered in design, the bend loss, H_b, should be computed by means of the previously discussed equations for head loss due to bends (Subsection 3.1.3). The computed value of the bend loss is added to the value of total head, H, obtained from the proper outlet control nomograph in order to obtain the true value of total head.

It should be noted that Figure 7-58, "Outlet Control Nomograph for Square Concrete Box Culverts" applies to square box culverts <u>only</u>. In the analysis of box culverts of rectangular cross section, the total head must be computed by means of the general pipe flow equation (7-15). The nomograph does not apply in this case.

<u>3.4.2 Design Tailwater</u>. The tailwater condition which prevails during the design event is termed the "design tailwater," DTW. The design tailwater may be a function of downstream conditions or of the culvert outlet conditions. The depth of water at the culvert outlet due to downstream conditions is termed the "tailwater," TW, and is discussed in Subsection 3.2.5.

The distance from the flow line at the culvert outlet to the elevation of the total energy line at the culvert outlet is termed the "head at the outlet," h_o, and is computed as follows:

$$h_o = \frac{D + dc}{2} \qquad (7\text{-}19)$$

where

h_o = head at the culvert outlet, in feet, for a nonsubmerged outlet condition

D = depth of the culvert, in feet

d_c = critical depth at the culvert outlet, in feet

The critical depth for various sizes and types of culverts may be determined from Figures 7-65 to 7-70. If the critical depth, d_c, read from the charts is greater than the depth of the culvert, D, then h_o is set equal to D.

Given the tailwater depth at the outlet, TW, and the total head at the outlet, h_o, the design tailwater, DTW, is determined as follows:

for $h_o > TW$

$DTW = h_o$

for $h_o < TW$

$DTW = TW$

3.4.3 Headwater Depth. Having established the total head and the design tailwater depth, the headwater depth is computed as follows:

$$HW = H + DTW - S_o L \tag{7-20}$$

where

HW = headwater depth for outlet control, in feet (all other terms are as previously defined.)

It should be noted that the product of the culvert slope times the culvert length, $S_o L$, is equal to the difference in elevation between the culvert inlet and the culvert outlet. This value may be substituted for the $S_o L$ term in the above equation.

3.5 Culvert Design Procedure

As noted previously, the procedure to design a culvert is a trial and error process. The design process begins with the assumption that inlet conditions control the capacity of a given culvert. This trial design capacity is then checked for outlet control conditions, and the smaller capacity is used for design purposes. This design process is summarized in the following five steps:

1. Assemble design data.

2. Assume a culvert size, and then adjust the size of the culvert by means of inlet control computations. The smallest size culvert which will pass the design flow rate at or below the allowable headwater depth is selected as the trial culvert size.

3. Find the actual headwater depth, HW, for the trial culvert size determined in Step 2, using:

   a. Inlet control computations.

   b. Outlet control computations.

   c. Comparison of headwater depths found in Steps 3a and 3b. The larger value controls actual flow conditions.

4. Compare HW to AHW.

   a. If HW < AHW, culvert is adequate. However, a smaller culvert may be considered.

b. If HW > AHW, increase culvert size and return to Step 3.

5. Compute outlet velocity.

a. For inlet control use Manning's equation.

b. For outlet control use $v = Q/A$.

3.5.1 Trial Size Selection. A trial culvert size can be determined by knowing or assuming the entrance velocity of the culvert. A typical entrance velocity is 10 fps. A trial size may also be determined using the appropriate inlet control nomograph. Using an HW/D ratio of approximately 1.5, a trial size can be found directly from the nomograph. If reasons for lesser or greater relative depth of headwater exist, another value may be used for this trial selection. If the trial size is obviously too large, multiple culverts should be considered. Raising the embankment height or using pipe arch and box culverts with width greater than height may also be considered.

3.5.2 Culvert Plans. Construction plans for culverts should indicate the type, size, and length of the culvert. In addition, the drainage area of the tributary watershed, the design discharge, and design frequency should be indicated. Finally, the elevation of the culvert flow line at the inlet and outlet should be on the profile portion of the plan-profile sheets.

3.6 Example Problems

Example 7-6. Culvert Design Using Nomographs

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 ft/ft
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. A trial culvert size is selected assuming HW/D = 1.5; therefore:

$$D = \frac{HW}{1.5} = \frac{10}{1.5} = 6.67 \text{ feet}$$

Try a 72-inch concrete pipe.

2. Find the actual headwater depth for the trial culvert size.

   a. For inlet control, using Figure 7-51, 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.

   HW = (1.23)(5.0) = 6.15 feet

   Q = 220 cfs, D = 60 inches, yielding HW/D = 1.70

   HW = (1.70)(5.0) = 8.5 feet

   b. For outlet control, using Figures 7-59 and 7-66, TW = 3.5 and 4.0 is less than D = 5.0 feet.

   From Table 7-14, K_e = 0.5, Q = 160 cfs, D = 60 inches, yielding d_c = 3.6 feet (Figure 7-66)

   Since d_c is less than D = 5.0 feet, h_0 is determined using equation 7-19.

$$h_0 = \frac{5.0 + 3.6}{2} = 4.3 \text{ feet}$$

   From Figure 7-59, H = 2.2 feet. Using equation 7-20,

$HW = 2.2 + 4.3 - (0.01)(200) = 4.5$ feet

$Q = 220$ cfs, $D = 60$ inches,
gives $d_c = 4.2$ feet (Figure 7-66)

Since d_c is less than $D = 5.0$ feet, h_0 is determined using equation 7-19.

$$h_0 = \frac{4.2 + 5.0}{2} = 4.6 \text{ feet}$$

From Figure 7-59, H = 4.2 feet.
Using equation 7-20,

$HW = 4.2 + 4.6 - (0.01)(200) = 6.8$ feet

c.

Flow (cfs)	Headwater (feet) 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 Q_{25}; however, HW = 10.8 for Q_{100}; therefore, it is not adequate.

4. Compute the outlet velocity. Since the culvert will operate under inlet control, use Manning's equation as expressed by equation 7-35.

$$v = \frac{0.592}{(0.012)}(5.0)^{2/3}(0.01)^{1/2}$$

$v = 14.4$ fps

SECTION 4.0 STORMWATER INLETS AND GUTTER FLOW

Stormwater inlets provide a place for surface runoff to enter a closed conduit. Since inlets are usually associated with pavement drainage, the characteristics of gutter flow must be determined as part of the design process. This section begins with a discussion of general considerations related to the design of stormwater inlets. A brief discussion related to the analysis of gutter flow is then presented. Section 4.0 concludes with specific design information related to curb-opening inlets, grate inlets, and combination inlets.

4.1 General Design Considerations

4.1.1 Inlet Types. Three types of inlets are available to remove stormwater from a paved surface:

1. Curb-opening inlets

2. Grate inlets

3. Combination inlets

A curb opening inlet is a vertical opening in a curb through which the gutter flow passes. If the gutter is depressed in the area of the curb opening, the capacity of the inlet is increased significantly. A grate inlet is an opening in the gutter covered by one or more grates through which the water falls. A combination inlet is comprised of a grate inlet placed directly in front of a curb-opening inlet. Curb-opening inlets are generally preferred to grated inlets because they are usually less susceptible to clogging. In addition, curb-opening inlets are less hazardous to cyclists. Although curb-opening inlets do not eliminate the possibility of clogging, their use is recommended as good engineering practice when sump conditions exist (i.e., stormwater enters the inlet from both directions).

4.1.2 Inlet Sizing and Spacing. Stormwater inlets are placed either on a continuous grade or in a sump condition. When stormwater enters an inlet from one direction on a continuous slope such that flow exceeding the inlet capacity is transported past the inlet, a continuous grade condition exists. When the inlet is located at a point where the gutter slope changes directions such that stormwater enters the inlet from both directions, a sump condition exists. Once the type of stormwater inlet and the locations of continuous grade and sump conditions are identified, the sizing and spacing of stormwater inlets is generally determined by evaluating the following inlet design factors and guidelines.

1. Design storm return period for the type of drainage system considered (see Table 5-1).

2. Allowable gutter spread for the design discharge (see Table 7-15).

3. Allowable depth of gutter flow for the design discharge (see Table 7-16).

4. Maximum distance between inlets allowed by the City Engineering Department of Montgomery (500 feet).

5. Intercept or reduction in the design discharge required at each inlet.

6. Stormwater inlets must be provided at all points where sump conditions exist.

7. Gutter flow should be intercepted prior to a bridge deck or highway intersection.

Criteria for evaluating each of the above design considerations are briefly described below.

<u>4.1.2.1 Design Storm</u>--Stormwater inlets should be sized and located such that a balanced conveyance system is obtained. The term "balanced system" means that inlet capacity is adequate to fully utilize the conveyance capacity of the system to which the inlet discharges. Guidelines for selecting a design storm in Montgomery are presented in Table 5-1. In most cases, a 25-year design storm will be used for inlet design. Higher design storm return periods should be considered if the inlets are to be located at critical use points in the City. In any case, the 100-year design storm should be considered for designating the pathway of overland flow when the conveyance system's capacity is exceeded.

<u>4.1.2.2 Allowable Gutter Spread</u>--The maximum allowable spread of gutter flow during the design storm on any street is 12 feet. Additional considerations for determining the maximum allowable gutter spread are presented in Table 7-15. For local traffic the gutter flow may spread to the crown of the street if it is less than 12 feet. At least one lane must be free of water on a collector street during the design storm. For arterial streets, at least one lane must be free of water in each direction. Finally, no encroachment of gutter flow should be allowed on any traffic lane during the design storm for a major freeway.

<u>4.1.2.3 Allowable Gutter Depth</u>--The maximum allowable depth of gutter flow is a function of the curb height. Recommended maximum depths of gutter flow are presented in Table 7-16 as 0.20, 0.40, and 0.45 feet for curb heights of 3, 6, and 8 inches, respectively.

<u>4.1.2.4 Inlet Intercept</u>--On a continuous grade, the intercept required at each inlet is generally less than 70 percent of the design discharge. However, additional considerations are required for bridges and highway intersections. When a bridge is located on a continuous grade, approaching gutter

flow should be intercepted prior to the bridge deck during the design storm. In addition, inlets should intercept gutter flow at the upstream side of an intersection such that no more than 2.0 cfs is allowed to flow into the intersection during the design discharge.

4.1.2.5 Inlets for Sump Conditions--For sump conditions, stormwater inlets should be sized such that 100 percent of the gutter flow arriving at the inlet during the design event is intercepted. The allowable depth of ponding is controlled by the criteria of maximum depth at the curb or maximum gutter spread, as discussed previously. The most restrictive of these criteria should establish the basis for sizing and locating inlets.

Three inlets should be considered where sump conditions exist, one at the sump and one on each side of this point where the grade elevation is approximately 0.2 feet higher than at the low point. These two additional inlets limit the deposition of sediment at the low point and also provide a safety factor if the sump inlet becomes clogged.

4.1.2.6 Summary--As a general rule, stormwater inlets should be placed at all sump points on a street, before intersections, before pedestrian crosswalks, and before bridge decks. The maximum distance between inlets allowed by the City Engineering Department of Montgomery is 500 feet. A maximum spread of 12 feet is allowed for gutter flow, with additional constraints based on the street classification. The maximum depth of gutter flow is a function of curb height. On a continuous grade, 70 percent or less of the design discharge should be intercepted at each inlet, while for sump conditions, 100 percent of the design discharge should be intercepted.

It should be noted that street inlets do not provide an efficient method for intercepting stormwater. They should not be used to intercept stormwater that can be intercepted by more efficient structures such as open channels.

4.2 Gutter Flow

A special form of Manning's equation is used to analyze open channel flow in shallow triangular channels such as street gutters. Manning's equation for shallow flow in triangular channels is expressed mathematically as follows:

$$Q_g = 0.56 \frac{Z}{n} S_g^{1/2} d^{8/3} \qquad (7\text{-}21)$$

where

Q_g = gutter flow rate, in cfs

Z = reciprocal of the gutter cross slope, in ft/ft

n = Manning's roughness coefficient (see Table 7-6 for gutters)

S_g = gutter slope, in ft/ft

d = depth of flow at the curb, in feet

The typical design problem requires that the depth of flow be determined for a given design discharge and gutter cross section (i.e., Z, n, and S_g). In this case, equation 7-21 can be rearranged as follows:

$$d = 1.24 \left[\frac{Q_g}{Z\sqrt{S}} \right]^{0.375} \qquad (7\text{-}22)$$

A nomograph for solving equation 7-21 was discussed in Subsection 2.2.3.1 of this chapter and is presented as Figure 7-40. Guidelines are presented on Figure 7-40 for the analysis of flow in nontriangular gutters such as curb and gutter sections.

4.3 Inlet Design

As noted in Subsection 4.1.2 of this chapter, stormwater inlets can be identified as being placed on a continuous grade or in a sump condition. Since the characteristics of flow are different for each of these conditions, a different hydraulic design procedure is required for continuous grade or sump condition inlets. Hydraulic design procedures are presented in this section for curb-opening, grate, and combination stormwater inlets. The design procedures for curb-opening and grate inlets are further classified as continuous grade or sump condition design procedures.

4.3.1 Curb-Opening Inlets. Curb-opening inlets consist of a longitudinal opening located in the face of a curb. The hydraulic capacity of a curb-opening inlet depends on where the inlet is located (i.e., continuous grade, or sump conditions), the length of the opening, and the depth of the water above the invert of the opening.

4.3.1.1 Continuous Grade--A curb-opening inlet located on a continuous grade functions as a falling head weir. The capacity of such an inlet is determined by a two-step graphical procedure. First, the length of a curb inlet required to intercept 100 percent of the gutter flow is determined. Second, the actual curb inlet length is compared to the 100 percent intercept length. If the actual length of the curb opening is greater than or equal to the 100 percent intercept length determined in the first step, all of the gutter flow will be intercepted for the design discharge. If the actual length is less than the 100 percent intercept length, bypass will occur. This two-step graphical procedure was developed by Izzard (1950) and is presented on Figures 7-71 and 7-72.

To determine the length of curb opening required to intercept the total gutter flow, Figure 7-71 is entered on the abscissa with the depth of flow at the curb, y. A straight line on the graph is then located for the appropriate inlet depression depth, a, and the discharge per foot of opening, Q_g/L_a, is obtained from the ordinate scale. The length of curb opening required to intercept the total gutter flow is then determined by dividing the design gutter flow, Q_g, by the discharge per foot of opening, Q_g/L_a.

This calculation is expressed mathematically as follows:

$$L_a = Q_g \Big/ \frac{Qg}{L_a} \qquad (7\text{-}23)$$

where

L_a = curb-opening length required to intercept the total gutter flow, in feet

Q_g = design gutter flow rate, in cfs

Q_g/L_a = ratio determined from Figure 7-70

To determine the capacity of an inlet of length, L, which is less than L_a, L is divided by L_a, and a is divided by y. The interception rate, Q_i/Q, is then determined by entering the abscissa of Figure 7-72 at the value of L/L_a. A line on the graph is then located for the appropriate a/y ratio, and the interception rate, Q_i/Q_g, is obtained from the ordinate scale. The capacity of an inlet of length L is then obtained by multiplying the interception rate, Q_i/Q_g by the design gutter flow, Q_g. This calculation is expressed mathematically as follows:

$$Q_i = Q_g \left[\frac{Q_i}{Q_g} \right] \qquad (7\text{-}24)$$

where

Q_i = inlet intercept rate, in cfs

Q_i/Q_g = ratio determined from Figure 7-71

(Q_g is defined above.)

The variables which are utilized in this two-step graphical procedure for curb inlets are illustrated on Figure 7-73. Since Figures 7-71 and 7-72 were derived theoretically to fit real world data, they are general in nature and may be applied to many pavement cross sections. These figures may also be used to determine the capacity of undepressed curb openings by setting the inlet depression depth, a, equal to zero. Example 7-7 at the end of this section illustrate this two-step graphical design procedure for curb inlets on a continuous grade.

4.3.1.2 Sump Conditions--The capacity of an unsubmerged curb-opening inlet located at a sump can be evaluated in the same manner as weir flow. Therefore, the capacity of an unsubmerged curb-opening inlet can be expressed mathematically by the following equation:

$$Q_i = 3.0Ld^{1.5} \qquad (7\text{-}25)$$

where

Q_i = inlet intercept rate, in cfs

L = length of the inlet, in feet

d = depth of water at the inlet opening, in feet

The capacity of a submerged curb-opening inlet located at a sump can be evaluated in the same manner as orifice flow. Therefore, the capacity of a submerged curb-opening inlet can be expressed mathematically by the following equation:

$$Q_i = 5.37\,A\left(d - \frac{h}{2}\right)^{1/2} \qquad (7\text{-}26)$$

where

A = waterway area of the inlet opening, in ft^2

h = height of the inlet opening, in feet

(Q_i and d are defined above.)

A graphical solution to both equations 7-25 and 7-26 is provided as a nomograph on Figure 7-74. The use of this nomograph is illustrated by example 7-8 at the end of this section.

<u>4.3.2 Grate Inlets</u>. A grate inlet consists of an opening in the pavement, usually adjacent to the curb, covered with a metal grate. The hydraulic capacity of a grate inlet depends on where the inlet is located (i.e., continuous grade or sump conditions). On a continuous grade, the capacity of a grate inlet is influenced by the depth and velocity of the approaching stormwater, the length and width of the grate, and the geometric configuration of the grate. At low points in a grade (i.e., sump conditions), the capacity of a grate inlet is controlled either by the perimeter of the grate or by the clear waterway area of the grate. The capacity at low points in a grade is independent of the geometric configuration of the inlet.

Six types of grate inlet configurations are illustrated on Figure 7-75. These grate inlet types are numbered 1 through 6 and will be referred to in this manual by these numbers as defined below:

<u>Type</u>

1 Bars with rectangular cross sections, aligned parallel to the direction of flow.

2 Bars with rectangular cross sections, aligned normal to the direction of flow.

3 Bars with rectangular cross sections, aligned normal to the direction of flow. Unlike inlet type two, these bars are tilted at an angle of 45° to the vertical in the upstream direction as shown on Figure 7-75.

4 Identical to inlet type three except that the top of the bar is beveled or rounded back from the leading edge.

Type

5 Bars with rectangular cross sections, aligned at an angle of 45° to the direction of flow.

6 Bars with a curvilinear cross section, aligned at an angle of 45° to the direction of flow. These bars are also tilted at an angle of 45° to the vertical in the upstream direction, as shown on Figure 7-75. This grate inlet configuration is known as the Rowland Inlet and has been patented. (Cassidy, 1966)

4.3.2.1 Continuous Grade--An efficient grate inlet is defined as an inlet that intercepts all the stormwater passing directly over it. An efficient grate inlet is, therefore, assumed to intercept all water flowing within the limits of the grate width and to bypass all water flowing outside the limits of that width. These hydraulic characteristics of an efficient grate inlet are illustrated on Figure 7-76. The quantity of water flowing directly over the grate can be calculated using the Manning equation as expressed below:

$$Q_e = \frac{1.49}{n} W \bar{d}^{5/3} S_g^{1/2} \qquad (7\text{-}27)$$

where

Q_e = intercept of the efficient grate, in cfs

W = width of the grate, in feet

$\bar{d}$ = average depth, in feet, of the approaching flow measured at the centerline of the grate

S_g = gutter slope, in feet/foot

Further details concerning the hydraulic analysis of open channel flow in gutters using Manning's equation were presented in Subsection 4.2 of this chapter.

Given the quantity of water which is available to an efficient grate inlet, it is necessary to determine if the grate is operating efficiently. If the grate is not operating efficiently, it is also necessary to know how much bypass of flow occurs directly over the grate.

To determine the length of grate required for efficient operation, the following equation is utilized:

$$L_o = \frac{m}{5.67} v d^{1/2} \quad (7\text{-}28)$$

where

L_o = length of grate required to obtain efficient operation, in feet

m = grate inlet constant, which depends on the type of grate inlet, (see Table 7-17)

v = average velocity of the approaching water, in ft/sec

d = maximum depth of approaching stormwater at the curb, in feet

The value of m is a measure of hydraulic efficiency for the type of grate inlet being considered. A low value of m indicates a highly efficient grate inlet, while a high value of m indicates an inefficient inlet. Typical values of m for the grate inlet types shown on Figure 7-75 are presented in Table 7-17. The average velocity of the approaching stormwater at the curb can be estimated by appropriate application of Manning's equation (7-27) and the continuity equation (7-2).

If the length of grate, L_o, determined according to equation 7-28, is less than or equal to the actual length of grate installed, the grate inlet is considered efficient. When the actual length of grate installed is less than the length of grate required to obtain efficient operation, L_o, the stormwater bypassed directly over the inlet must be determined. The amount of bypass occurring when the length required for efficient operation, L_o, is longer than the actual length installed, L, the following equation is used:

$$q = Q_e \left(1 - \frac{L^2}{L_o^2}\right)^2 \quad (7\text{-}29)$$

where

q = that portion of stormwater flowing within the limits of the grate which is bypassed, in cfs

L = actual length of the inlet, in feet

(Q_e and L_o are defined above.)

The procedure for determining the intercept of a grate inlet on a continuous grade can be summarized in the following steps:

1. Calculate the total design flow approaching within the limits of the grate, using equation 7-27.

2. Calculate the length of grate required to obtain efficient operation of the grate inlet, using equation 7-28.

3. Compare the required length, L_o, to the actual length installed, L. If the actual length, L, is greater than or equal to the required length, L_o, then the actual intercept, Q_i, is equal to the efficient intercept, Q_e, from Step 1. If the actual length is less than the required length, continue to Step 4.

4. Calculate that portion of the stormwater flow within the limits of the grate which is bypassed, q. Using equation 7-29, the actual intercept, Q_i, is then equal to the efficient intercept, Q_e, minus the bypassed stormwater, q. This calculation is expressed mathematically as follows:

$$Q_i = Q_e - q \qquad (7\text{-}30)$$

where all parameters are defined above.

This hydraulic design procedure for grate inlets operating on a continuous grade is illustrated by example 7-9 at the end of this section.

4.3.2.2 Sump Conditions--Depending on the depth of water at the grate, a grate inlet operating under sump conditions will function as a weir or an orifice. Therefore, the capacity of a grate inlet is independent of the geometric configuration of the grate.

Weir flow governs the capacity of a grate inlet operating under sump conditions when the depth of flow is less than 0.4 feet. The capacity of a grate inlet operating under sump conditions can thus be calculated using equation 7-31 when the depth of flow is less than 0.4 feet.

$$Q_i = 3.0\ p\ d^{1.5} \qquad (7\text{-}31)$$

where

p = effective perimeter of the grate, in feet

d = depth of water above the top of the grate, in feet

(Q_i is defined above.)

Only that portion of the grate's perimeter along which flow enters the inlet should be considered when determining the effective perimeter.

Orifice flow governs the capacity of a grate inlet operating under sump conditions when the depth of flow is greater than 1.4 feet. The capacity of a grate inlet operating under sump conditions can thus be calculated using equation 7-32 when the depth of flow is greater than 1.4 feet.

$$Q_i = 5.37\ A\ d^{0.5} \qquad (7\text{-}32)$$

where

A = the clear waterway area of the grate, in ft^2

(Q_i and d are defined above.)

Between the depths of 0.4 feet and 1.4 feet, the capacity of the grate is undefined and may be assumed to be the lesser of the two values determined from equations 7-31 and 7-32.

In practice, the effective waterway area or perimeter determined using equations 7-31 and 7-32 should be at least doubled, primarily because of the tendency for trash to collect on the grate. Where the danger of clogging is slight, a safety factor less than two might be considered. As discussed in Subsection 4.3.3, if a combination inlet is used, the safety factor can be reduced.

A graphical solution to equations 7-31 and 7-32 is provided on Figure 7-77. The hydraulic design procedure for grate inlets operating under sump conditions is illustrated by example 7-10 at the end of this section.

<u>4.3.3 Combination Inlets</u>. Considering combination inlets located on a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate may be assumed equal to the capacity of the grate inlet alone. Use of such inlets on a continuous grade is, therefore, uneconomical except in areas where considerable debris may be expected. In such cases, the curb opening provides a safety factor against clogging of the grate. When the curb opening extends upstream from the grate, the capacity of the inlet may be estimated as the capacity of that portion of the curb opening upstream of the grate, plus the capacity of the grate. The proper procedure for computing the capacity of such an inlet is to compute the intercept of the curb opening for the portion of the opening located upstream from the grate. This intercept is then subtracted from the total gutter flow, which yields the curb opening bypass. The grate inlet intercept is then computed using the

curb inlet bypass as the approach gutter flow. The capacity of the combination inlet is the sum of the curb opening intercept and the grate intercept.

All debris carried by the stormwater runoff that is not passed by upstream inlets must be passed by the inlet located at the low point. Therefore, these inlets are prone to clogging. As discussed previously in Subsection 4.3.2.2, the capacity of a grate inlet at a low point may be computed by the proper equation or by use of Figure 7-77. Also, it has been recommended that the computed effective perimeter, or clear waterway area, of the grate be doubled in order to provide a safety factor against clogging. However, if a combination inlet is used, the curb opening will provide the necessary safety factor against clogging, and the effective perimeter or clear area of the grate need only be equal to that given by the computations. Thus, the use of combination inlets at low points in grade is recommended.

4.4 Inlet Reduction Factors

To account for unforeseen malfunctions due to clogging with debris, the theoretical capacity of inlets, determined by procedures presented above, should be reduced for design purposes. The percentages of theoretical capacities allowed for inlet designs in Montgomery are presented in Table 7-18.

4.5 Example Problems

Example 7-7. Curb-Opening Inlet on a Continuous Grade

Given the following data for a curb-opening inlet on a continuous grade, find the inlet intercept.

Curb height = 6 inches
Crown slope = 0.01 ft/ft
Pavement cross slope = 0.03 ft/ft, Z = 33.3
Concrete gutter troweled finish
Inlet depression = 3 inches
Inlet length = 4 feet

1. From Table 7-16, the maximum depth of flow for a 6-inch curb is 0.40 feet. For a cross slope of 0.03 ft/ft and an allowable gutter spread of 12 feet, the maximum allowable curb depth is 0.36 feet. From Table 7-6, n = 0.012. Using equation 7-21,

$$Q_g = (0.56)\,\frac{(33.3)}{(0.012)}\,(0.01)^{1/2}\,(0.36)^{8/3}$$

$$Q_g = 10.2 \text{ cfs}$$

2. From Figure 7-71, $Q/L_a = 0.32$ for $Y = 0.36$ feet and $a = 3$ inches. Using equation 7-23,

$$L_a = \frac{10.2}{31.9} = 31.9 \text{ feet}$$

$$\frac{L}{La} = \frac{4}{31.9} = 0.13$$

$$\frac{a}{y} = \frac{0.25}{0.36} = 0.69$$

From Figure 7-72, $Q_i/Q = 0.19$. Using equation 7-24,

$$Q_i = (10.2)(0.19) = 1.94 \text{ cfs}$$

For design purposes, this theoretical capacity is multiplied by the reduction factor of 0.80 from Table 7-18. Therefore, the design intercept capacity is:

$(1.94)(0.8) = \underline{1.55 \text{ cfs}}$

<u>Example 7-8</u>. Curb-Opening Inlet at a Sump

Given the following data for a curb-opening inlet at a sump, find the length of inlet required.

Q_g from one direction = 4.0 cfs
Q_g from the other direction = 6.0 cfs
Curb height = 6 inches
Inlet depression = 3 inches
Height of curb inlet = 6 inches = h

1. From Table 7-16, the maximum depth of flow for a 6-inch curb is 0.40 feet. Assume the pavement cross slope is sufficient to allow a 12-foot gutter spread at a depth of 0.40 feet.

$$\frac{d}{h} = \frac{0.40}{0.50} = 0.80$$

From Figure 7-74, $Q/L = 0.8$ cfs/ft.

2. $Q = 4 + 6 = 10.0$ cfs; therefore,

$$L = \frac{(10)}{0.8} = 12.5 \text{ feet}$$

For design purposes, this theoretical length should be divided by the reduction factor of 0.80 from Table 7-18. Therefore, the design length is

$$L = \frac{12.5}{0.8} = 15.6 \text{ feet}$$

Example 7-9. Grate Inlet on a Continuous Grade

Given the following data for a longitudinal bar grate inlet (inlet type 1 on Figure 7-75) on a continuous grade, determine the length of grate required to obtain efficient operation. If the actual length of grate is less than the length of grate required to obtain efficient operation, find the amount of bypass.

Curb height = 6 inches
Crown slope = 0.01 ft/ft
Pavement cross slope = 0.03 ft/ft
Concrete gutter, troweled finish
Width of grate = 2.0 feet
Length of grate = 4.0 feet

1. From Table 7-16, the maximum depth of flow for a 6-inch curb is 0.40 feet. For a cross slope of 0.03 and an allowable gutter spread of 12 feet, the maximum allowable curb depth is 0.36 feet and $\bar{d} = 0.33$ feet. From Table 7-6, $n = 0.012$. Using equation 7-21,

$$Q_e = \frac{1.49}{(0.012)}(2)(0.33)^{5/3}(0.01)^{1/2}$$

$$Q_e = 3.9 \text{ cfs}$$

$$v = \frac{Q_e}{A}, \ A = 0.66 \text{ ft}^2$$

$$v = \frac{3.9}{0.66} = 5.91 \text{ fps}$$

2. Determine the length of grate required for efficient operation using equation 7-28. From Table 7-17, m = 2.7

$$L_0 = \frac{(2.7)}{(5.67)}(5.91)(0.36)^{1/2}$$

$$L_0 = 1.69 \text{ feet}$$

For design purposes, this theoretical length should be divided by the reduction factor of 0.60 from Table 7-18. Therefore,

$$L_0 = \frac{1.69}{0.60} = 2.11 \text{ feet}$$

Since L_0 is ~~greater~~ than 2 feet, the installed grate ~~does not~~ intercepts 100 percent.

3. The bypass is estimated using equation 7-29.

$$q = (3.9)\left[1 - \frac{(2)^2}{(2.11)^2}\right]^2$$

$$q = 0.04 \text{ cfs}$$

Using equation 7-30, the actual intercept is

$$Q_i = 3.9 - 0.04 = 3.86 \text{ cfs}$$

Example 7-10. Grate Inlet at a Sump

Given the following data for a longitudinal bar grate inlet at a sump, determine the inlet intercept rate. Assume the pavement cross slope is sufficient to allow a 12-foot gutter spread at a depth of 0.40 feet.

Curb height = 6 inches
Grate width, b = 2 feet
Grate length, a = 3 feet
Grate spacing, w = 0.15 feet

1. At a depth of 0.40 feet, weir flow governs the capacity of a grate inlet at a sump. From Figure 7-77,

$P = 2(a + b) = 2(2 + 3) = 10.0$ feet

Using equation 7-31

$Q_i = (3.0)(10.0)(0.40)^{1.5}$

$Q_i = 7.6$ cfs

For design purposes, this theoretical capacity should be multiplied by the reduction factor of 0.50 from Table 7-18. Therefore, the design capacity is

$Q_i = (0.5)(7.6) = 3.8$ cfs

SECTION 5.0--STORM SEWERS

The hydraulic design of a storm sewer system generally requires two major computations. The first provides a determination of the peak stormwater flow rate arriving at particular design points in the system. The second involves sizing the storm sewers such that they contain these peak flow rates. Step one is primarily a hydrologic problem, whereas step two is primarily a hydraulic problem. Details concerning the hydrologic and hydraulic design of storm sewer systems are presented in this chapter. It should be noted that the hydraulic computations for large or complex storm sewer systems can become unmanageable if done without computer assistance. A brief description of computer-aided sewer design procedures is presented in Appendix B.

Since this manual does not deal with storm sewer structural requirements, construction materials, appurtenances, and construction methods, additional reference material should be consulted by the design engineer. These references include the ASCE manual and report on Engineering Practice Number 37 (also known as WPCF Manual of Practice Number 9) published in 1970, and two books by the American Concrete Pipe Association (1980 and 1978). The Concrete Pipe Design Manual (American Concrete Pipe Association, 1978) is the recommended reference for structural design requirements.

Prior to conducting the hydrologic and hydraulic computations related to storm sewer design, the general layout of the drainage system must be determined. Thus, Section 5 includes the following major topics:

1. Drainage system layout
2. Hydrologic computations
3. Hydraulic computations

5.1--Drainage System Layout

The first step in the design of storm drain systems is the determination of drop inlet and manhole structure locations. This determination is not attempted until such time as the pavement drainage (drop inlet design) is complete and the inlet sizes and intercepts have been established. Factors to be considered when laying out a storm sewer system include conduit location, alignment, type, and slope. In addition, the installation of manholes must be considered.

Easement requirements for storm sewer pipes in Montgomery must also be considered when the drainage system layout is developed. For 15- to 36-inch pipes, a 10-foot easement width is required, while for 42- to 72-inch pipes a 15-foot easement is required. If box culverts or other types of structures are used, the easement should provide 5 feet of space on each of the outside dimensions to the structure.

5.1.1--Conduit Location. Trunk or main line conduits are to be located outside the roadway pavement, if practicable. Both main line and lateral conduit should be given sufficient study to ensure the selection of the most feasible plan. The final location of a conduit system should be established such that its length is a minimum consistent with hydraulic requirements and such that the entire system is economically designed for both construction and maintenance.

Flowline depth or vertical locations of conduits are generally determined by size of conduit and slope requirements. However, such factors as lateral connections and vertical clearance of obstructions must also be considered in the determination of flowline depth. When a conduit is located under the pavement, the top of the conduit should be held to a depth of at least 6 inches below the bottom of the base course. The crowns of all pipes connecting to inlets and manholes are to be held at the same elevation, if practicable.

5.1.2 Conduit Alignment. Certain hydraulic losses are effected by changes in alignment. Therefore, any change in alignment between connected structures should be avoided, if possible. This is especially true on trunk or main line segments of a storm sewer system. The alignment when combining lines of conduit should be arranged to minimize head loss at the junction. An angular change in alignment of conduit 30 inches or less in diameter is permissible on lateral lines to effect right angle entry into an inlet or manhole.

5.1.3 Conduit Type. Permissible types of conduit for storm sewers include the following:

1. Non-reinforced concrete--ASTM C14.

2. Reinforced concrete pipe--ASTM C76.

   Class III is a minimum requirement.
   Class IV is required under streets.

3. Precast concrete box sections--ASTM C789.

4. Cast in-place concrete box sections.

Any variation from these types of storm sewer conduit must be approved by the City Engineering Department of Montgomery.

Structural design requirements for storm conduit sections should be developed using the information published by the American Concrete Pipe Association (1978). It should be noted that when elliptical pipe is substituted for circular pipe, the hydraulic capacity of the elliptical pipe is to be equivalent to that of the circular pipe, given the same conditions. Example 7-11 at the end of this section demonstrates the type of calculations which must be performed to ensure that a concrete arch pipe will carry the same flow as a circular pipe.

5.1.4 Conduit Slope. As a general rule, a uniform slope is maintained between structures. A break in slope between any two structures is used only when unavoidable. If practicable, storm sewers are designed with conduit slopes sufficient to develop a self-cleaning velocity of 3 feet per second when flowing at a depth of one-fourth of the pipe diameter.

5.1.5 Manholes. Manholes are installed at all changes in pipe grade or size; at changes in pipe alignment where the pipe diameter is 30 inches or less; at all intersections; and at intervals not greater than 400 feet for conduit 15 inches or less in diameter, and 500 feet for conduit 18 inches or more in diameter. The minimum diameter for manholes is 42 inches, regardless of pipe size. Manholes are not to be placed in traffic lanes.

As a general rule, a drop inlet is not used as a manhole. It is permissible, however, when consistent with good design and in the interest of economy, to construct an inlet to serve as a manhole, provided stormwater alone is involved.

5.2 Hydrologic Computations

After the geometric layout of the storm drainage system has been established, the conduits are sized to carry the intercepted gutter flow. When a storm sewer serves as a direct outfall for a stormwater inlet without connecting to other storm conduits, no additional hydrologic computations are required, and the conduit can be sized to carry the maximum intercepted flow rate. However, if this conduit is one component of a large storm sewer system, a hydrologic channel routing procedure should be employed from each inlet point to the downstream point in question since peak flows will be modified by channel storage. The magnitude of this reduction is a function of the length of the conduits and the velocity of flow through the conduits. Three methods of performing the hydrologic computations for multi-conduit storm sewer system are presented in the following discussion: (1) the Summation of Flows Method, (2) the Rational Method, and (3) the Inlet Hydrograph Method. The Inlet Hydrograph Method is presented in greatest detail since it will provide the most realistic design. These three methods are compared in example 7-17 at the end of this section.

5.2.1 Summation of Flows. This method does not take into account the impact of channel storage on peak flow rates as intercepted stormwater is transported through a long storm sewer system. By the Summation of Flows Method the hydrologic computations for sewer design are performed exactly as the title suggests. Each peak flow rate which is intercepted by all inlets upstream of the design point are added together. This sum is then used to size the conduit at the design point in question. The Summation of Flows Method is the simplest and most conservative of the three methods presented.

The fact that peak flows arrive at the design point at different points in time cannot be accounted for by the Summation of Flows Method. In addition, it does not account for the fact that the individual inlet peak flows will be reduced due to the influence of channel storage in the upstream portion of the storm sewer system. This method cannot be considered good engineering practice for large systems because it is not a reasonable representation of the behavior of the actual system. In general, this method can yield large overestimates of the peak flows arriving at the lower portions of a storm sewer system.

5.2.2 Rational Method. A peak rate of flow is calculated at each design point in the storm sewer system by the Rational Method. These computations are based on a new design storm at each design point. The design storm return period may be the same at each point; however, the time of concentration must vary. The time of concentration is calculated for each design point according to equation 5-9 in Chapter 5, as the sum of the inlet time and the reach travel time. Since the duration of the design storm is increased as the design proceeds in the downstream direction, the design rainfall intensity is decreased. Thus, for large systems, the design flow rates will be lower than those obtained by the Summation of Flows Method. Such an application of the Rational Method has a logical basis and indirectly accounts for the fact that peak flow rates arrive at each individual inlet at different points in time. However, the Rational Method does not account for channel storage which is available in the upstream portion of the storm sewer system. In addition, the Rational Method implies that all runoff originating from a given inlet area is intercepted by its respective inlet. Therefore, the design assumes the bypass at each inlet to be zero.

Detailed procedures for application of the Rational Method for designing storm sewers are presented by Yen et al., (1974) and the ASCE/WPCF Joint Committee (1970).

5.2.3 Inlet Hydrograph Method. According to the Inlet Hydrograph Method, the time distribution of runoff entering each individual inlet is assumed to be triangular, with the peak equal to the peak rate of inlet intercept and the time base equal to 2T. The term T, in minutes, is defined to be the time from the beginning of intense rainfall to the time at the end of the intense rainfall period. For the purposes of design, the rainfall is assumed to occur at a uniform rate for a duration equal to the time of concentration of the inlet drainage area. Since the design storm rainfall is intense and is uniformly distributed in time, the term T is equal to the duration of the design storm. In this manner, an inlet hydrograph is defined for each inlet in the storm sewer system. The basic features of the inlet intercept hydrograph are illustrated on Figure 7-78.

The peak rate of flow in the gutter, Q_g, shown on Figure 7-78, is equal to the peak rate of runoff originating from the inlet drainage area in question, Q_i, plus the inlet bypass, Q_b, from the upstream inlet. The peak rate of runoff from the inlet drainage area in question, Q_i, can be determined by any appropriate stormwater runoff estimating procedure as presented in Chapter 5. In practice, the Rational Method is generally used to estimate the peak rate of runoff originating from inlet drainage areas less than 50 acres. The inlet bypass, Q_b, can be determined using the appropriate procedures presented in Section 4.0 of this chapter. The term Q_i is equal to the peak of the inlet intercept hydrograph, which is also equal to the inlet capacity for the design conditions. The inlet bypass, Q_b, is defined according to the following equation:

$$Q_b = Q_g - Qi \qquad (7\text{-}33)$$

where all terms given are defined above.

The conduit connecting the inlet with the next segment of the system must be sized to carry the full inlet intercept peak, Q_i. However, as the intercepted stormwater moves through this first conduit, the time base of the hydrograph is lengthened. Since the total volume of runoff which leaves the conduit must equal the total volume of runoff which entered the conduit and the time base is increased, it follows that the peak rate of runoff must be reduced. This reduction is, in fact, a linear function of the travel time in the conduit and is calculated as follows:

$$Q_o = Q_i \frac{(2T)}{(2T + 0.8\ L/v)} \qquad (7\text{-}34)$$

where

Q_o = peak flow rate of the outflow hydrograph, in cfs

Q_i = peak flow rate of the inflow (inlet) hydrograph, in cfs

2T = time base of inflow (inlet) hydrograph, in minutes

L = length of stormwater conduit, in feet, from the inlet to the design point

v = average velocity of flow through the conduit, in feet per minute

The time base of the outflow hydrograph is equal to 2T + 0.8 L/v, and the time to peak of the outflow hydrograph is taken as T + 0.8 L/v. A typical routed inlet hydrograph is illustrated on Figure 7-79.

The area under the hydrograph represents a volume of water. The difference in area between the inflow hydrograph and the rising limb of the outflow hydrograph represents the volume of water which enters storage (i.e., that water which fills the pipe). Conversely, the difference in area between the outflow hydrograph and the falling limb of the inflow hydrograph represents the volume of water which leaves storage.

In order to determine the peak flow rate at some design point in the downstream portion of the storm sewer system, all upstream inlet hydrographs must be routed to the design point and added together. This will produce a hydrograph at the design point. The peak of this hydrograph is then used as the design flow rate. For example, consider a manhole which drains four inlets. To determine the peak flow rate arriving at this manhole, each of the four inlet hydrographs is routed to the manhole by means of the routing equation (7-34).

These routed inlet hydrographs are then added together to yield the inflow hydrograph at the manhole. In practice, this can be accomplished by plotting each routed inlet hydrograph with a common time axis on rectangular coordinate graph paper and adding with a pair of dividers. The peak of the resultant hydrograph is then used to design the outflow conduit. This procedure using four routed inlet hydrographs is illustrated on Figure 7-79.

The Inlet Hydrograph Method provides for the fact that peak flows originating at each individual inlet are arriving at various locations in the system at different points in time. In addition, the method also provides for the reduction in individual peaks due to the effect of channel storage on the inflow hydrographs. This method is recommended as good engineering practice in Montgomery for any storm sewer system greater than 1,200 feet in length.

5.3 Hydraulic Computations

The objective of the hydraulic design for a storm sewer system is to provide a balanced system in which all segments will be used to their full capacity without adversely affecting the drainage of any tributary area. The hydraulic computations are based on the appropriate peak runoff rates as developed according to the procedures discussed in the previous section (i.e., Subsection 5.2). Two types of flow can occur in closed conduits. If a free water surface subject to atmospheric pressure exists, the pipe flow can be considered a form of open-channel flow (see Figure 7-80). When the conduit is flowing full, the pipe is generally flowing under pressure (see Figure 7-81). Hydraulic computations for pipes flowing under open-channel and pipe flow conditions are presented in the discussion which follows.

5.3.1 Open-Channel Flow. Storm sewer systems are generally designed as non-pressure pipe systems. Under non-pressure conditions, the capacity of closed conduits can be analyzed by applying Manning's equation for uniform flow. As shown on Figure 7-80, the hydraulic grade line is the free water surface elevation and is parallel to the energy grade line under non-pressure conditions. For these conditions, Manning's equation for closed conduits can be expressed as follows:

$$v = \frac{0.592}{n} \; d_s^{2/3} \; S_o^{1/2} \qquad (7\text{-}35)$$

or

$$Q = \frac{0.465}{n} \; d_s^{8/3} \; S_o^{1/2} \qquad (7\text{-}36)$$

where

Q = design discharge, in cfs

n = Manning's roughness coefficient (see Table 7-2)

d_s = diameter of the closed stormwater conduit, in feet

S_o = pipe slope, in feet per foot

As presented in Table 7-2, Manning's roughness coefficient for concrete pipe installed in Montgomery should be designed using a value of 0.012. Design charts based on Manning's equation for concrete pipes are presented by the American Concrete Pipe Association (1978). As an additional aid, the nomographs presented as Figures 7-8 to 7-11 can be used in combination with Table 7-10 to solve Manning's equation.

Given the appropriate peak runoff rate for the design point in question, the conduit is sized to carry this peak flow as an open channel using Manning's equation. In general, conduits sized using Manning's equation will have an ultimate capacity greater than the design capacity since a hydraulic head, or pressure, may develop at the manholes.

5.3.2 Pressure Flow. Although stormwater conduits are designed based on open-channel flow, they do not operate as open channels during the design event; rather, they function as a system of interconnected culverts. Hence, storm drains can be analyzed using the nomographs for culvert flow presented in Section 3.0 of this chapter. Such an analysis can be performed to determine the water surface elevations at critical points such as manholes and drop inlets.

The designer should begin with the outlet conduit and proceed upstream. The headwater depth at the entrance of the outlet conduit is computed by both inlet and outlet control computations. The larger value governs. The elevation of the headwater of the outlet conduit defines the water surface elevation in the manhole drained by the outlet conduit. This water surface elevation also defines the tailwater elevation of those conduits which drain into the manhole. Given this tailwater and the size and flow rate of the upstream conduits, their headwater elevations may be computed. This process is repeated until the water surface elevation at each manhole and drop inlet has been determined.

The calculated water surface elevation must be at least 1 foot below the intake of drop inlets and at least 2 feet below the top of manhole covers. If these criteria are not met, appropriate adjustments in conduit size and/or structure depth should be made until the water surface elevations are within allowable limits.

5.4 Design Procedure

The material presented in the above discussion can be summarized as a nine-step storm sewer design procedure. These nine steps are identified as follows:

1. Determine inlet locations and geometric layout of system.
2. Compute inlet intercepts.
3. Size the inlet outfall conduits (open channel flow).
4. Proceed to the most upstream design point in the system.
5. Route to the design point all inlet hydrographs upstream of the design point.
6. Sum these hydrographs to determine the design hydrograph.
7. Size the outfall conduit at the design point (open channel flow).
8. Move to the next downstream design point in the system and repeat Steps 5, 6, and 7. Continue until all conduits in the system are sized.
9. Analyze the system, based on culvert flow, in order to determine the water surface elevation at critical points.

Example 7-12 at the end of this chapter illustrates portions of this storm sewer design procedure and compares the Summation of Flows, Rational, and Inlet Hydrograph Methods.

5.5 Example Problems

Example 7-11. Hydraulic Sizing of Concrete Arch Pipe

Given the following data, size a circular concrete pipe, and then determine the hydraulically equivalent size of concrete arch pipe.

Q_{25} = 140 cfs
n = 0.012 (Table 7-2)
Maximum allowable slope = 0.0045 ft/ft

1. Try a 60-inch-diameter circular pipe.

$A = 19.635\ ft^2$

$R = 1.250$ feet

Using equation 7-12,

$$K = \frac{(1.49)}{(0.012)}(19.635)(1.25)^{2/3} = 2,829$$

Using equation 7-13

$$S = \left[\frac{140}{2829}\right]^2 = 0.0024\ ft/ft$$

Try a 54-inch-diameter circular pipe.

$A = 15.904\ ft^2$

$R = 1.125$ feet

$$K = \frac{(1.49)}{(0.012)}(15.904)(1.125)^{2/3} = 2,136$$

$$S = \left[\frac{140}{2,136}\right]^2 = 0.0043\ ft/ft$$

2. Find the size of concrete arch pipe which is hydraulically equivalent to this 54-inch circular pipe.

An area equivalent to the 54-inch circular pipe is obtained with a 40-inch by 65-inch arch pipe.

$A = 14.3\ ft^2$

$R = 1.01$ feet

$$K = \frac{(1.49)}{(0.012)}(14.3)(1.01)^{2/3} = 1,787$$

$$S = \left[\frac{140}{1,787}\right]^2 = 0.006\ ft/ft$$

This exceeds maximum slope.

Try a 45-inch by 73-inch arch pipe.

$A = 17.7\ ft^2$

$R = 1.13$ feet

$$K = \frac{(1.49)}{(0.012)}(17.7)(1.13)^{2/3} = 2,384$$

$$S = \left[\frac{140}{2,384}\right]^2 = 0.0034\ ft/ft$$

Therefore, the 45-inch by 73-inch arch pipe is hydraulically equivalent to 54-inch circular pipe, for conditions of this problem.

Example 7-12. Storm Sewer System Design by Three Methods

Given the geometric storm sewer system layout shown on Figure 7-82, and the inlet area data given in the table which follows:

Inlet	Drainage Area (acres)	t_c (min)	Duration of Storm (min)	I (in/hr)	Runoff C	Q_i (cfs)
1	2.0	3.5	5	6.8	.8	10.9
2	3.0	4.0	5	6.8	.6	12.2
3	2.5	3.8	5	6.8	.7	11.9
4	2.5	3.8	5	6.8	.4	6.8
5	2.0	3.5	5	6.8	.5	6.8
6	2.5	3.8	5	6.8	.6	10.2
7	2.0	3.5	5	6.8	.7	9.5

Note: The data considered for this example are not for a sewer system in Montgomery.

Assume all runoff originating in a given inlet area is intercepted by its respective inlet. Therefore, there is no bypass and the discharges given in the above table are the inlet flows.

Find the pipe sizes required for each component of the system by each of the following design methods.

A. Summation of Flows Method
B. Rational Method
C. Inlet Hydrograph Method

A. Summation of Flows Method: This method is the simplest and most conservative of the three methods investigated. By this method, the conduits are sized to carry the summation of flows originating at all inlets upstream of the design point. The sizes are selected by use of open channel flow charts developed for concrete pipe (n = 0.013). The design procedure is illustrated in the table which follows:

SUMMATION OF FLOWS METHOD DESIGN

Conduit	Design Flow Rate (cfs)	Size Required (inches)	Length (feet)
I_1M_1	10.9	18	20
I_2M_2	12.2	21	130
M_1M_2	23.1	30	400
I_3M_2	11.9	21	20
I_4M_2	6.8	15	150
M_2M_3	41.8	36	700
I_5M_3	6.8	15	20
I_6M_3	10.2	18	150
M_3M_4	58.8	36	600
I_7M_4	9.5	18	170
M_4O	68.3	42	200

B. Rational Method: This method consists of computing a design flow rate based on equation 5-5. A new design storm duration and intensity is derived for each downstream component of the system. The new storm duration is computed as the sum of the time for each area and the travel time in the upstream portion of the storm sewer system according to equation 5-9.

Inlet conduits (i.e., I_1M_1 through I_7M_4) are sized to carry the full inlet flow. These design flow rates and the resultant conduit sizes will be the same as for the Summation of Flows Method. Travel times in the inlet conduits were estimated from open channel charts and are given in the following table:

Pipe	Slope (%)	Length (feet)	Q_i, (cfs)	Size (inches)	Velocity (ft/sec)	Travel Time (minutes)
I_1M_1	1.0	20	10.9	18	6.6	.050
I_2M_1	0.8	130	12.2	21	6.5	.333
I_3M_2	0.8	20	11.9	21	6.5	.051
I_4M_2	1.0	150	6.8	15	6.0	.418
I_5M_3	1.0	20	6.8	15	6.0	.056
I_6M_3	0.9	150	10.2	18	6.3	.400
I_7M_4	1.0	170	9.5	18	6.3	.450

The design will proceed with the sizing of conduit M_1M_2. This pipe must be designed to carry the flow from inlets 1 and 2. The first step is to determine the design flow rate as follows:

Inlet	Area	t_I	t_3	$(t_I + t_3)$	t_c	$I_T(t_c)$	C_T	Q_T
1	2.0	3.5	.050	3.55	5.0	6.8	.8	10.9
2	3.0	4.0	.333	4.33	5.0	6.8	.6	12.2

Design Flow Rate = 23.1 cfs

Because the total time $(T_I + t_3)$ was less than 5 minutes, the design storm duration is still set at 5 minutes and the design flow rate is the same as for the Summation of Flows Method. Thus, the conduit size for pipe M_1M_2 will be the same as for the Summation of Flows Method (30 inches).

In order to proceed with the design of conduit M_2M_3, the travel time in conduit M_1M_2 is estimated as follows:

Pipe	Slope (%)	Length (feet)	Q_i, (cfs)	Size (inches)	Velocity (ft/sec)	Travel Time (minutes)
M_1M_2	0.7	400	23.1	30	7.3	.951

Conduit M_2M_3 is sized to carry the flows from inlets 1, 2, 3, and 4. This flow rate is determined as follows:

Inlet	Area (acres)	t_I (min)	t_3 (min)	(t_I+t_3) (min)	t_c (min)	$I_T(t_c)$ (in/hr)	Runoff C_T	Q_T
1	2.0	3.5	.965	4.47	5.3	6.6	.8	10.6
2	3.0	4.0	1.248	5.25	5.3	6.6	.6	11.9
3	2.5	3.8	.051	3.85	5.3	6.6	.7	11.6
4	2.5	3.8	.418	4.22	5.3	6.6	.4	6.6

Design Flow Rate = 40.7 cfs

The above design flow rate is less, but only slightly less, than the design flow rate obtained by the Summation of Flows Method. This difference in design flow rates is not enough to change the required conduit size. Therefore, pipe M_2M_3 is sized at 36 inches.

To design conduit M_3M_4, it is necessary to compute the travel time in pipe M_2M_3 as follows:

Pipe	Slope (%)	Length (feet)	Q_i, (cfs)	Size (inches)	Velocity (ft/sec)	Travel Time (minutes)
M_2M_3	0.7	700	40.7	36	8.5	1.373

The peak rate or runoff at manhole 3 may now be computed.

Inlet	Area (acres)	t_I (min)	t_3 (min)	(t_I+t_3) (min)	t_c (min)	$I_T(t_c)$ (in/hr)	Runoff C_T	Q_T
1	2.0	3.5	2.338	5.84	6.6	6.1	.8	9.8
2	3.0	4.0	2.621	6.62	6.6	6.1	.6	11.0
3	2.5	3.8	1.424	5.22	6.6	6.1	.7	10.7
4	2.5	3.8	1.791	5.59	6.6	6.1	.4	6.1
5	2.0	3.5	.056	3.56	6.6	6.1	.5	6.1
6	2.5	3.8	.400	4.20	6.6	6.1	.6	9.2

Design Flow Rate = 52.9 cfs

This flow rate is about a 10 percent reduction compared to that obtained by the Summation of Flows Method. However, the reduction is not sufficient to reduce the required pipe size, and 36-inch conduit is still required for pipe M_3M_4.

The travel time in pipe M_3M_4 is now estimated.

Pipe	Slope (%)	Length (feet)	Q_i, (cfs)	Size (inches)	Velocity (ft/sec)	Travel Time (minutes)
M_3M_4	0.8	600	52.9	36	9.8	1.020

The design flow rate for conduit M_4O may now be computed.

Inlet	Area (acres)	t_I (min)	t_3 (min)	(t_I+t_3) (min)	t_c (min)	$I_T(t_c)$ (in/hr)	Runoff C_T	Q_T
1	2.0	3.5	3.358	6.86	7.6	5.9	.8	9.4
2	3.0	4.0	3.641	7.64	7.6	5.9	.6	10.6
3	2.5	3.8	2.444	6.24	7.6	5.9	.7	10.3
4	2.5	3.8	2.811	6.61	7.6	5.9	.4	5.9
5	2.0	3.5	1.076	4.58	7.6	5.9	.5	5.9
6	2.0	3.8	1.420	5.22	7.6	5.9	.6	7.1
7	2.0	3.5	.450	3.95	7.6	5.9	.7	8.3

Design Flow Rate = 57.5 cfs

In this case, the design flow rate has been reduced approximately 15 percent below the design flow rate computed by the Summation of Flows Method. Also, the required pipe size is reduced from 42 inches to 36 inches. The resultant design by the Rational Method is given in the following table:

RATIONAL METHOD DESIGN

Conduit	Design Flow Rate	Size Required (inches)	Length (feet)
I_1M_1	10.9	18	20
I_2M_1	12.2	21	130
M_1M_2	23.1	30	400
I_3M_2	11.9	21	20
I_4M_2	6.8	15	150
M_2M_3	40.7	36	700
I_5M_3	6.8	15	20
I_6M_3	10.2	18	150
M_3M_4	52.9	36	600
I_7M_4	9.5	18	170
M_4O	57.5	36	200

C. Inlet Hydrograph Method: By this method, the duration of the design storm is held constant (5 minutes in this example), and the inlet hydrographs are routed through the storm sewer system to the design point. These routed inflow hydrographs are then added and the peak flow rate is taken from this summation. This peak flow rate is used as the design flow rate for the conduit under consideration. A detailed explanation of the method and the basic routing equation (7-34) is given in Subsection 5.2.3.

As with the Summation of Flows and the Rational Methods, the inlet conduits (I_1M_1 through I_7M_4) must be sized to carry the full inlet flows and are, therefore, the same size as in the two previous designs.

The design will begin with the sizing of conduit M_1M_2. The peak flow rates of the routed inlet hydrographs are computed as shown in the following table. These hydrographs are then plotted and summed as illustrated on Figure 7-83 in order to determine the design flow rate.

Inlet	Q_i (cfs)	2T (min)	t_3 (min)	$2T/(2T + .8T_t)$ (Equation 7-34) (min)	Q_o (cfs)
1	10.9	10	.050	.996	10.9
2	12.2	10	.333	.974	11.9

From Figure 7-83, it is determined that the peak flow rate in manhole 1 is 22.8 cfs. Pipe M_1M_2 is sized and the travel time estimated from the open channel charts as follows:

Pipe	Slope (%)	Q design (cfs)	Size (inches)	Velocity (ft/sec)	Length (feet)	t_3 (min)
M_1M_2	0.7	22.8	30	7.5	400	.890

Although the design discharge is slightly lower when computed by this method than when computed by the previous two methods, the required pipe size remains the same (30 inches).

The routed hydrographs at manhole 2 are now computed in order to size conduit M_2M_3.

Inlet	Q_i (cfs)	2T (min)	t_3 (min)	$2T/(2T + .8T_t)$ (Equation 7-34) (min)	Q_o (cfs)
1	10.9	10	.940	.930	10.1
2	12.2	10	1.223	.911	11.1
3	11.9	10	.051	.996	11.9
4	6.8	10	.418	.968	6.6

From Figure 7-84, the design flow rate is determined to be 37.0 cfs. Conduit M_2M_3 is sized and the travel time estimated as follows:

Pipe	Slope (%)	Q design (cfs)	Size (inches)	Velocity (ft/sec)	Length (feet)	t_3 (min)
M_2M_3	0.7	37.0	30	7.8	700	1.496

The above design flow rate is approximately 12 percent less than the flow rate given by the summation of flows method and has reduced the required pipe size from 36 inches to 30 inches.

The routed hydrographs at manhole 3 are now computed.

Inlet	Q_i (cfs)	2T (min)	t_3 (min)	$2T/(2T + .8T_t)$ (Equation 7-34) (min)	Q_o (cfs)
1	10.9	10	2.436	.837	9.1
2	12.2	10	2.719	.821	10.0
3	11.9	10	1.547	.890	10.6
4	6.8	10	1.914	.867	5.9
5	6.8	10	.056	.995	6.8
6	10.2	10	.400	.965	9.9

From Figure 7-85, the design flow rate is determined to be 46.5 cfs. Pipe M_3M_4 is sized and the travel time estimated based on the above flow rate as follows:

Pipe	Slope (%)	Q design (cfs)	Size (inches)	Velocity (ft/sec)	Length (feet)	t_3 (min)
M_3M_4	0.8	46.5	36	9.0	600	1.111

The above design flow rate is approximately 21 percent less than the design flow rate obtained by the Summation of Flows Method and 12 percent less than the design flow rate obtained by the Rational Method. However, in this case, the pipe selected is 36 inches by all three design methods.

The routed hydrographs at manhole 4 are computed as follows:

Inlet	Q_i (cfs)	2T (min)	t_3 (min)	$2T/(2T + .8T_t)$ (Equation 7-34) (min)	Q_o (cfs)
1	10.9	10	3.547	.779	8.5
2	12.2	10	3.830	.765	9.3
3	11.9	10	2.658	.825	9.8
4	6.8	10	3.025	.805	5.5
5	6.8	10	1.167	.915	6.2
6	10.2	10	1.511	.892	9.1
7	9.5	10	0.450	.965	9.2

The design discharge from Figure 7-86 is 50 cfs, which is a 27 percent reduction in design flow rate compared to that obtained by the Summation of Flows Method and a 13 percent reduction in design flow rate compared to that obtained by the Rational Method. Conduit M_4O is sized as follows:

Pipe	Slope (%)	Q design (cfs)	Size (inches)	Velocity (ft/sec)	Length (feet)	t_3 (min)
M_4O	0.6	50	36	8.7	200	1.383

The resultant pipe size is a reduction compared to that obtained by the Summation of Flows Method, but is the same as that obtained by the Rational Method of storm sewer design.

The Inlet Method design for the subject storm sewer system is summarized in the following table.

INLET HYDROGRAPH METHOD DESIGN

Conduit	Design Flow Rate (cfs)	Size Required (inches)	Length (feet)
I_1M_1	10.9	18	20
I_2M_2	12.2	21	130
M_1M_2	22.8	30	400
I_3M_2	11.9	21	20
I_4M_2	6.8	15	150
M_2M_3	37.0	30	700
I_5M_3	6.8	15	20
I_6M_3	10.2	18	150
M_3M_4	46.5	36	600
I_7M_4	9.5	18	170
M_4O	50.0	36	200

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LIST OF SYMBOLS--CHAPTER 7

E = specific energy head, in feet

d = depth of open-channel flow, in feet

v = average channel velocity, in ft/sec, defined as the discharge divided by the area of flow, derived from equation 7-2

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

Q = discharge, in cfs

cfs = cubic feet per second

fps = feet per second

A = cross-sectional area of open channel, in ft^2

d_1 and d_2 = depth of open channel channel flow at channel sections 1 and 2, respectively, in feet

v_1 and v_2 = average channel velocities at channel sections 1 and 2, respectively, in ft/sec

z_1 and z_2 = channel elevations above an arbitrary datum at channel sections 1 and 2, respectively, in ft

h_{loss} = head or energy loss between channel sections 1 and 2

v_c = critical velocity, in ft/sec

d_m = mean depth of flow, in feet,

n = Manning's roughness coefficient

R = hydraulic radius of the channel, in feet

S = slope of the energy gradient, in feet per foot

P = wetted perimeter of the channel cross section, in feet

K = conveyance of a given open channel

k = median stone diameter, in feet

v_s = average velocity against stone lining, in fps

H = total head, or the elevation difference between the headwater (HW) and tailwater (TW), in feet

K_e = culvert entrance loss coefficient for a given inlet

L = length of a culvert barrel, in feet

K_b = culvert bend loss coefficient

Δ = the angle of a culvert bend, in degrees

HW = headwater depth, in feet

D = the height or diameter of a culvert, in feet

h_o = the head at a culvert outlet, in feet, for a nonsubmerged outlet condition

d_c = the critical depth at the culvert outlet, in feet

DTW = design tailwater for a culvert, in feet

TW = tailwater depth for a culvert, in feet

Q_g = gutter flow rate, in cfs

Z = reciprocal of the gutter cross slope, in feet/foot

S_g = gutter longitudinal slope, in feet/foot

Q_i = inlet intercept rate, in cfs

h = height of curb inlet opening, in feet

A_i = waterway area of a curb inlet opening, in ft^2

Q_e = intercept of an efficient grate inlet, in cfs

w = width of a grate inlet, in feet

$\bar{d}$ = average depth of flow approaching a grate inlet measured at the gate centerline

L_o = length of grate required to obtain efficient operation, in feet

m = grate inlet constant which depends on the type of grate inlet

q = that portion of stormwater flowing within the limits of a grate inlet which is bypassed, in cfs

L = actual length of a grate inlet, in feet

p = the effective perimeter of a grate inlet, in feet

Q_o = peak flow rate of an outflow hydrograph, in cfs

d_s = diameter of closed stormwater conduit, in feet

S_o = pipe slope, in feet per foot

Table 7-1
CRITICAL DEPTH EQUATIONS FOR VARIOUS
CHANNEL CROSS SECTIONS

Type of Channel Cross Section	Critical Depth Equation
Rectangular[a]	$d_c = 0.315 \left(\frac{Q}{B}\right)^{2/3}$
Trapezoidal[b]	$d_c = \frac{4zE - 3b + \sqrt{16z^2E^2 + 16zEb + 9b^2}}{10z}$
Triangular	$d_c = 0.574 \left(\frac{Q}{Z}\right)^{2/5}$
Circular[c]	$d_c = 0.325 \left(\frac{Q}{D}\right)^{2/3} + 0.083D$
Irregular	$\frac{A^3}{T} = \frac{Q^2}{g}$

where

A = cross-sectional area of flow, in ft^2.

B = width of a rectangular channel, in ft.

b = bottom width of a trapezoidal channel, in feet.

D = diameter of circular conduit, in feet.

g = acceleration of gravity, 32.2 ft/sec^2.

E = specific head in section, in feet (Equation 7-1).

Q = design discharge, in cfs.

T = top width of water surface, in feet.

v = mean velocity of flow, in ft^2/sec.

z = side slopes of a channel (horizontal to vertical).

Source: Federal Highway Administration (1965) Hydraulic Design Series No. 4.

[a]Critical depth can also be determined using Figures 7-12 to 7-25 in this manual, or from design charts presented by the SCS (1950) in NEH Section 5 "Hydraulics."

[b]Critical depth can also be determined using Figures 7-26 to 7-39 in this manual or from design charts presented by the SCS (1950) in NEH Section 5 "Hydraulics."

[c]Accurate only when d_c/D lies between 0.3 and 0.9

Table 7-2
MANNING'S n VALUES FOR CONVEYANCE
SYSTEM DESIGN IN MONTGOMERY

Type of Channel	Manning's n Value
Concrete Pipe	0.012
Concrete Channels (Troweled Bottom Broomed Side Slopes)	0.015
Natural or Excavated Channels	Use Cowans Equation Below and Table 7-3

Cowans Equation[1]: $n = (n_0 + n_1 + n_2 + n_3 + n_4)m_5$

[1]From Chow (1959).

Table 7-3
COEFFICIENTS FOR COMPUTING MANNING'S n
FOR NATURAL OR EXCAVATED CHANNELS USING COWANS EQUATION[1]

Channel Conditions		Values[2]	
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 occasionally		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	m_5	1.000
	Appreciable		1.150
	Severe		1.300

[1]Cowans equation is presented in Table 7-2.
[2]From Chow (1959).

Table 7-4
MANNING'S n VALUES FOR CLOSED CONDUITS[a]

Type of Closed Conduit	Range of Manning's n[b]
A. Concrete pipe	0.011-0.013
B. Corrugated-metal pipe or pipe-arch:	
1. 2-2/3 by 1/2-in. corrugation (riveted pipe):[c]	
a. Plain or fully coated	0.024
b. Paved invert (range values are for 25 and 50 percent of circumference paved):	
(1) Flow full depth	0.021-0.018
(2) Flow 0.8 depth	0.021-0.016
(3) Flow 0.6 depth	0.019-0.013
2. 6 by 2-inch corrugation (field bolted)	0.03
C. Vitrified clay pipe	0.012-0.014
D. Cast-iron, uncoated	0.013
E. Steel pipe	0.009-0.011
F. Brick	0.014-0.017
G. Monolithic concrete	
1. Wood forms, rough	0.015-0.017
2. Wood forms, smooth	0.012-0.013
3. Steel forms	0.012-0.013
H. Cemented rubble masonry walls	
1. Concrete floor and top	0.017-0.022
2. Natural floor	0.019-0.025
I. Laminated treated wood	0.015-0.017
J. Vitrified clay liner plates	0.015

Source: Federal Highway Administration (1961)

[a]Estimates are by the Federal Highway Administration unless other wise noted.

[b]Ranges indicated are for good to fair construction. For poor quality construction use larger values of n.

[c]Friction Factors in Corrugated Metal Pipe, by M. J. Webster and L. R. Metcalf, Corps of Engineers, Department of the Army; published in Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, vol. 85, No. HY9, September 1959, Paper No. 2148, pp. 35-67.

Table 7-5
MANNING'S n VALUES FOR NONVEGETATIVE LINING OF OPEN CHANNELS[a]

Type of Lined Open Channel	Range of Manning's n[b]
A. Concrete, with surfaces as indicated	
1. Formed, no finish	0.013-0.017
2. Trowel finish	0.012-0.014
3. Float finish	0.013-0.014
4. Float finish, some gravel on bottom	0.015-0.017
5. Gunite, good section	0.016-0.019
6. Gunite, wavy section	0.018-0.022
B. Concrete, bottom float finished, sides as indicated:	
1. Dressed stone in mortar	0.015-0.017
2. Random stone in mortar	0.017-0.020
3. Cement rubble masonry	0.020-0.025
4. Cement rubble masonry, plastered	0.016-0.020
5. Dry rubble (riprap)	0.020-0.030
C. Gravel bottom, sides as indicated:	
1. Formed concrete	0.017-0.020
2. Random stone in mortar	0.020-0.023
3. Dry rubble (riprap)	0.023-0.033
D. Brick	0.014-0.017
E. Asphalt:	
1. Smooth	0.013
2. Rough	0.016
F. Wood, planed, clean	0.011-0.013
G. Concrete-lined excavated rock:	
1. Good section	0.017-0.020
2. Irregular section	0.022-0.027

Source: Federal Highway Administration (1961)

[a]Estimates are by the Federal Highway Administration.

[b]Ranges indicated are for good to fair construction. For poor quality construction use larger values of n.

Table 7-6
MANNING'S n VALUES FOR STREET AND EXPRESSWAY GUTTERS

Type of Gutter	Range of Manning's n[a]
A. Concrete gutter, troweled finish	0.012
B. Asphalt pavement:	
1. Smooth texture	0.013
2. Rough texture	0.016
C. Concrete gutter with asphalt pavement:	
1. Smooth	0.013
2. Rough	0.015
D. Concrete pavement:	
1. Float finish	0.014
2. Broom finish	0.016
E. For gutters with small slope, where sediment may accumulate, increase above values of n by	0.002

Source: Federal Highway Administration (1961)

[a]Estimates are by the Federal Highway Administration

Table 7-7

MANNING'S n VALUES FOR EXCAVATED OPEN CHANNELS[a]
(STRAIGHT ALINEMENT,[b] AND NATURAL LINING)

Type of Excavated Open Channel	Range of Manning's n[c]
A. Earth, uniform section:	
1. Clean, recently completed	0.016-0.018
2. Clean, after weathering	0.018-0.020
3. With short grass, few weeds	0.022-0.027
4. In gravelly soil, uniform section, clean	0.022-0.025
B. Earth fairly uniform section:	
1. No vegetation	0.022-0.025
2. Grass, some weeds	0.025-0.030
3. Dense weeds or aquatic plants in deep channels	0.030-0.035
4. Sides clean, gravel bottom	0.025-0.030
5. Sides clean, cobble bottom	0.030-0.040
C. Dragline excavated or dredged:	
1. No vegetation	0.028-0.033
2. Light brush on banks	0.035-0.050
D. Rock:	
1. Based on design section	0.035
2. Based on actual mean section:	
a. Smooth and uniform	0.035-0.040
b. Jagged and irregular	0.040-0.045
E. Channels not maintained, weeds and brush uncut:	
1. Dense weeds, high as flow depth	0.08-0.12
2. Clean bottom, brush on sides	0.05-0.08
3. Clean bottom, brush on sides, highest stage of flow	0.07-0.11
4. Dense brush, high stage	0.10-0.14

Source: Federal Highway Administration (1961)

[a]Estimates are by the Federal Highway Administration. For important work and where the accurate determination of water profiles is necessary, the designer is urged to consult the followin references and to select n by comparison of the specific conditions iwt the channels tested: Flow of Water in Irrigation and Similar Channels, by F. C. Scobey, Division of Irrigation Soil Conservation Service, U.S. Department of Agriculture, Tech. Bull. No. 652, Feb. 1939 and Flow of water in Drainage Channels, by C. E. Ramser, Division of Agricultural Engineering, Bureau of Public Roads, U.S. Department of Agriculture, Tech. Bull. No. 129, Nov. 1929.

[b]With channel of an alinement other than straight, loss of head by resistance forces will be increased. A small increase in value of n may be made, to allow for the additional loss of energy.

[c]Ranges indicated are for good to fair construction. For poor quality construction use larger values of n.

Table 7-8
MANNING'S n VALUES FOR NATURAL STREAM CHANNELS[a]

Type of Natural Channel	Range of Manning's n[b]
A. Minor streams[c] (surface width at flood stage less than 100 ft	
1. Fairly regular section:	
a. Some grass and weeds, little or no brush	0.030-0.35
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035-0.05
c. Some weeds, light brush on banks	0.035-0.05
d. Some weeds, heavy brush on banks	0.05-0.07
e. Some weeds, dense willows on banks	0.06-0.08
f. For tress within channel, with branches submerged at high stage, increase all above balues by	0.01-0.02
2. Irregular sections, with pools, slight channel meander; increase values given in 1a-e about	0.01-0.02
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.04-0.05
b. Bottom of cobbles, with large boulders	0.05-0.07
B. Flood plains (adjacent to natural streams):	
1. Pasture, no brush:	
a. Short grass	0.030-0.035
b. High grass	0.035-0.05
2. Cultivated areas:	
a. No crop	0.03-0.04
b. Mature row crops	0.035-0.045
c. Mature field crops	0.04-0.05
3. Heavy weeds, scattered brush	0.05-0.07
4. Light brush and trees[d]	
a. Winter	0.05-0.06
b. Summer	0.06-0.08
5. Medium to dense brush[d]	
a. Winter	0.07-0.11
b. Summer	0.10-0.16
6. Dense willows, summer, not bent over by current	0.15-0.20
7. Cleared land with tree stumps, 100-150 per acre	
a. No sprouts	0.04-0.05
b. With heavy growth of sprouts	0.06-0.08
8. Heavy stand of timber, a few down trees, little undergrowth	
a. Flood depth below branches	0.10-0.12
b. Flood depth reaches branches	0.12-0.16

Table 7-8--Continued

Type of Natural Channel	Range of Manning's n[b]
C. Major streams (surface width at flood stage more than 100 ft): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Vaues of n may be somewhat reduced. Follow recommendation in publication cited if possible. The value of n for larger streams of most regular section, with no boulders or brush, may be in the range of	0.028-0.33

Source: Federal Highway Administration (1961)

[a]The tabulated values for n have been derived from data reported by C. E. Ramser (1929) and from other incomplete data.

[b]For calculation of stage or discharge in a natural stream, it is recommended that the designer consult the local District Office of the Surface Water Branch of the U.S. Geological Survey to obtain data regrading values of n applicable to streams in the Montgomery area. Where this procedure is not followed, the table may be used as a guide.

[c]The tentative values of n cited are principally derived from measurements made on fairly short but straight reaches of natural streams. Where slopes calculated from flood elevations along a considerable length of channel, involving meanders and bends, are to be used in velocity calculations by the Manning formula, the value of n must be increased to provide for the additional loss of energy caused by bends. The increase may be in the range of perhaps 3 to 15 percent.

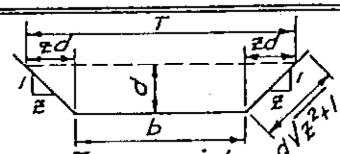
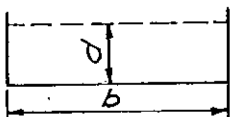
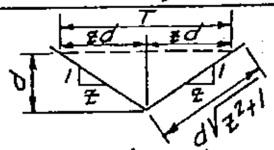
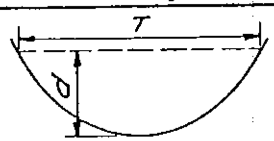
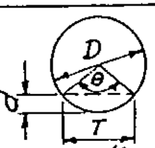
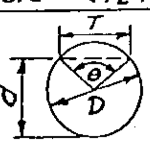
[d]The presence of foliage on trees and brush under flood stage will materially increase the vaule of n. Therefore, roughness coefficients for vegetation in leaf will be larger than for bare branches. For tress in channels or on banks, and for brush on banks where submergence of branches increases with depth of flow, n will increase with rising stage.

Table 7-9
CLASSIFICATION OF VEGETAL COVERS AS TO DEGREE OF RETARDANCE

Retardance	Cover	Condition
A	Weeping lovegrass	Excellent stand, tall, (average 30 inches)
	Yellow bluestem Ischaemum	Do , tall, (average 30 inches)
B	Kudzu	Very dense growth, uncut
	Bermudagrass	Good stand, tall, (avg. 12 inches)
	Native grass mixture (little bluestem, blue grama, and other long and short midwest grasses)	Good stand, unmowed
	Weeping lovegrass	Good stand, tall, (average 24 in)
	Lespedeza sericea	Good stand, not woody, tall (average 19 inches)
	Alfalfa	Good stand, uncut, (average 11 in)
	Weeping love grass	Good stand, mowed, (average 13 in)
	Kudzu	Dense growth, uncut
	Blue grama	Good stand, uncut, (average 13 in)
C	Crabgass	Fair stand, uncut (10 to 48 inches)
	Bermudagrass	Good stand, mowed (average 6 in)
	Common lespedeza	Good stand, uncut (average 11 in)
	Grass-legume mixture--summer (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, headed (6 to 12 in)
	Centipedegrass	Very dense cover (average 6 in)
	Kentucky bluegrass	Good stand, headed (6 to 12 in)
D	Bermudagrass	Grand stand, cut to 2.5-inch height
	Common lespedeza	Excellent stand, uncut (average 4.5 inches)
	Buffalograss	Good stand, uncut (3 to 6 inches)
	Grass-legume mixture--fall spring (Orchardgrass, red-top, Italian ryegrass, and common lespedeza)	Good stand, uncut (4 to 5 inches)
	Lespedeza sericea	After cutting t 2-inch height. Very good stand before cutting.
E	Bermudagrass	Good stand, cut to 1.5 inches height
	Bermudagrass	Burned stubble.

Source: SCS TP-61 (1947)

Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform

Section	Area a	Wetted Perimeter p	Hydraulic Radius r	Top Width T
 Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$\dfrac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$
 Rectangle	bd	$b+2d$	$\dfrac{bd}{b+2d}$	b
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\dfrac{zd}{2\sqrt{z^2+1}}$	$2zd$
 Parabola	$\frac{2}{3}dT$	$T+\dfrac{8d^2}{3T}$ [1]	$\dfrac{2dT^2}{3T^2+8d^2}$ [1]	$\dfrac{3a}{2d}$
 Circle - <½ full [2]	$\dfrac{D^2}{8}\left(\dfrac{\pi\theta}{180}-\sin\theta\right)$	$\dfrac{\pi D\theta}{360}$	$\dfrac{45D}{\pi\theta}\left(\dfrac{\pi\theta}{180}-\sin\theta\right)$	$D\sin\dfrac{\theta}{2}$ or $2\sqrt{d(D-d)}$
 Circle - >½ full [3]	$\dfrac{D^2}{8}\left(2\pi-\dfrac{\pi\theta}{180}+\sin\theta\right)$	$\dfrac{\pi D(360-\theta)}{360}$	$\dfrac{45D}{\pi(360-\theta)}\left(2\pi-\dfrac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\dfrac{\theta}{2}$ or $2\sqrt{d(D-d)}$

[1] Satisfactory approximation for the interval $0<\frac{d}{T}\leqq 0.25$
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

TABLE 7-10. Open-channel characteristics of various cross-sections.

Table 7-11
GUIDE TO SELECTION OF VEGETAL RETARDANCE

Stand	Average Length Of Vegetation	Degree Of Retardance
Good	Longer than 30"	A
	11 to 24"	B
	6 to 10"	C
	2 to 6"	D
	Less than 2"	E
Fair	Longer than 30"	B
	11 to 24"	C
	6 to 10"	D
	2 to 6"	D
	Less than 2"	E

Source: SCS TP-61 (1947)

Table 7-12
PERMISSIBLE VELOCITIES FOR CHANNELS LINED WITH UNIFORM STANDS OF VARIOUS GRASS COVERS, WELL MAINTAINED[a]

Cover	Slope Range[b]	Permissible Velocity: Erosion Resistant Soils (ft/sec)	Permissible Velocity: Easily Eroded Soils (ft/sec)
Bermudagrass	0 to 5	8	6
	5 to 10	7	5
	Over 10	6	4
Buffalograss Kentucky bluegrass Smooth brome Blue grama	0 to 5	7	5
	5 to 10	6	4
	Over 10	5	3
Grass mixture	0 to 5[b]	5	4
	5 to 10	4	3
Lespedeza sericea Weeping lovegrass Yellow bluestem Kudzu Alfalfa	0 to 5[c]	3.5	2.5
Crabgrass Common lespedeza[d] Sudangrass[d]	0 to 5[e]	3.5	2.5

Source: Handbook of Channel Design for Soil and Water Conservation SCS-TP (1947).

[a] Use velocities exceeding 5 feet per second only where good covers and proper maintenance can be obtained.

[b] Do not use on slopes steeper than 10 percent except for side slopes in a combination channel.

[c] Do not use on slopes steeper than 5 percent except for side slopes in a combination channel.

[d] Annuals--used on mild slopes or as temporary protection until permanent covers are established.

[e] Use on slopes steeper than 5 percent is not recommended.

Table 7-13
PERMISSIBLE VELOCITIES FOR CHANNELS WITH ERODIBLE LININGS[a]
BASED ON UNIFORM FLOW IN CONTINUOUSLY WET, AGED CHANNELS[a]

Soil Type or Lining (earth, no vegetation)	Manning's Roughness Coefficient[b] n	Maximum Permissible Velocities for: Clear (ft/sec)	Water Carrying Fine Silts (ft/sec)	Water Carrying Sand and Gravel (ft/sec)
Fine sand (noncolloidal)	0.020	1.5	2.5	1.5
Sandy loam (noncolloidal)	0.020	1.7	2.5	2.0
Silt loam (noncolloidal)	0.020	2.0	3.0	2.0
Ordinary firm loam	0.020	2.5	3.5	2.2
Volcanic ash	0.020	2.5	3.5	2.0
Fine gravel	0.020	2.5	5.0	3.7
Stiff clay (very colloidal)	0.025	3.7	5.0	3.0
Graded, loam to cobbles (noncolloidal)	0.030	3.7	5.0	5.0
Graded, silt to cobbles (colloidal)	0.030	4.0	5.5	5.0
Alluvial silts (noncolloidal)	0.020	2.0	3.5	2.0
Alluvial silts (colloidal)	0.025	3.7	5.0	3.0
Coarse gravel (noncolloidal)	0.025	4.0	6.0	6.5
Cobbles and shingles	0.035	5.0	5.5	6.5
Shales and hard pans	0.025	6.0	6.0	5.0

Source: Handbook of Channel Design for Soil and Water Conservation, SCS-TP-64 (1947).

[a] Although not specifically stated in the original recommendations by the special committee on irrigation research, American Society of Civil Engineers, 1926, these values apply only to channels with mild slopes.
[b] From Lane (1955).

Table 7-14
CULVERT ENTRANCE LOSS COEFFICIENTS

Type of Structure and Design of 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[1]	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls	
Square-edge	0.5
Mitered to conform to fill slope	0.7
End-Section conforming to fill slope[1]	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension or beveled top edge	0.2
Wingwalls at 10° to 30° 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

Source: U.S. DOT HEC No. 5, (1961).

[1]Note: "End section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers.

Table 7-15
MAXIMUM ALLOWABLE GUTTER SPREAD FOR
VARIOUS STREET CLASSIFICATIONS

Classification	Definition	Maximum Allowable Spread[a]
Local	Minor traffic carrier within a neighborhood, usually characterized by two moving lanes and parking along curb. No through traffic from one neighborhood to another.	No curb over-topping,[b] flow may spread to crown of street.
Collector	Collect and distribute traffic between arterial and local streets. May be two to four moving lanes and parking along curb.	No curb over-topping[b] flow spread must leave at least one lane free of water.
Arterial	Permits rapid and relatively unimpeded traffic flow throughout a city. Four to six lanes are usual, and parking along curb may be prohibited. Construction will often include a median strip.	No curb over-topping,[b] flow spread must leave at least one lane free of water in each direction.
Freeway	Permits rapid and unimpeded traffic flow through and around a city. Access is completely controlled by interchanges. There may be up to eight lanes of traffic and parking is not permitted.	No encroachment is allowed on any traffic lane.

Source: Wright-McLaughlin Engineers (1969).

[a]The maximum allowable spread of gutter flow during the design storm on any street is 12 feet.

[b]Where no curbing exists, encroachment shall not extend over property lines.

Table 7-16
ALLOWABLE DEPTH OF GUTTER FLOW

Curb Height (inches)	Maximum Depth of Flow (feet)
3	0.20
6	0.40
8	0.45

Table 7-17
GRATE INLET m VALUES FOR DIFFERENT
TYPES OF INLETS

Inlet Type[a]	"m" Value
1	2.4 to 3.0
2	8
3	4.0 to 5.0
4	3.0
5	5.6
6	2.2 to 2.3

Source: Wycoff (1973).

[a]See Figure 7-75 for illustrations of the grate inlet types.

Table 7-18
REDUCTION FACTORS TO APPLY TO INLETS

Condition	Inlet Type	Percentage of Theoretical Capacity Allowed
Sump	Curb Opening	80
Sump	Grated	50
Sump	Combination	65
Continuous Grade	Curb Opening	80
Continuous Grade	Deflector	75
Continuous Grade	Longitudinal Bar Grated	60
Continuous Grade	Transverse Bar Grate or Longitudinal Bar Grate incorporating transverse bars	50
Continuous Grade	Combination	110% of that listed for type of grate utilized

Source: Wright-McLaughlin Engineers (1969).

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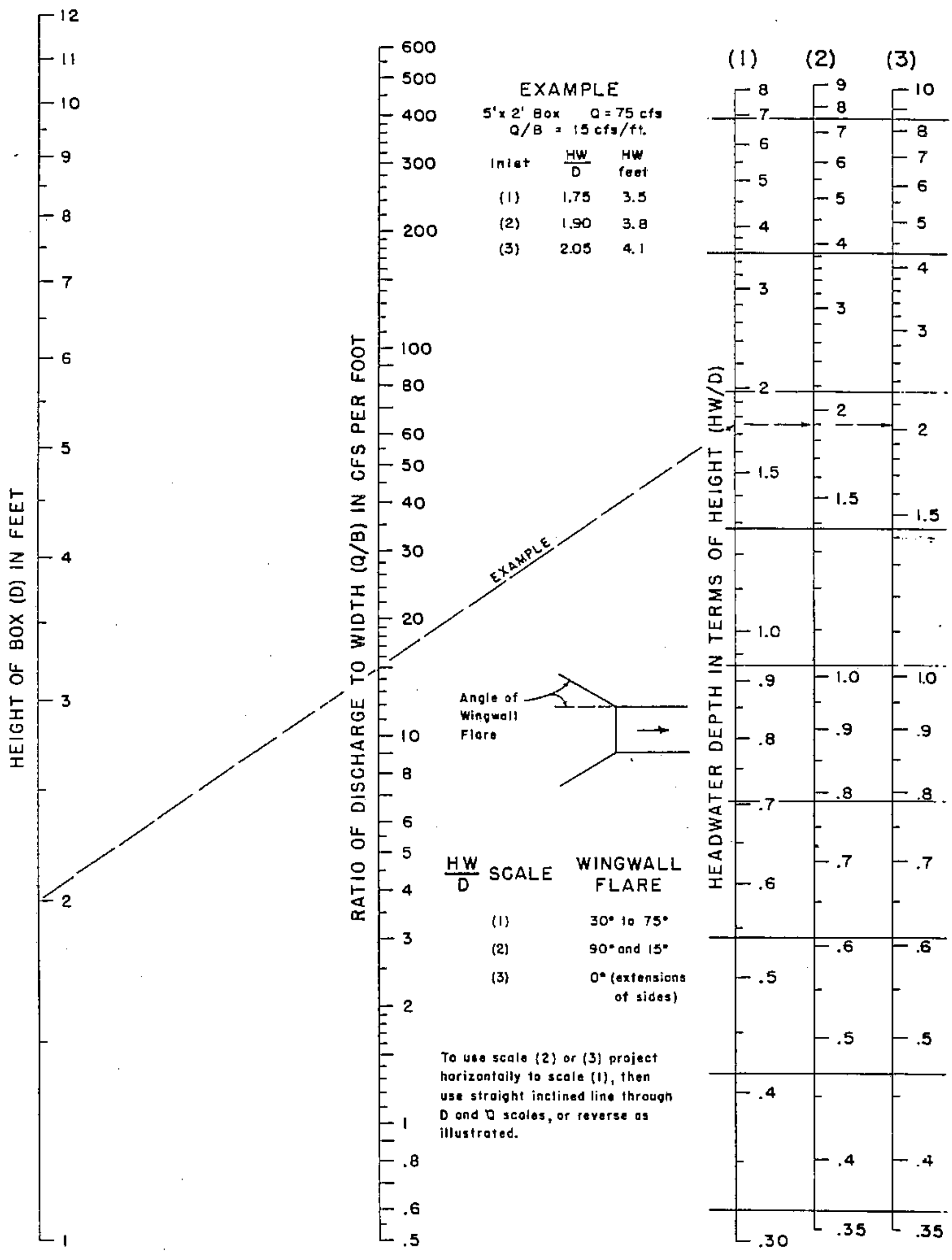

Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-50. Inlet control nomograph for box culverts .

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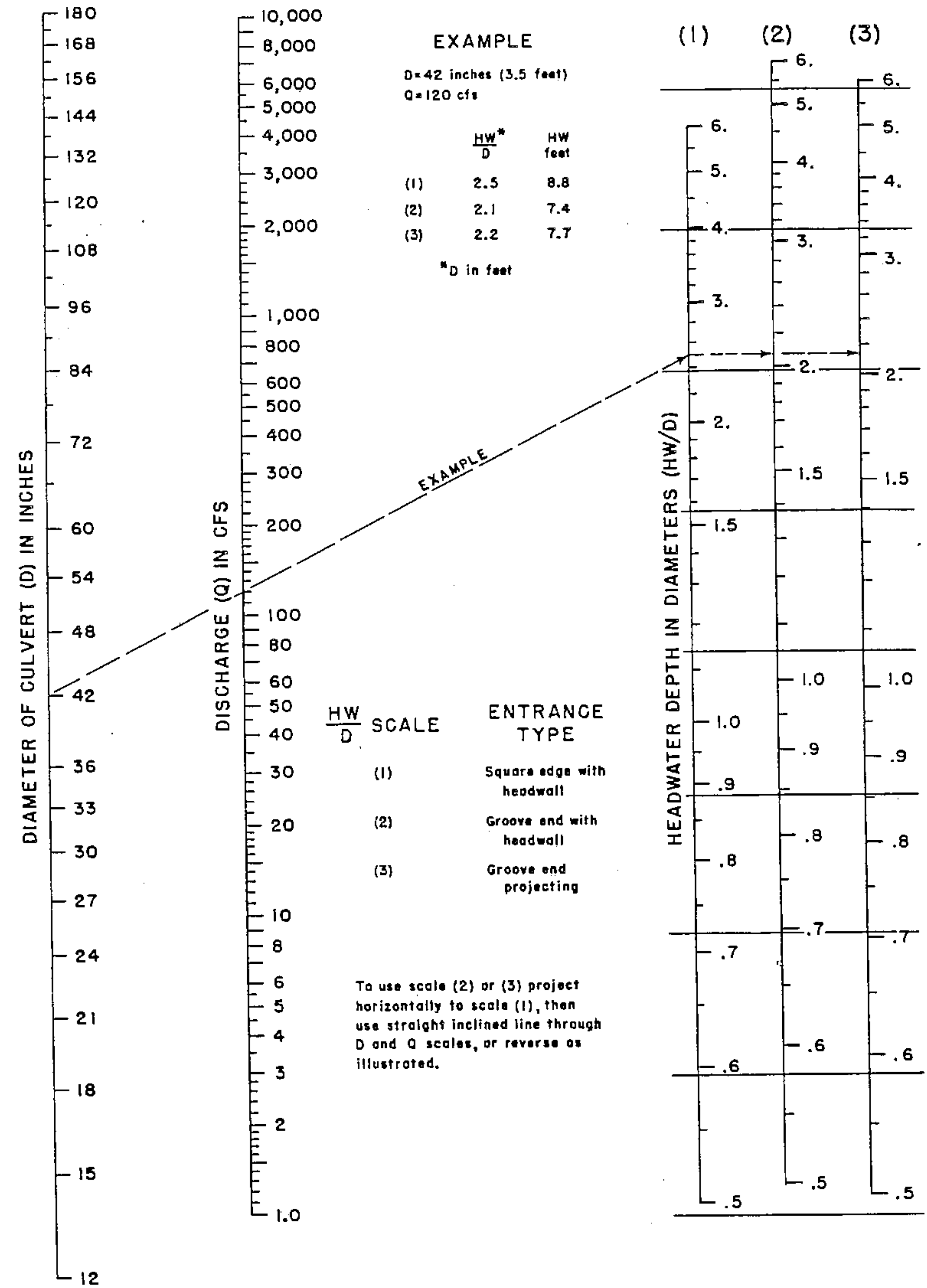
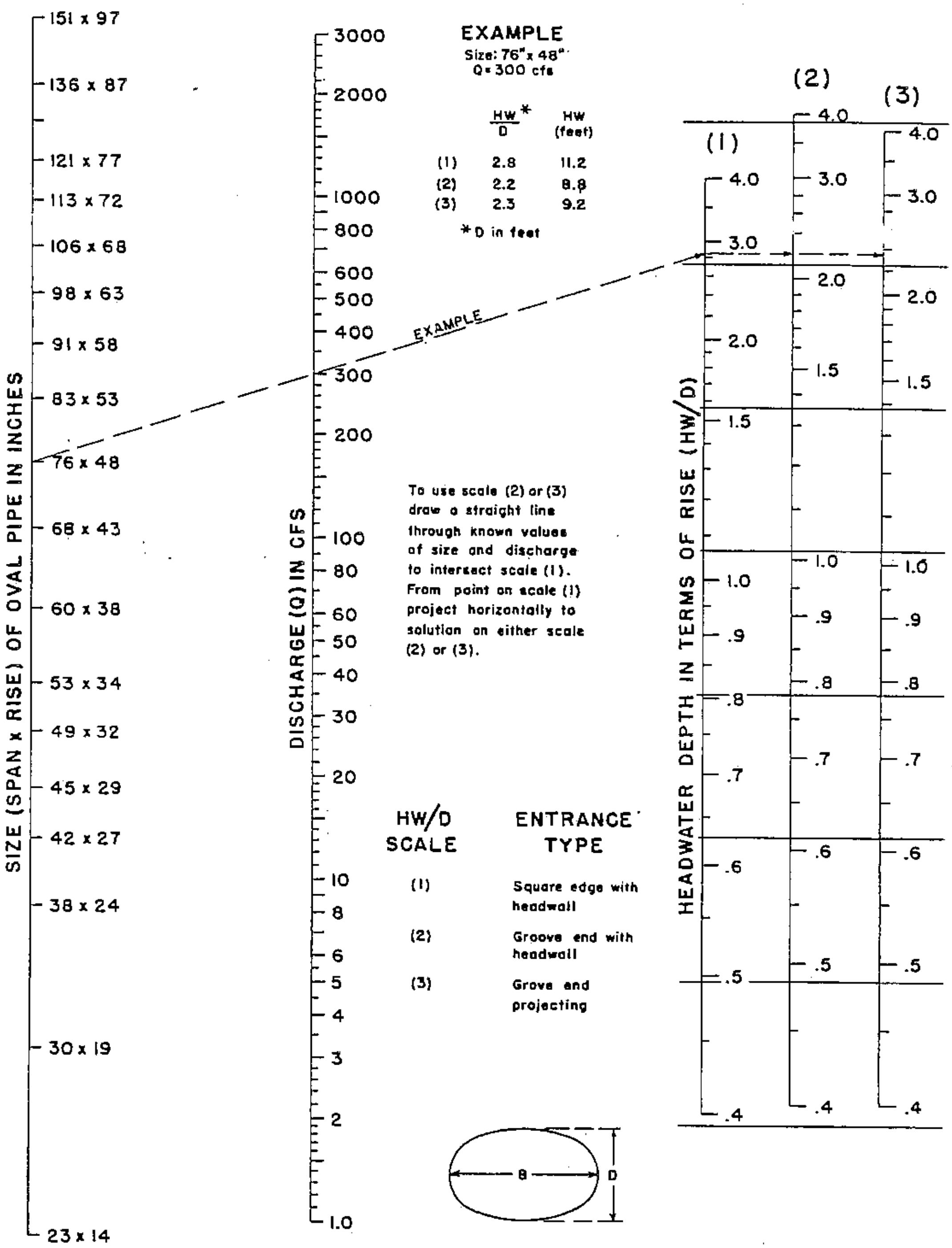


Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-51. Inlet control nomograph for concrete pipe culverts.

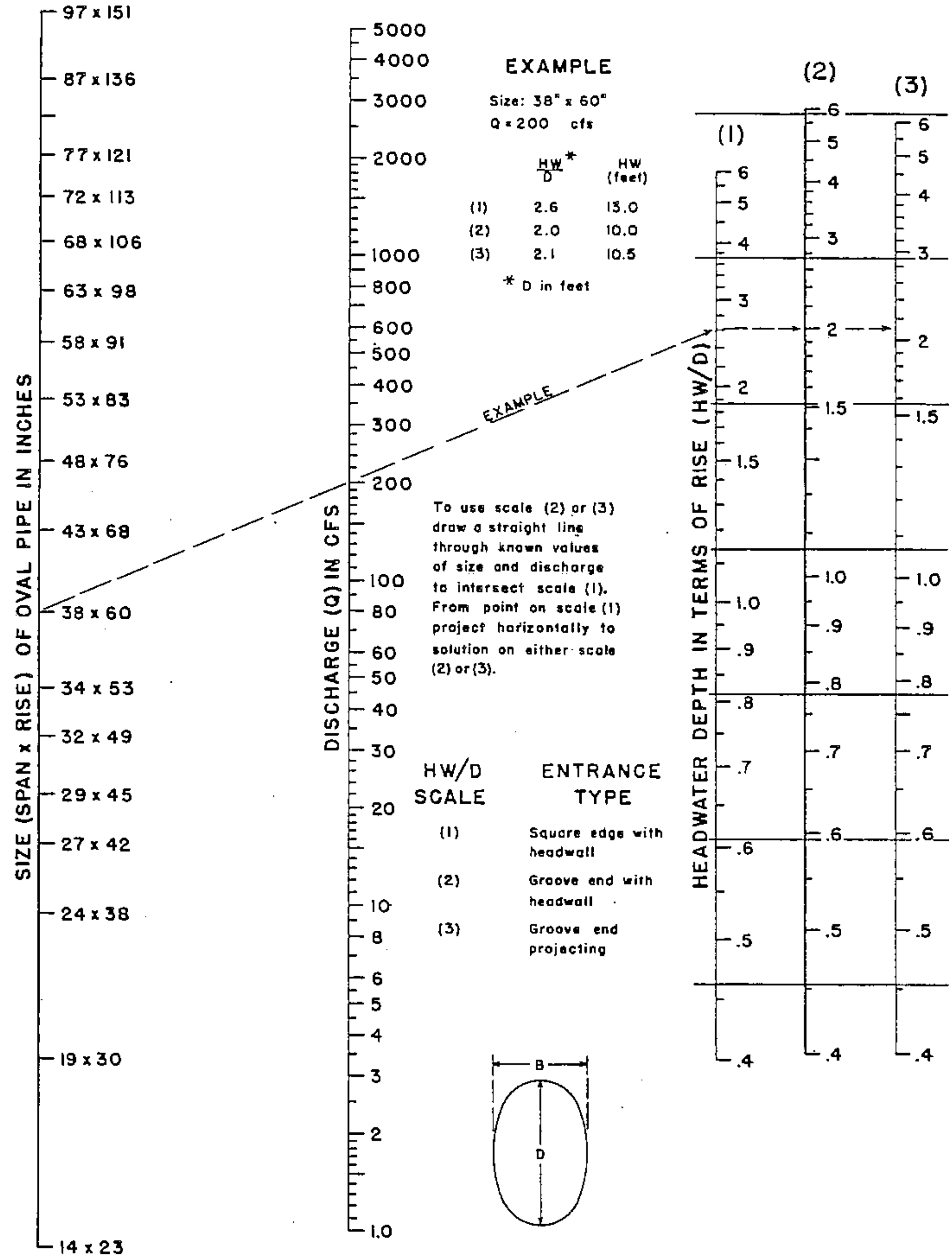
MG14186.AO



Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-52. Inlet control nomograph for oval concrete pipe culverts with the long axis horizontal.

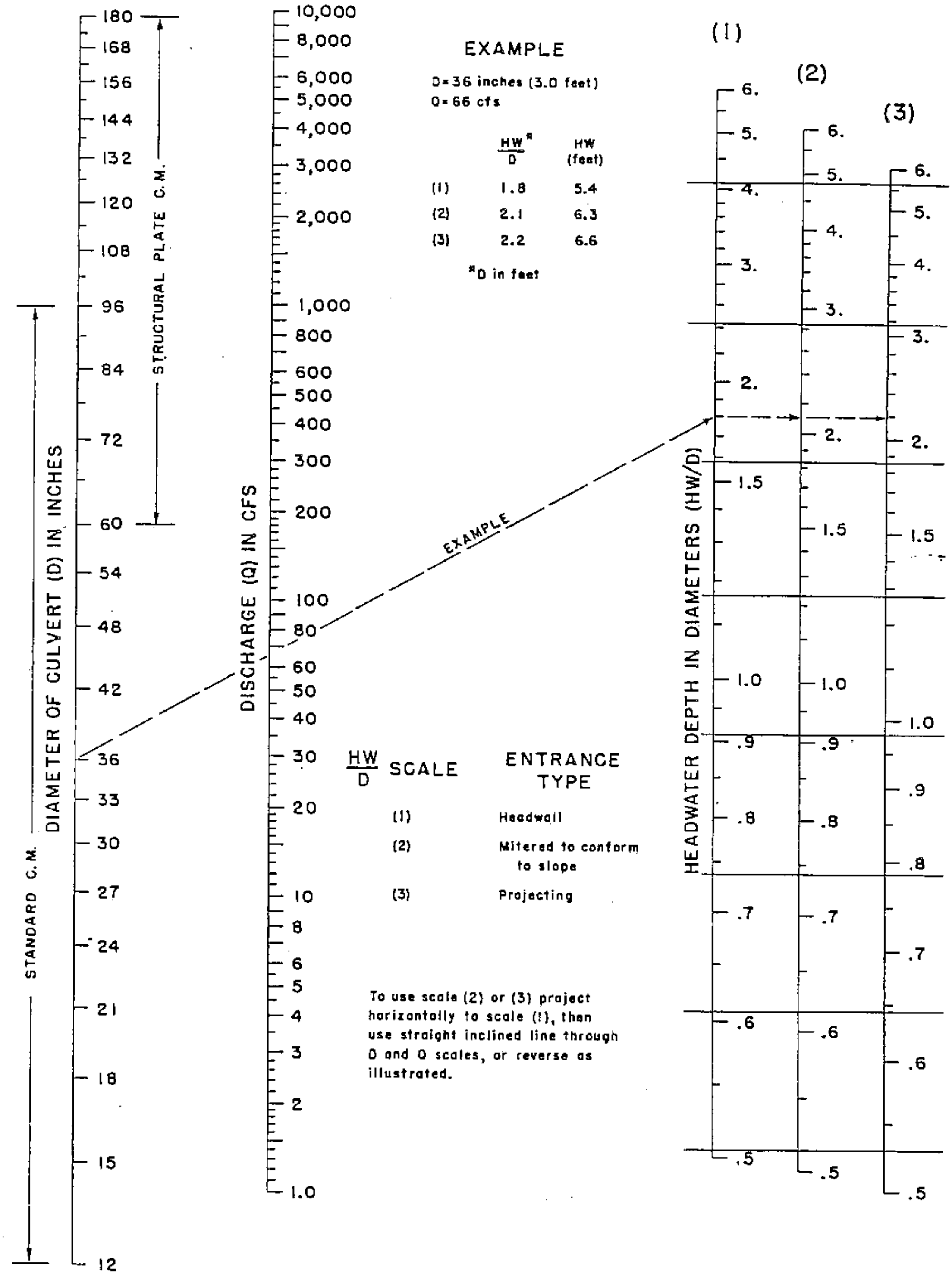
MG14186.AO



Source: U.S. DOT, FHA, HEC-5 (1965)

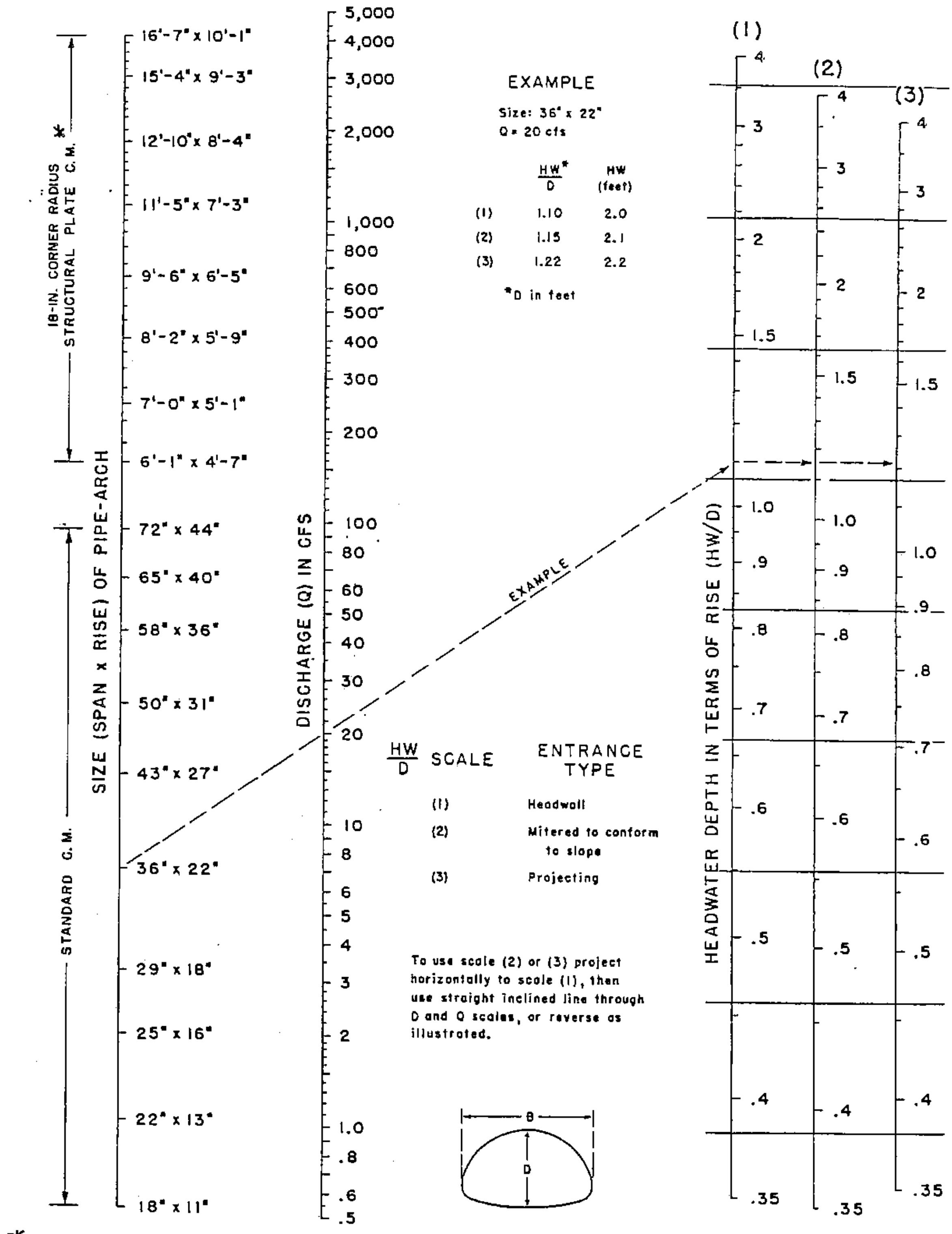
FIGURE 7-53. Inlet control nomograph for oval concrete pipe culverts with the long axis vertical.

MG141B6.AO



Source: U.S. DOT, FHA, HEC-5 (1965)

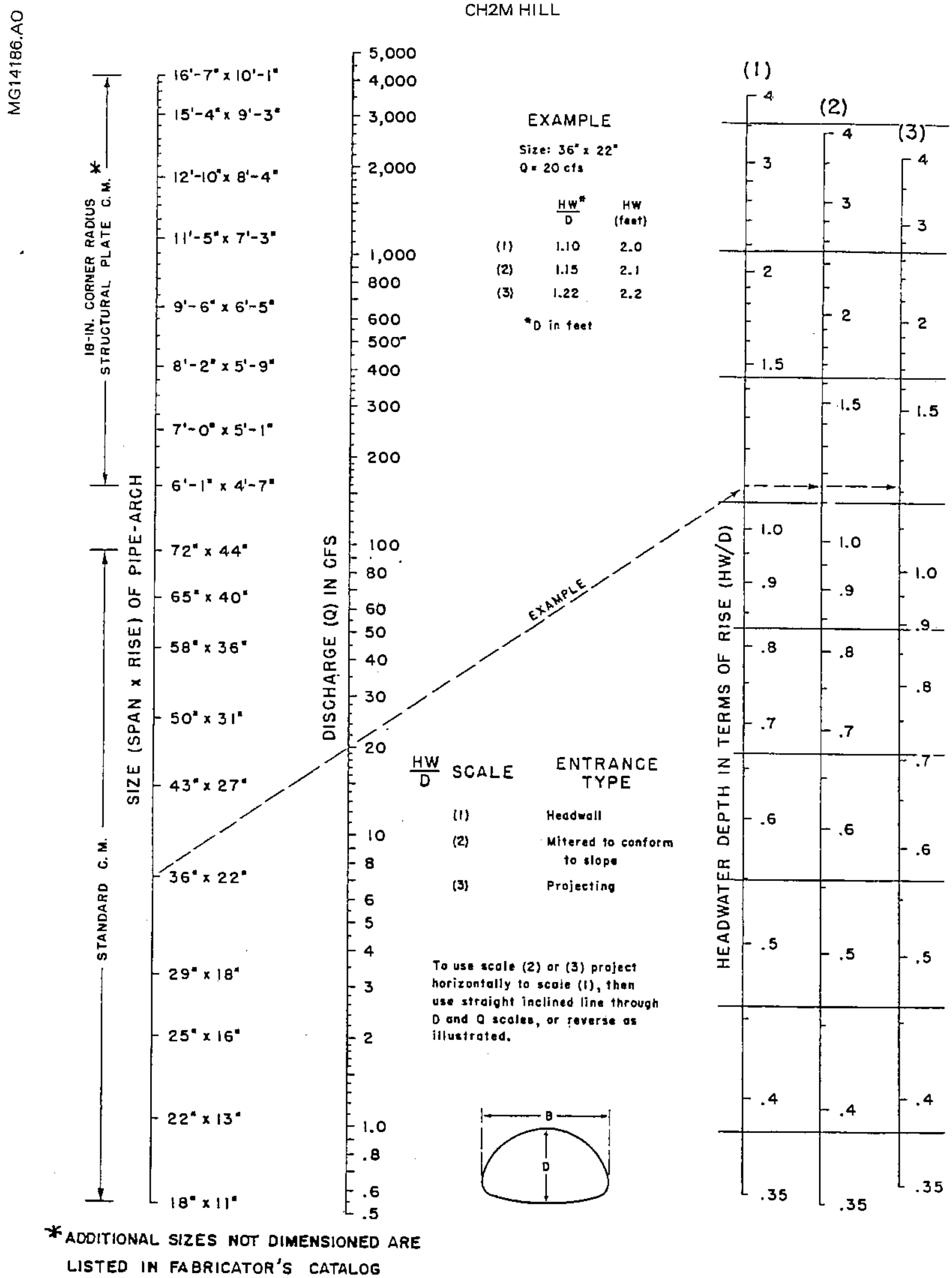
FIGURE 7-54. Inlet control nomograph for corrugated metal pipe culverts.



*ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATOR'S CATALOG

Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-55. Inlet control nomograph for corrugated metal pipe-arch culverts.

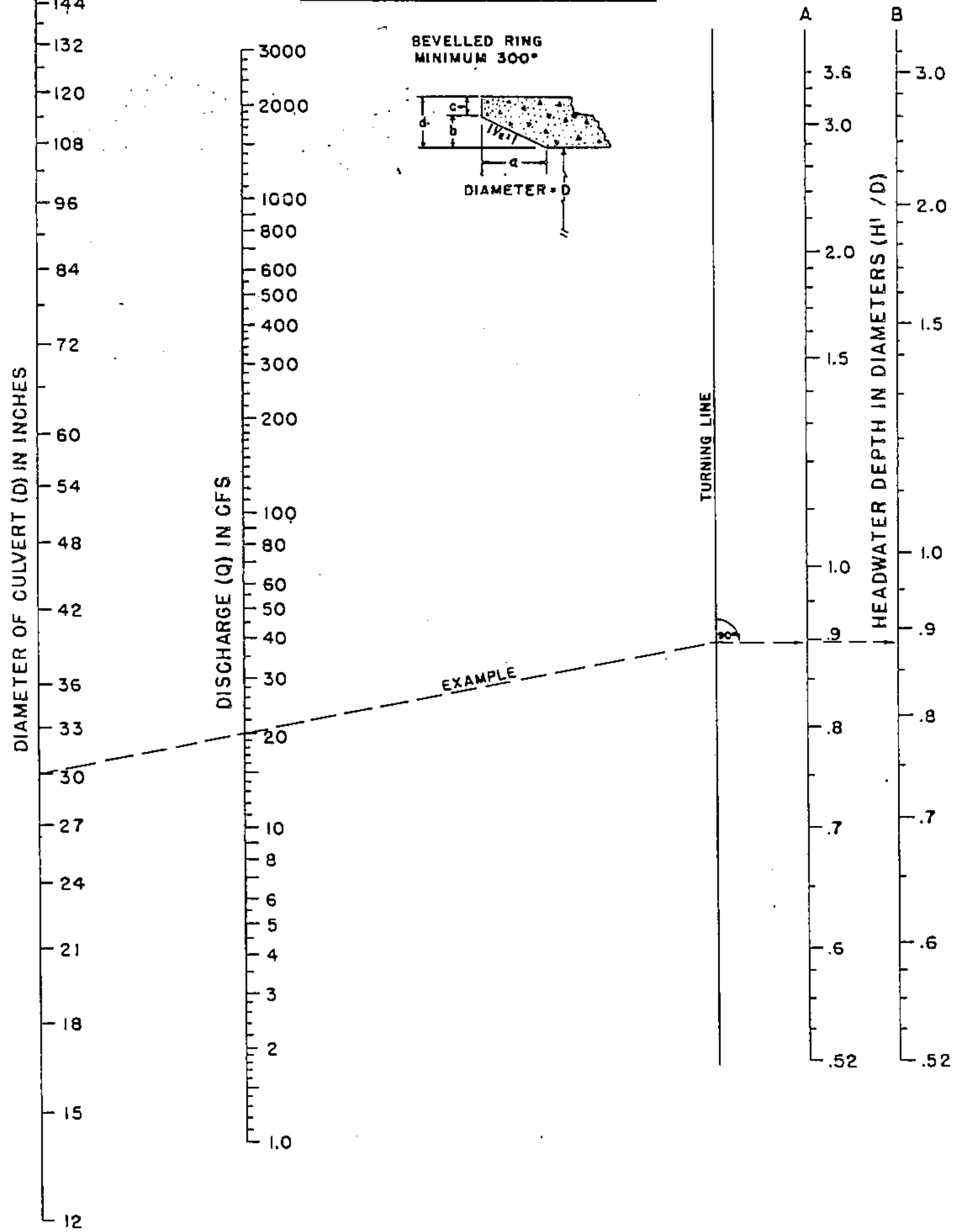


Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-55. Inlet control nomograph for corrugated metal pipe-arch culverts.

MG14186.AO

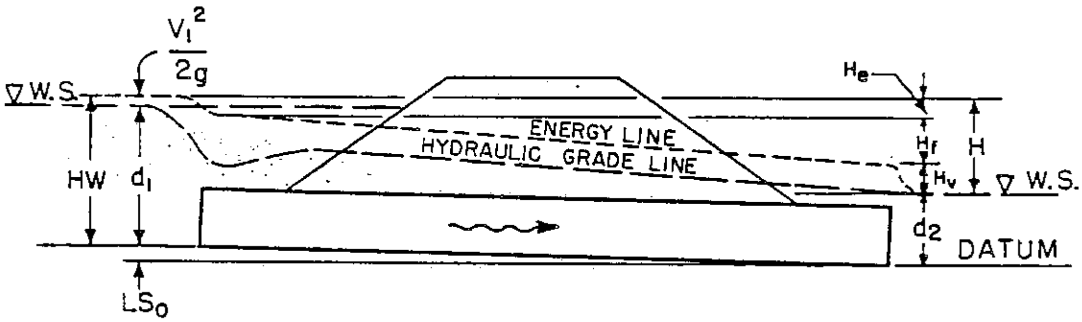
$\frac{b}{D}$	$\frac{a}{D}$	$\frac{c}{D}$	$\frac{d}{D}$	ENTRANCE TYPE
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B



Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-56. Inlet control nomograph for circular pipe culverts with beveled rings.

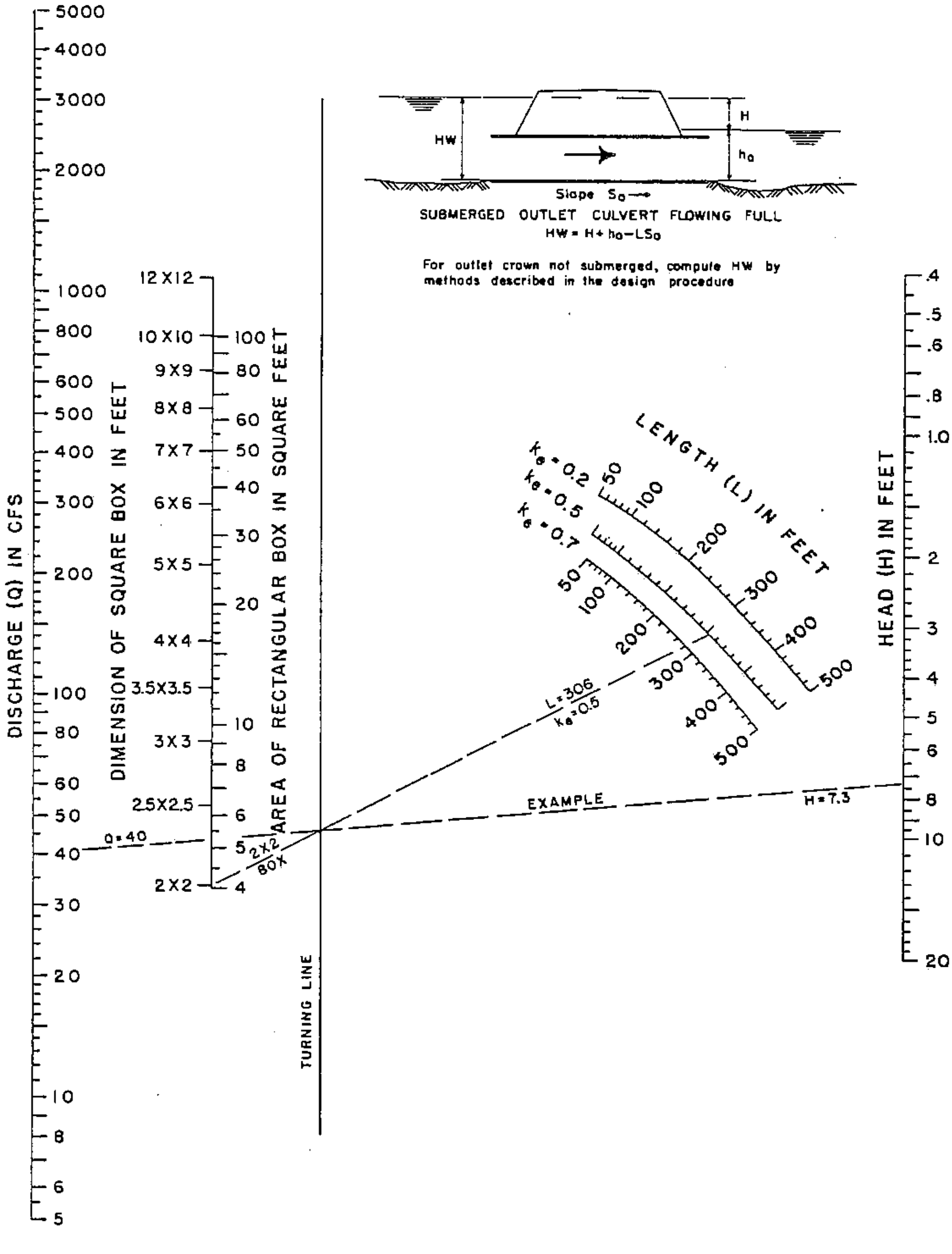
MG14186.AO



Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-57. A culvert flowing full under outlet control.

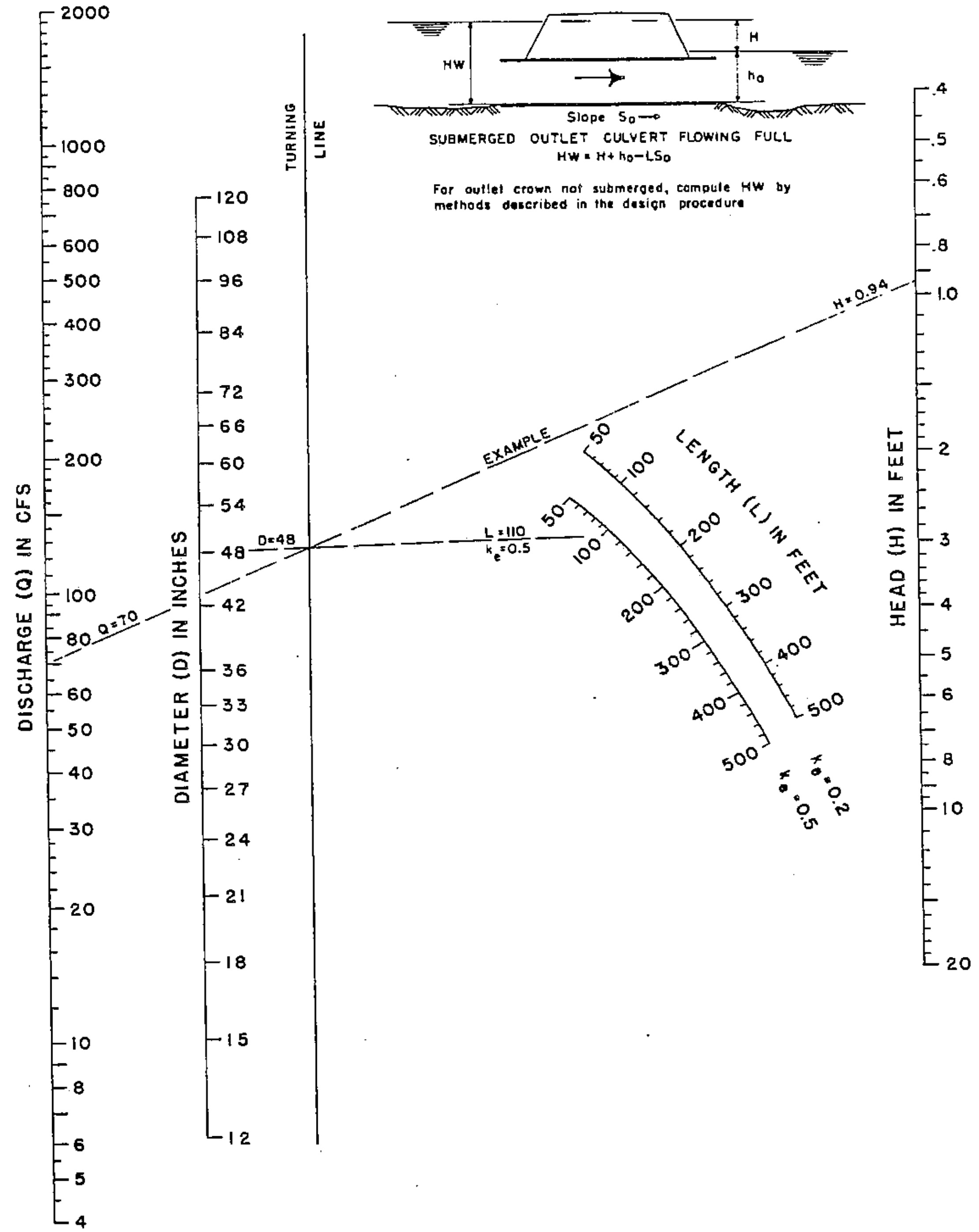
MG14186.AO



Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-58. Outlet control nomograph for square concrete box culverts flowing full. n = 0.012

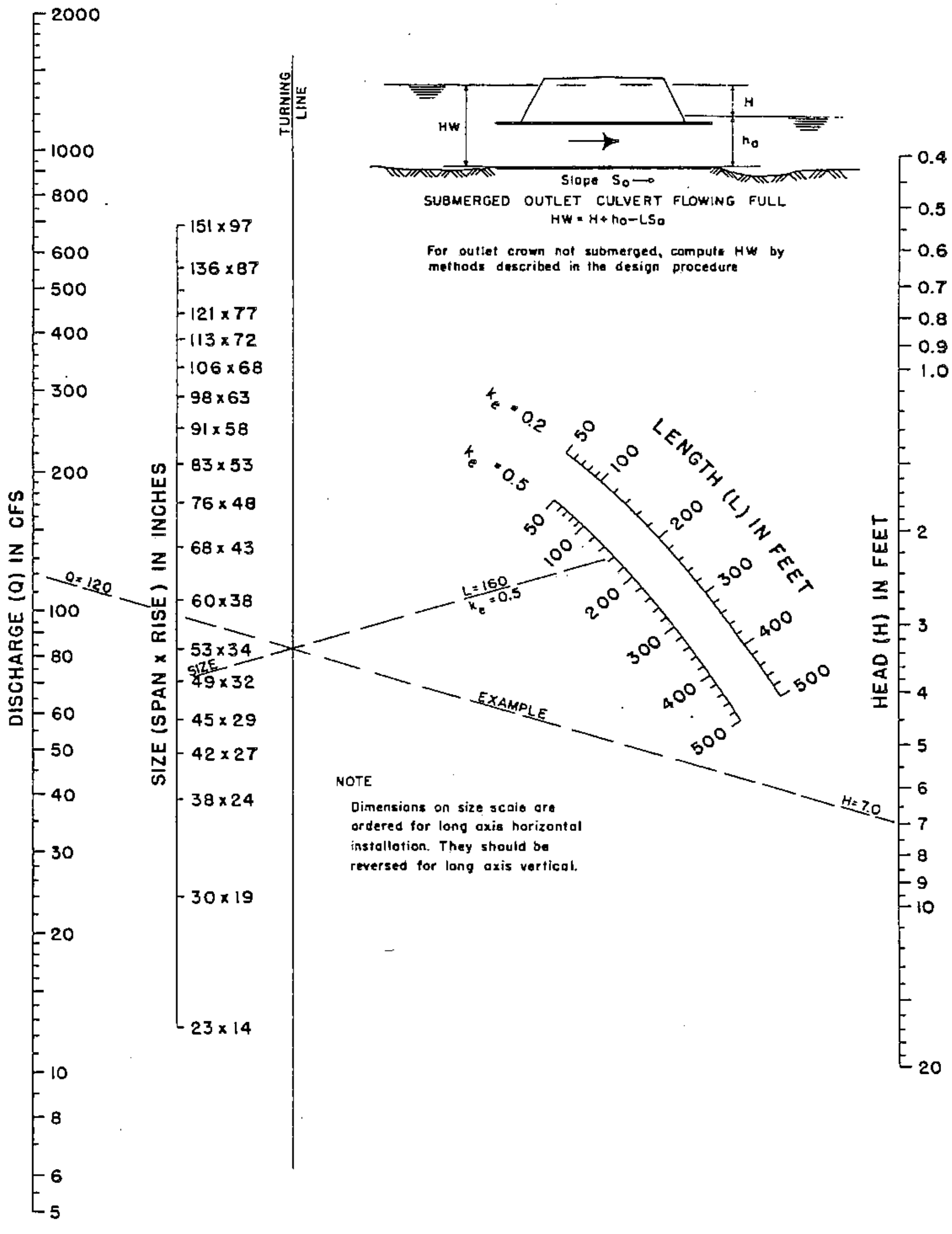
MG14186.AO



Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-59. Outlet control nomograph for concrete pipe culverts flowing full. n=0.012

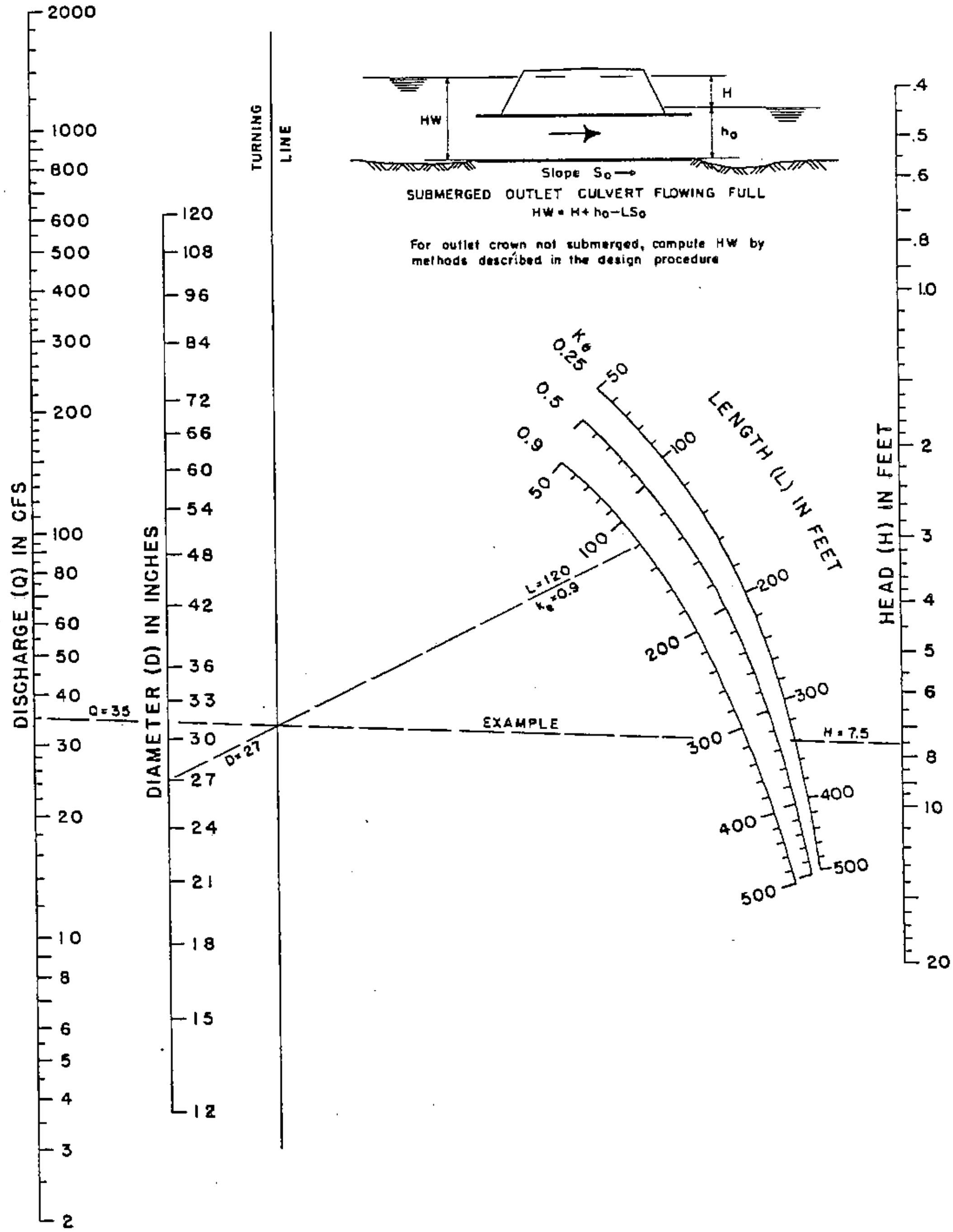
MG14186.AO



Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-60. Outlet control nomograph for oval concrete pipe culverts with the long axis horizontal or vertical flowing full.
n=0.012

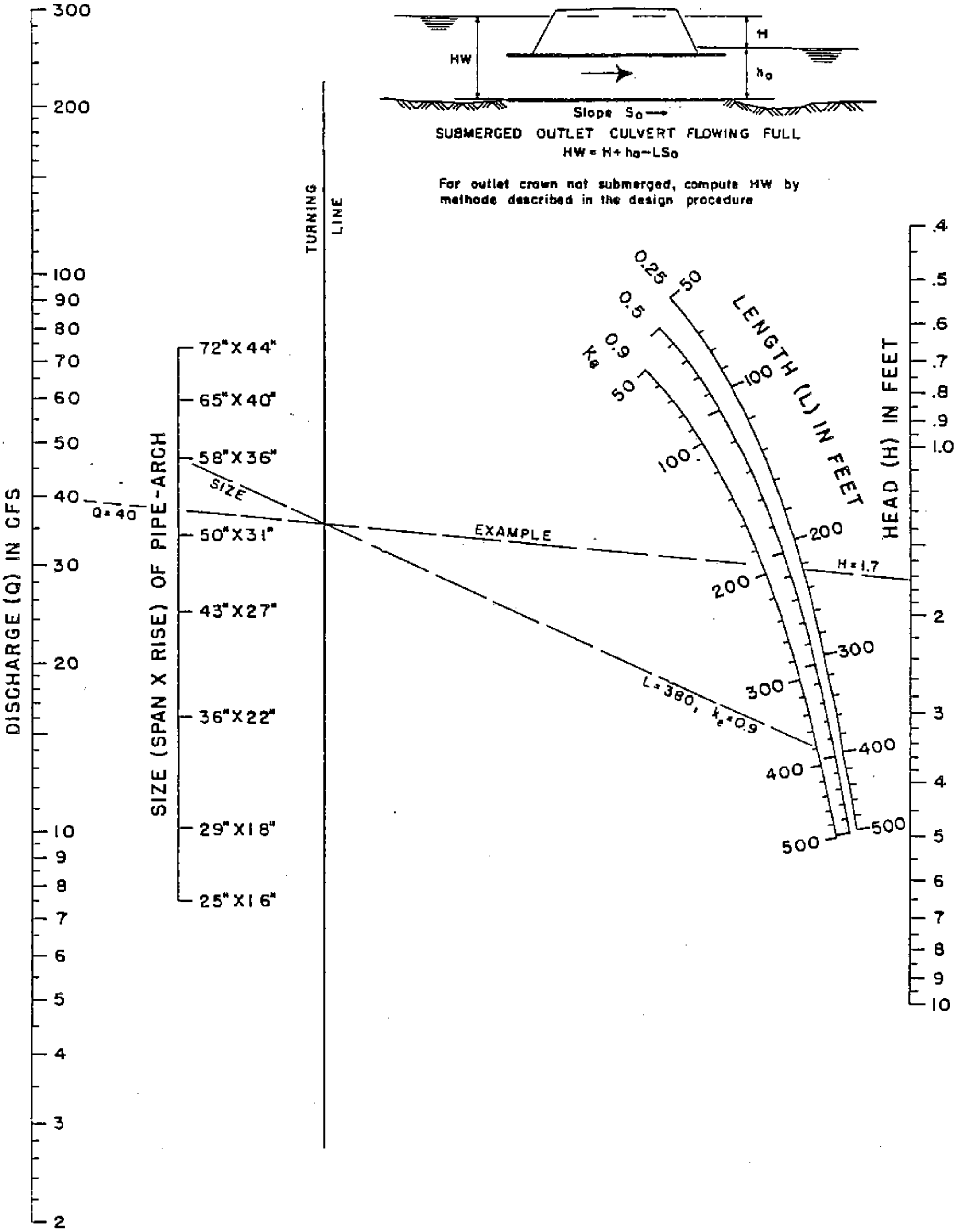
MG14186.AO



Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-61. Outlet control nomograph for corrugated metal pipe culverts flowing full. n = 0.024

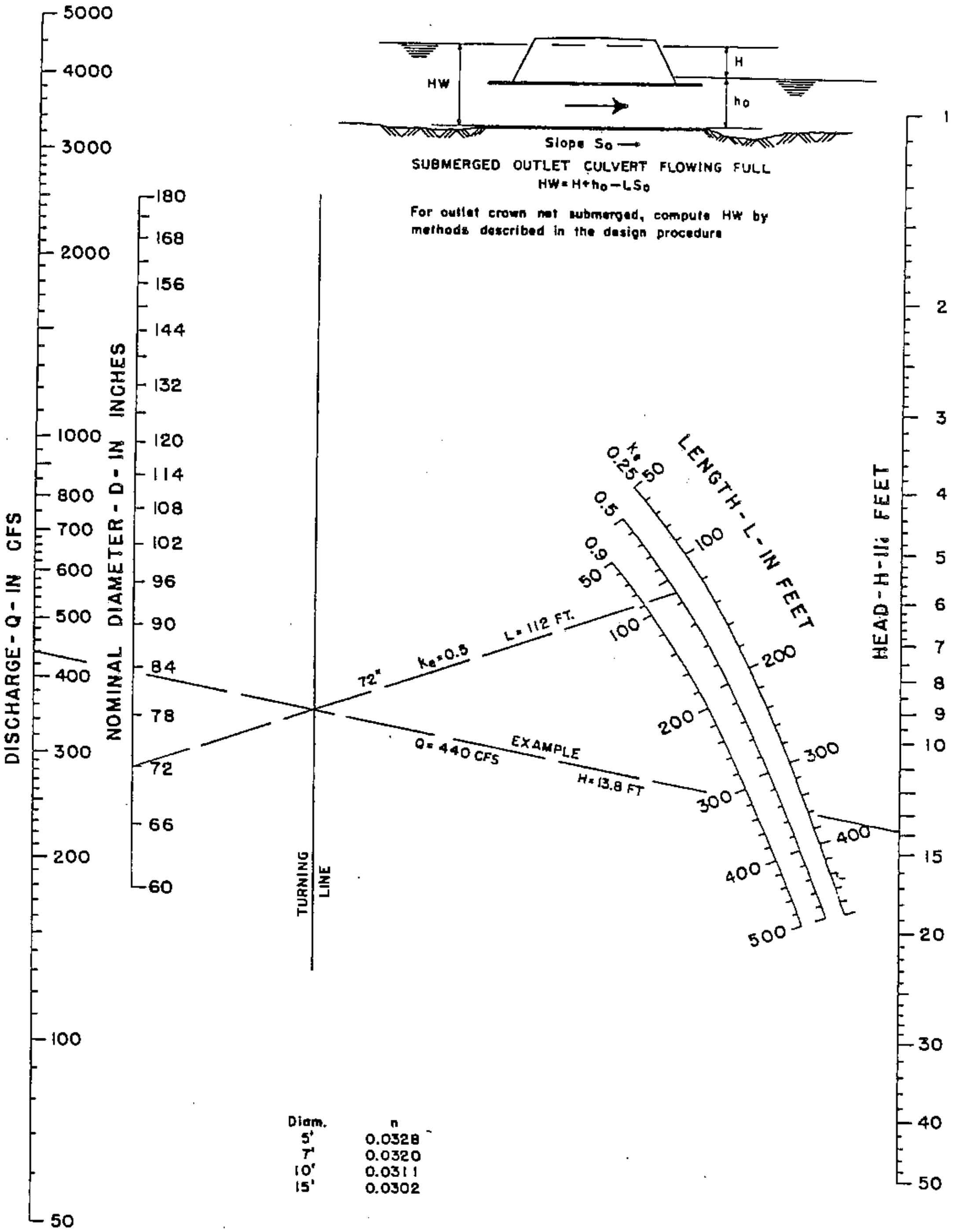
MG14186.AO



Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-62. Outlet control nomograph for standard corrugated metal pipe-arch culverts flowing full. n = 0.024

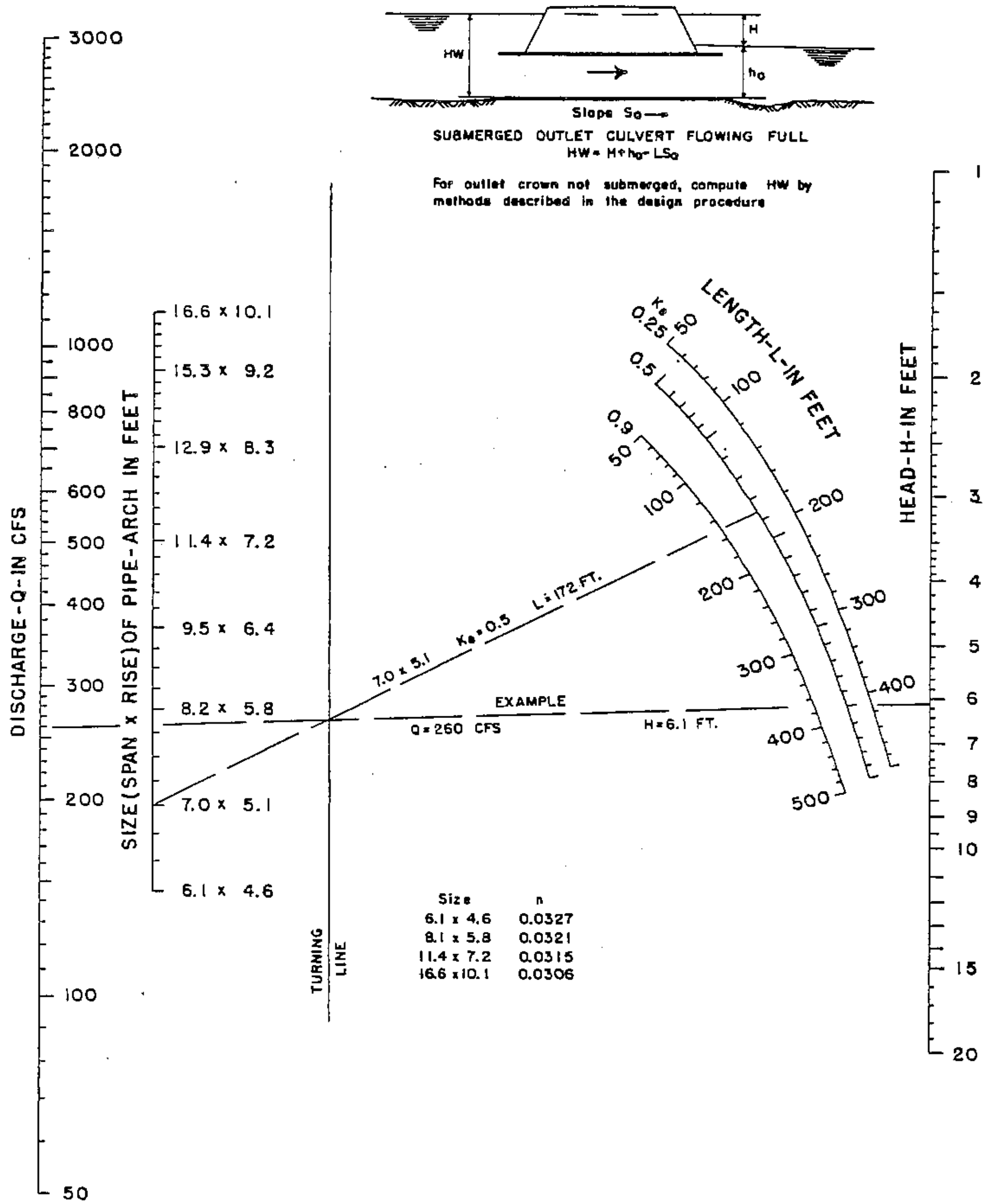
MG14186.AO



Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-63. Outlet control nomograph for structural plate metal pipe culverts flowing full. $n = 0.0328$ to 0.0302

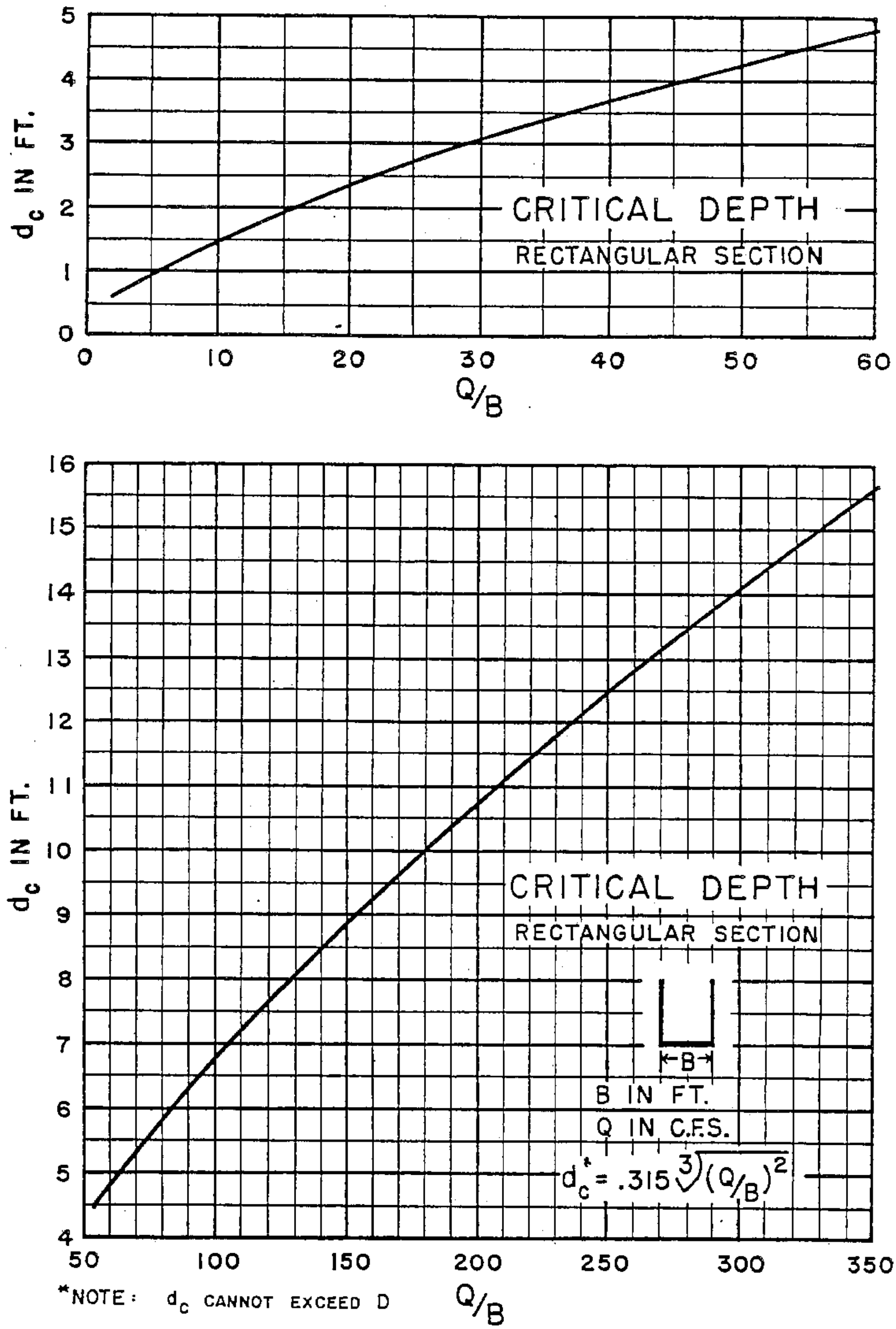
MG14186.AO



Source: U.S. DOT, FHA, HEC-5 (1965)

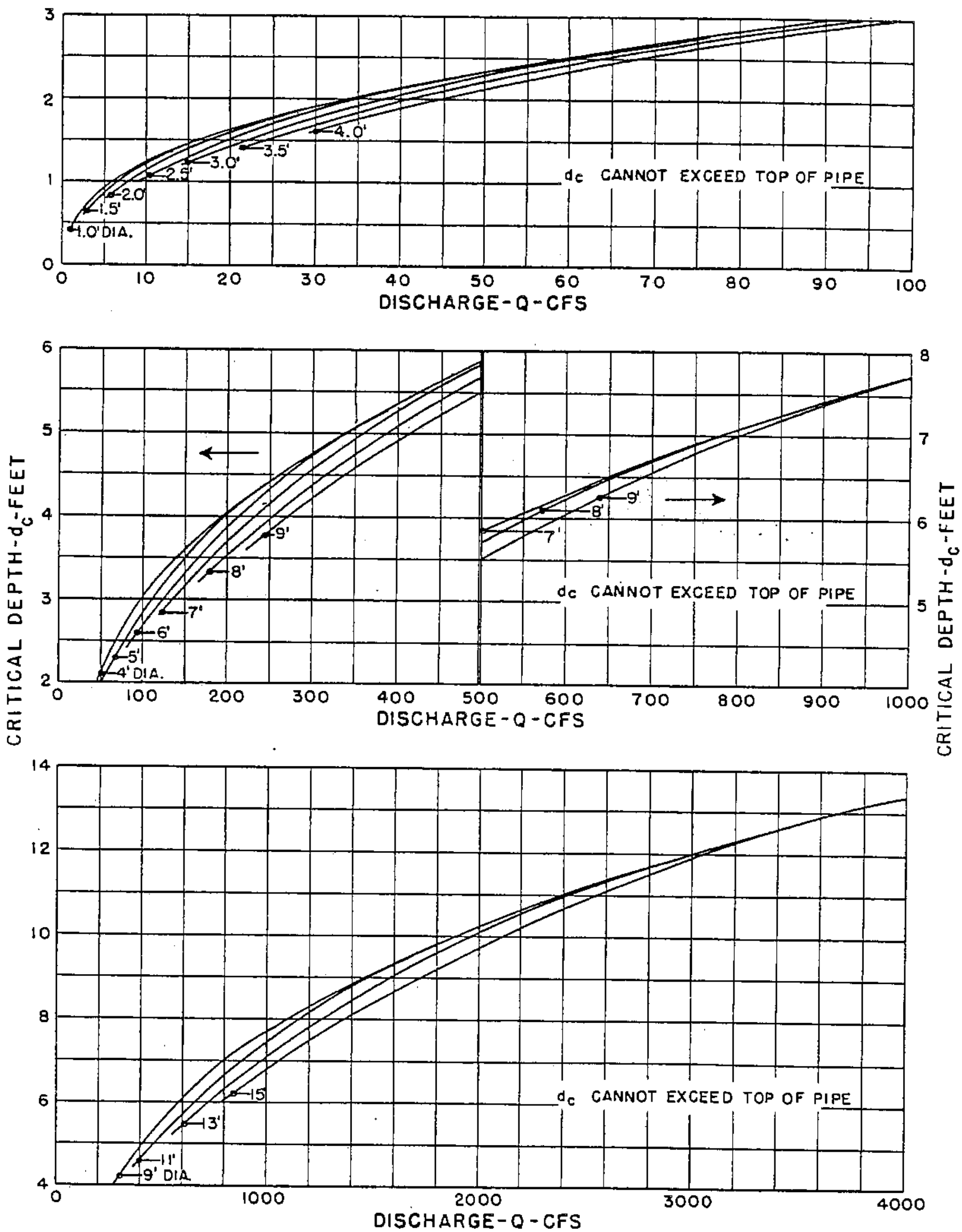
FIGURE 7-64. Outlet control nomograph for structural plate corrugated metal pipe-arch culverts, 18-inch corner radius, flowing full.
n = 0.0327 to 0.0306

MG14186.AO



Source: U.S. DOT, FHA, HEC-5 (1965)

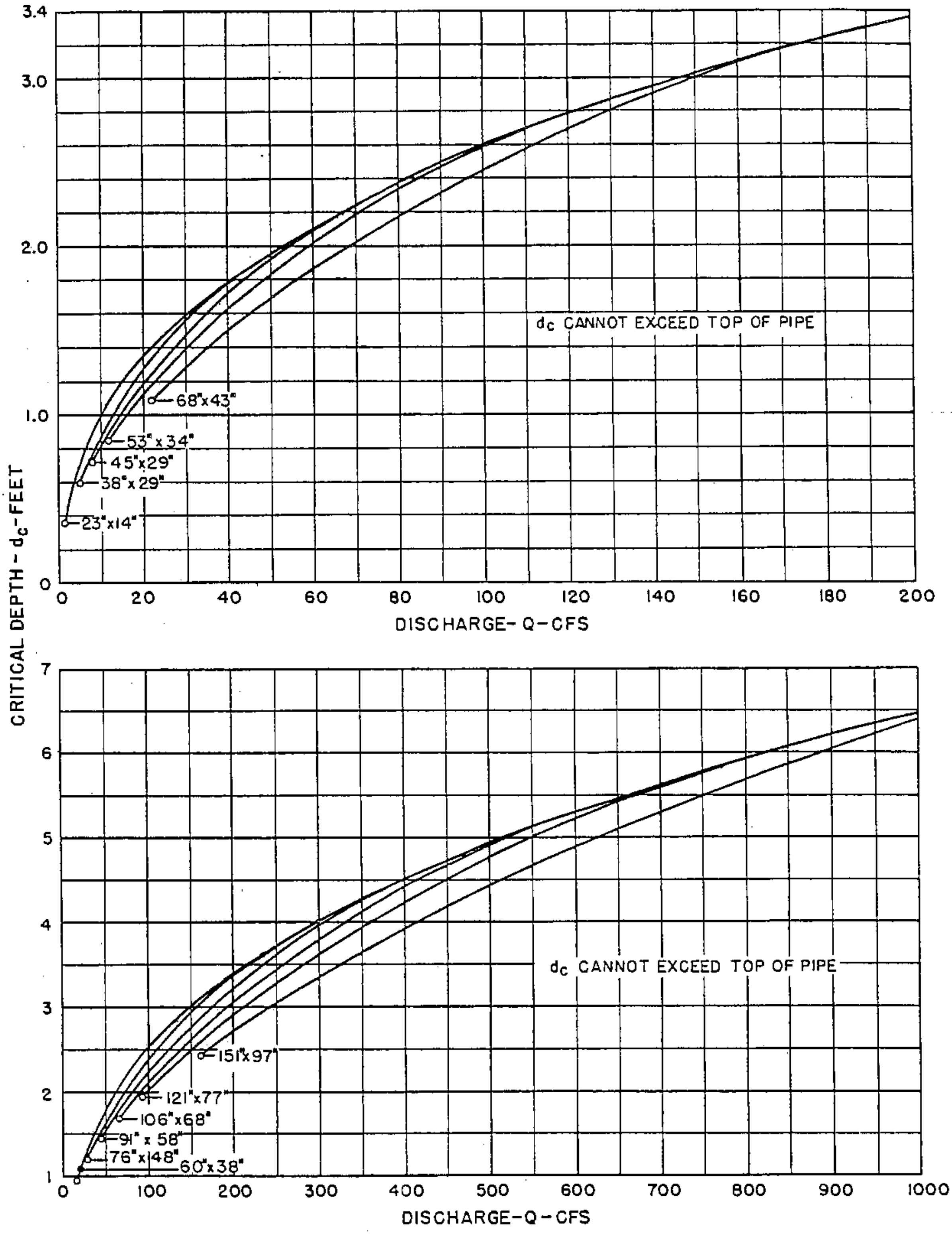
FIGURE 7-65. Critical depth for a rectangular section culvert.



Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-66. Critical depth for a circular pipe culvert.

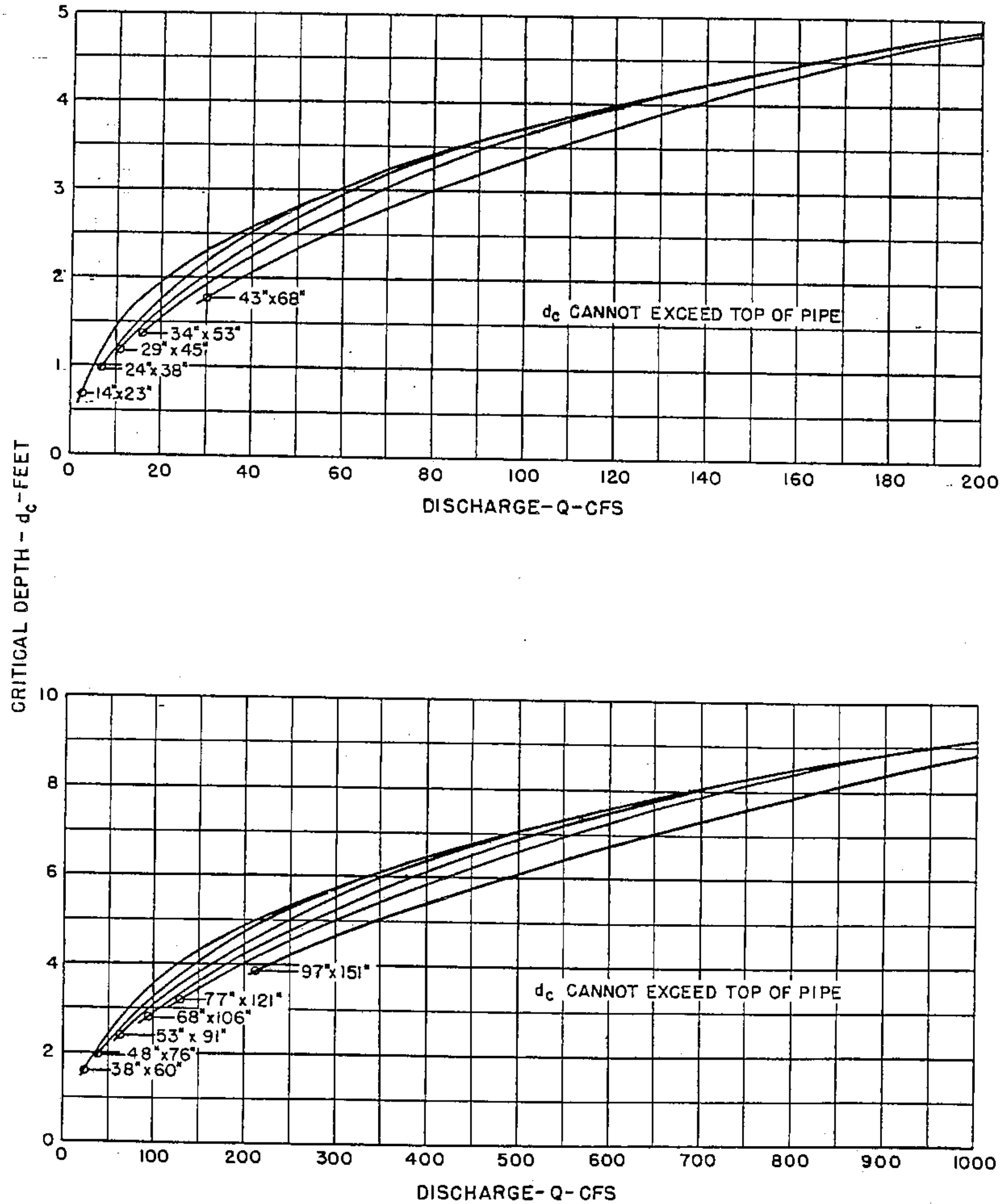
MG14186.AO



Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-67. Critical depth for an oval concrete pipe culvert with the long axis horizontal.

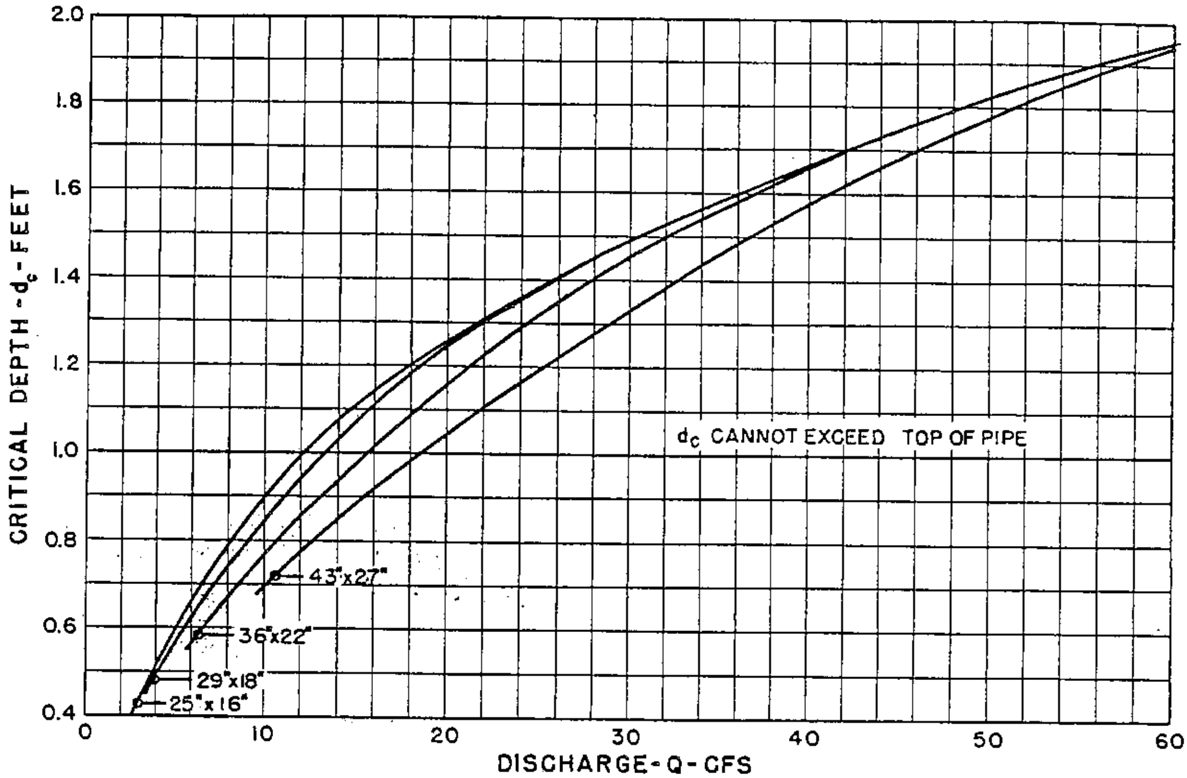
MG14186.AO

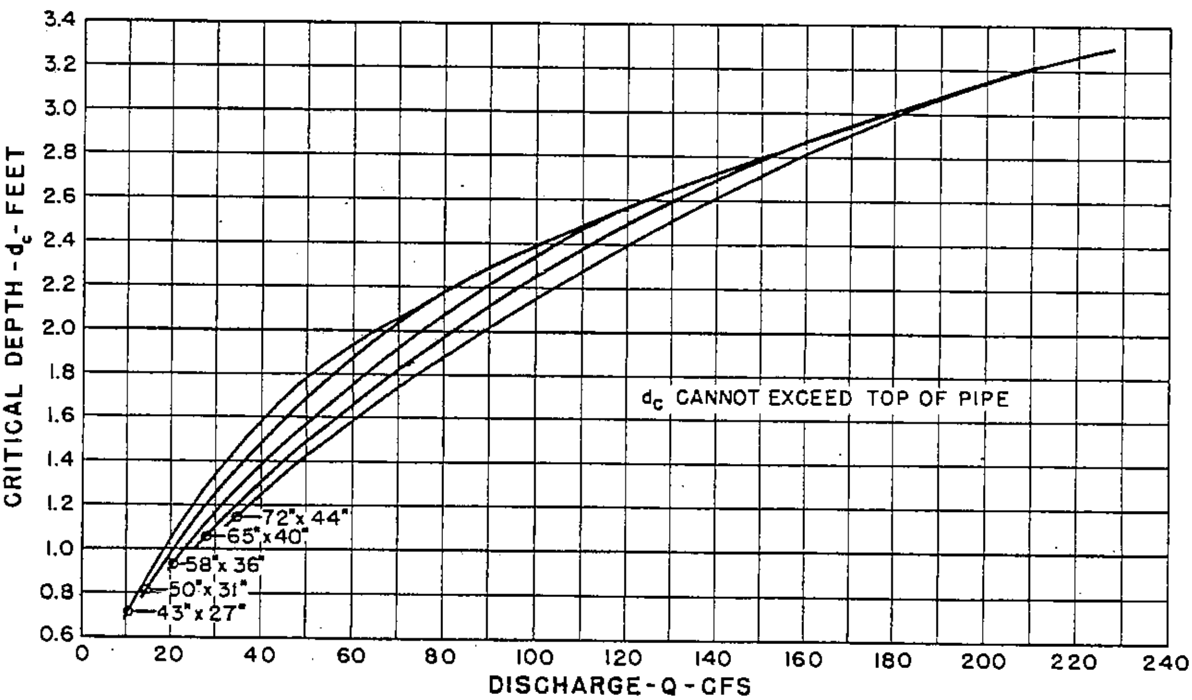


Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-68. Critical depth for an oval concrete pipe culvert with the long axis vertical.

MG14186.AO

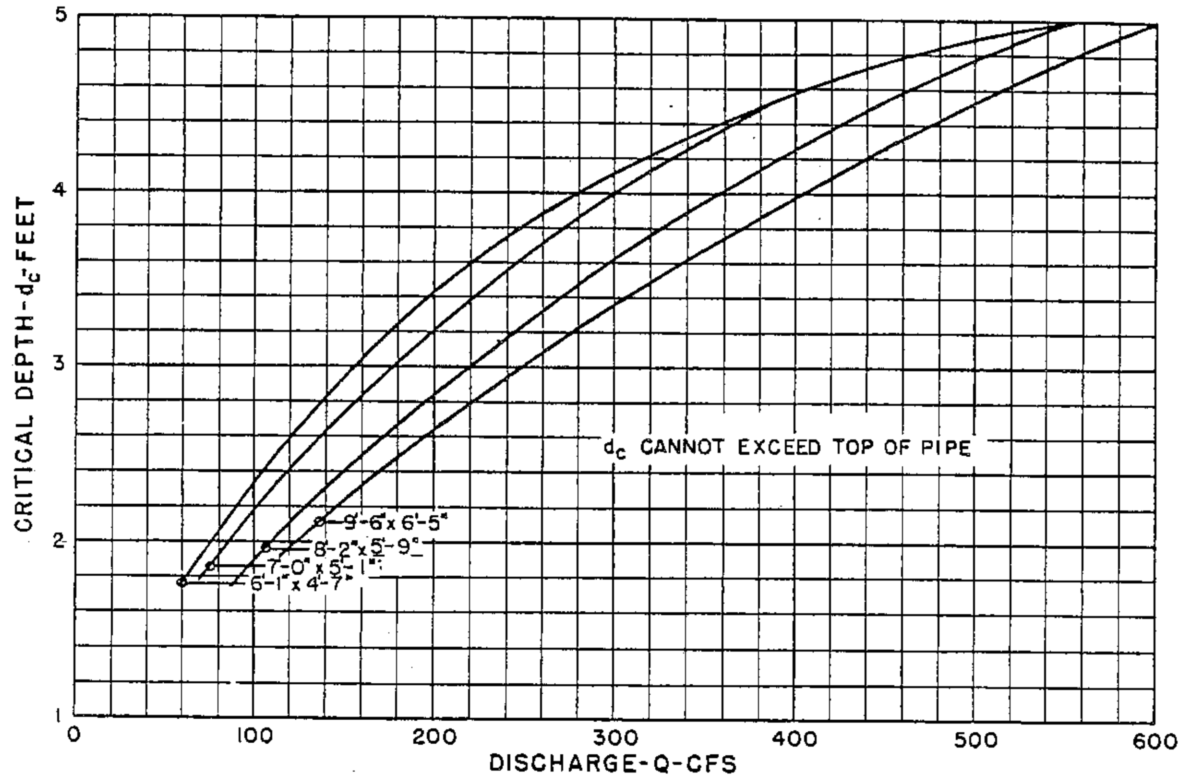


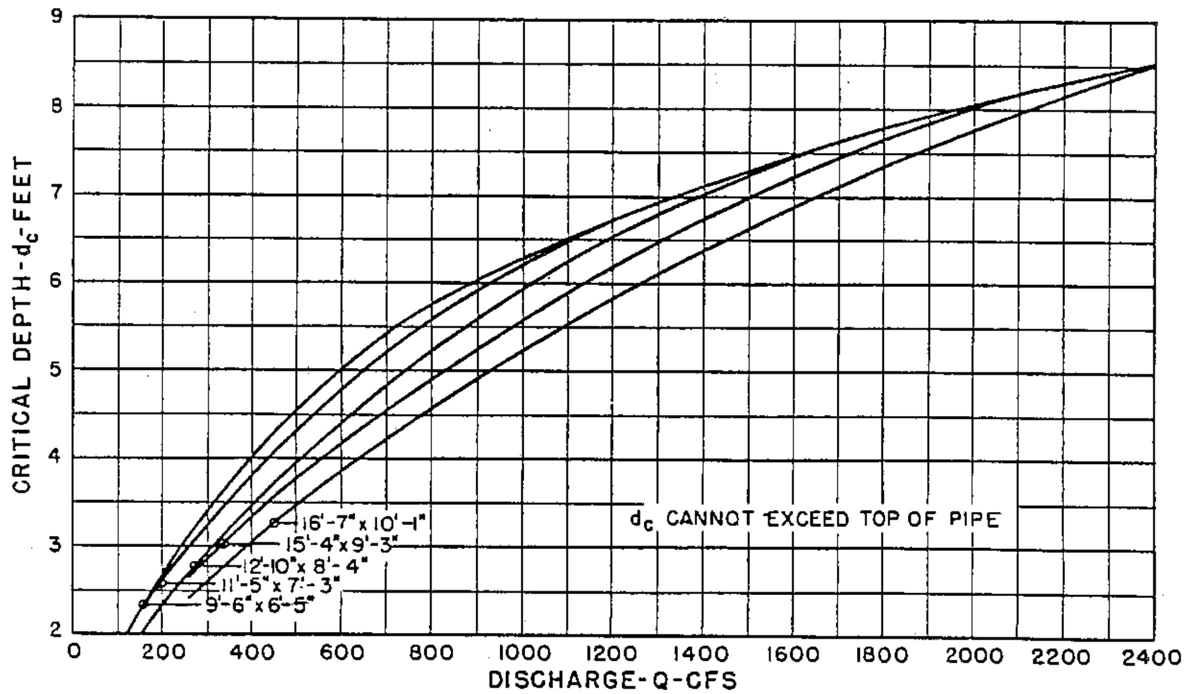


Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-69. Critical depth for a standard corrugated metal pipe-arch culvert.

MG14186.AO





Source: U.S. DOT, FHA, HEC-5 (1965)

FIGURE 7-70. Critical depth for a structural plate corrugated metal pipe-arch culvert.

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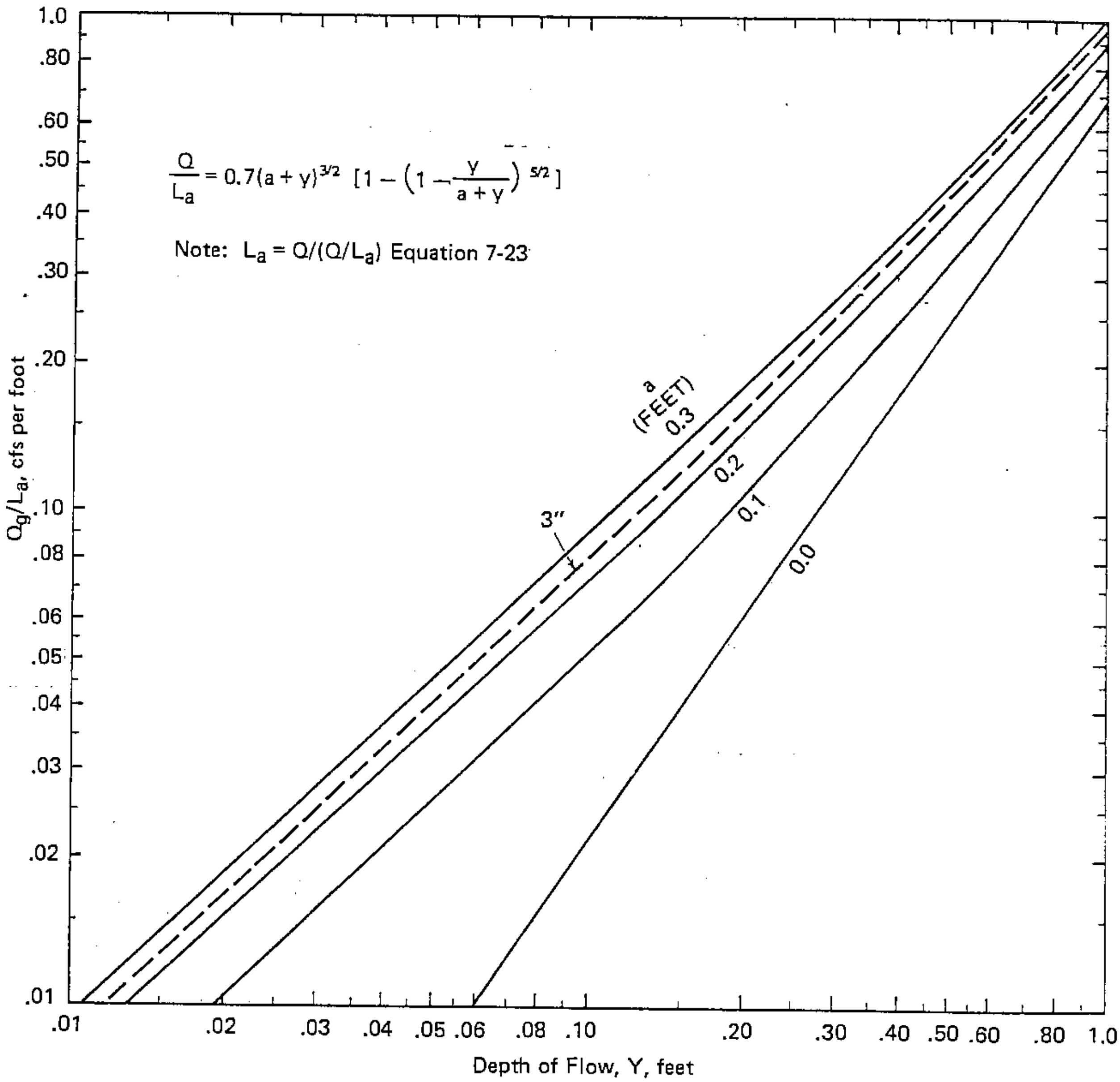

a = depth of local depression, feet
Y = depth of gutter flow at curb, upstream from the local depression, feet
Q_g = gutter flow rates, cfs
L_a = length of curb opening required for 100% intercept

Source: Izzard (1950).

FIGURE 7-71. Discharge per foot of curb opening for 100% intercept on a continuous grade.

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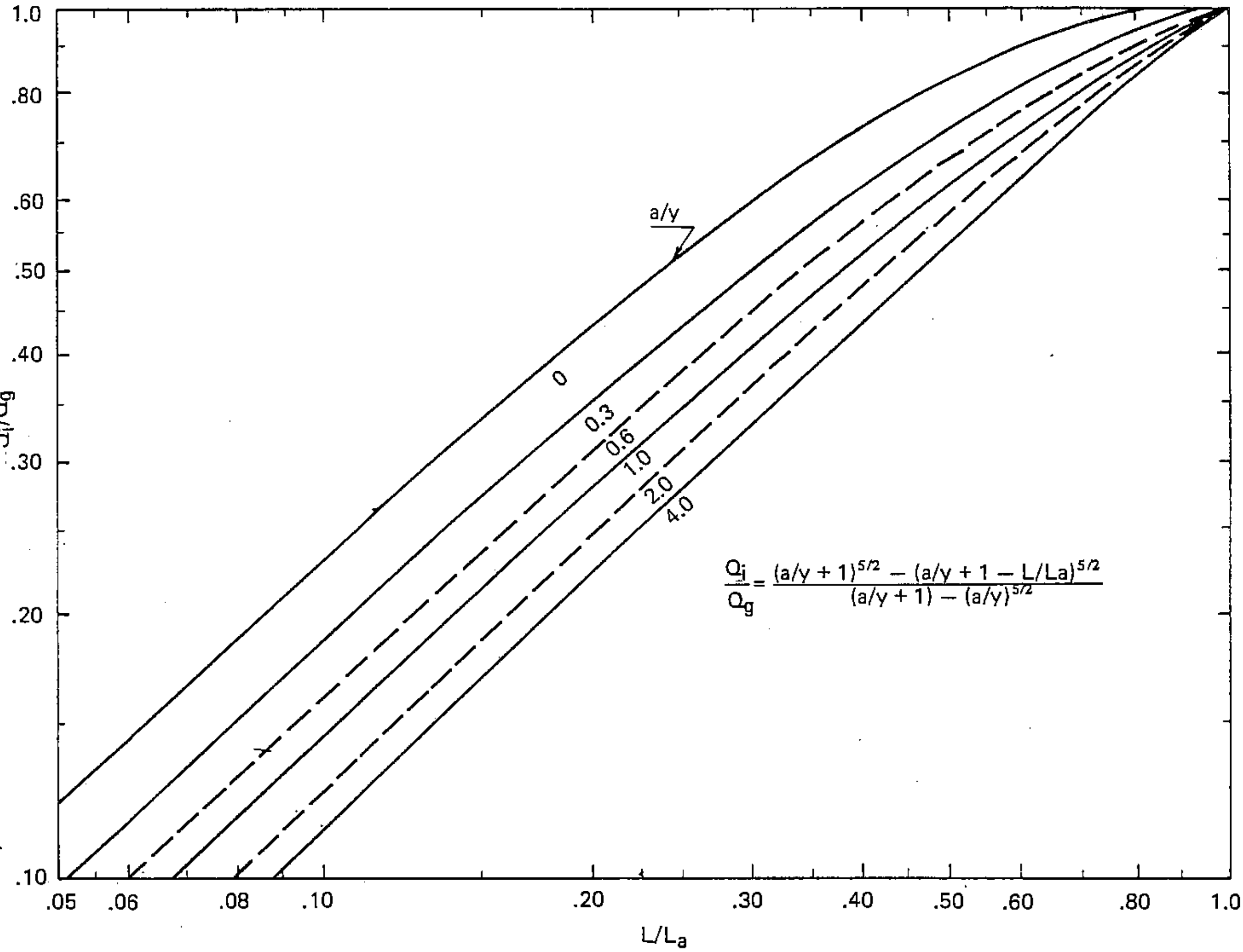
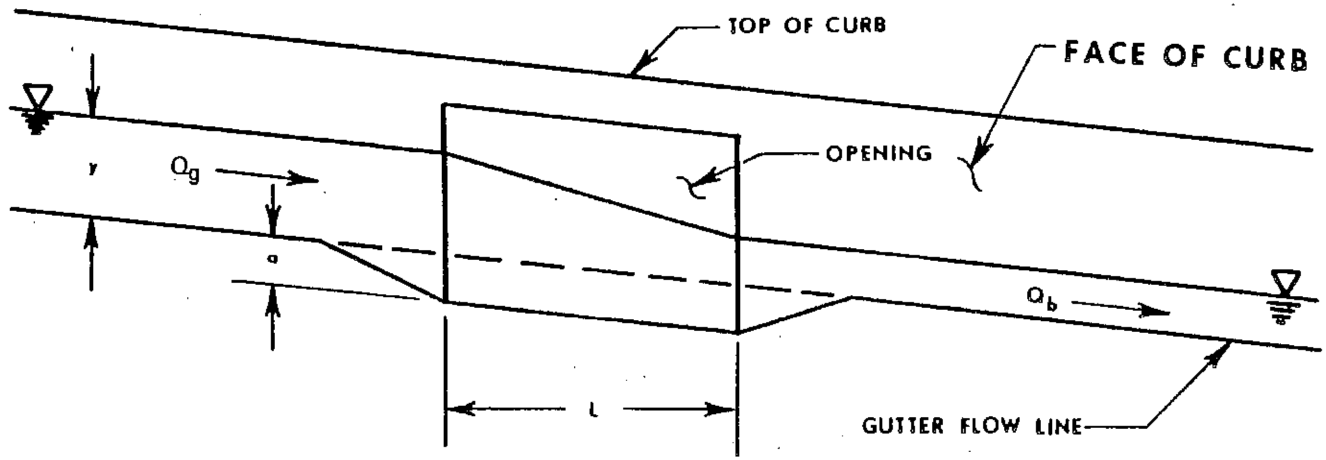


L = actual length of curb opening, feet
L_a = length of curb opening required for 100% intercept
a = depth of local depression, feet
y = depth of flow at curb, upstream from the local depression, feet
Q_i = inlet intercept, cfs
Q_g = gutter flow rate, cfs

Source: Izzard (1950).

FIGURE 7-72. Partial interception ratio for curb opening inlets on a continuous grade.



$$Q_i = Q - Q_b$$

Q_i = Intercepted flow, in cfs
y = Depth of gutter flow at curb, upstream from the local depression, in feet
Q_g = Gutter flow rate, in cfs
a = Depth of local depression, in feet
L = Actual length of curb opening, in feet
Q_b = Gutter flow bypass rate, in cfs

FIGURE 7-73. Parameter definitions for a depressed curb opening inlet on a continuous grade.

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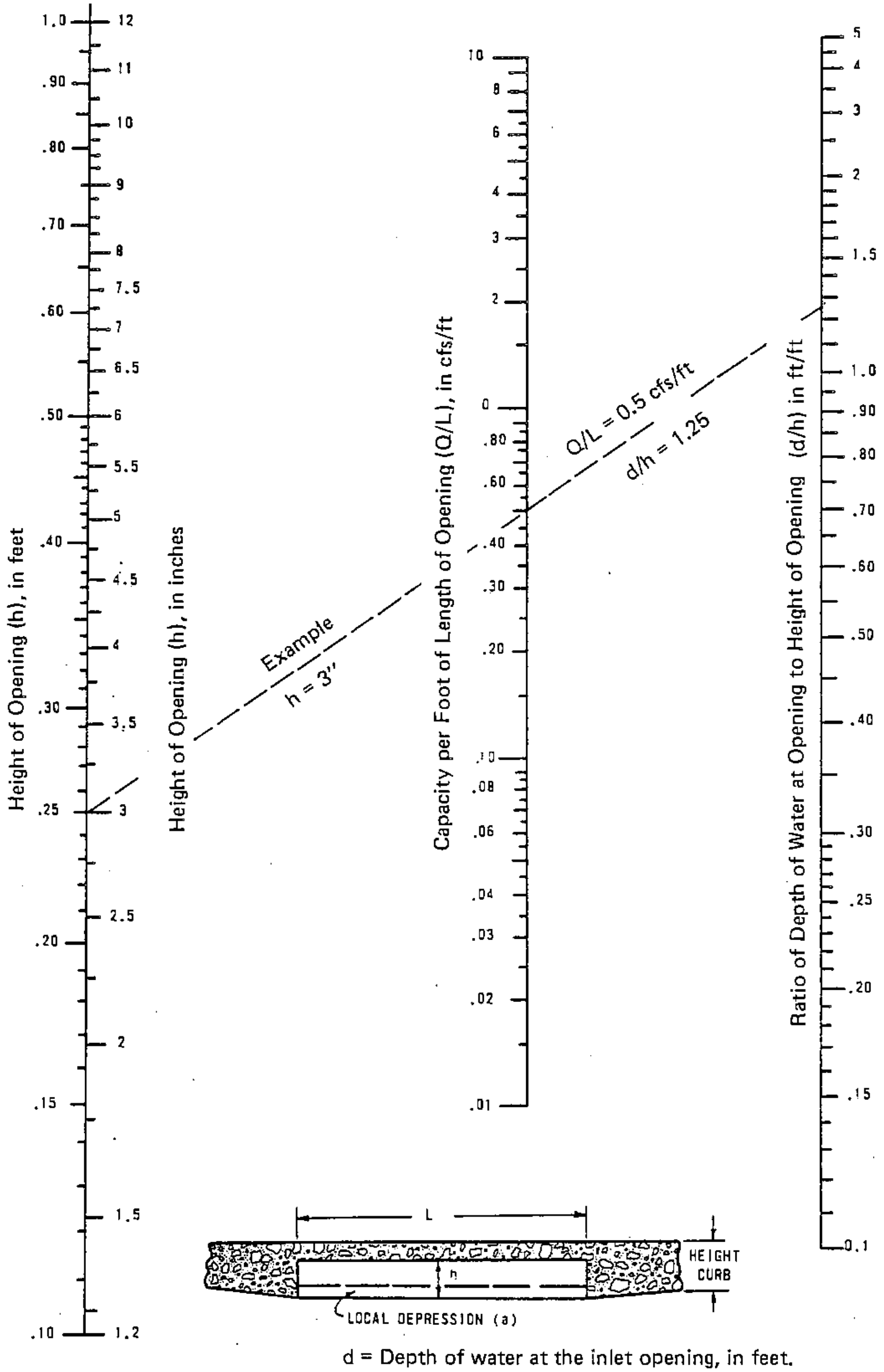

FIGURE 7-74. Nomograph for capacity of curb opening inlets at low points.

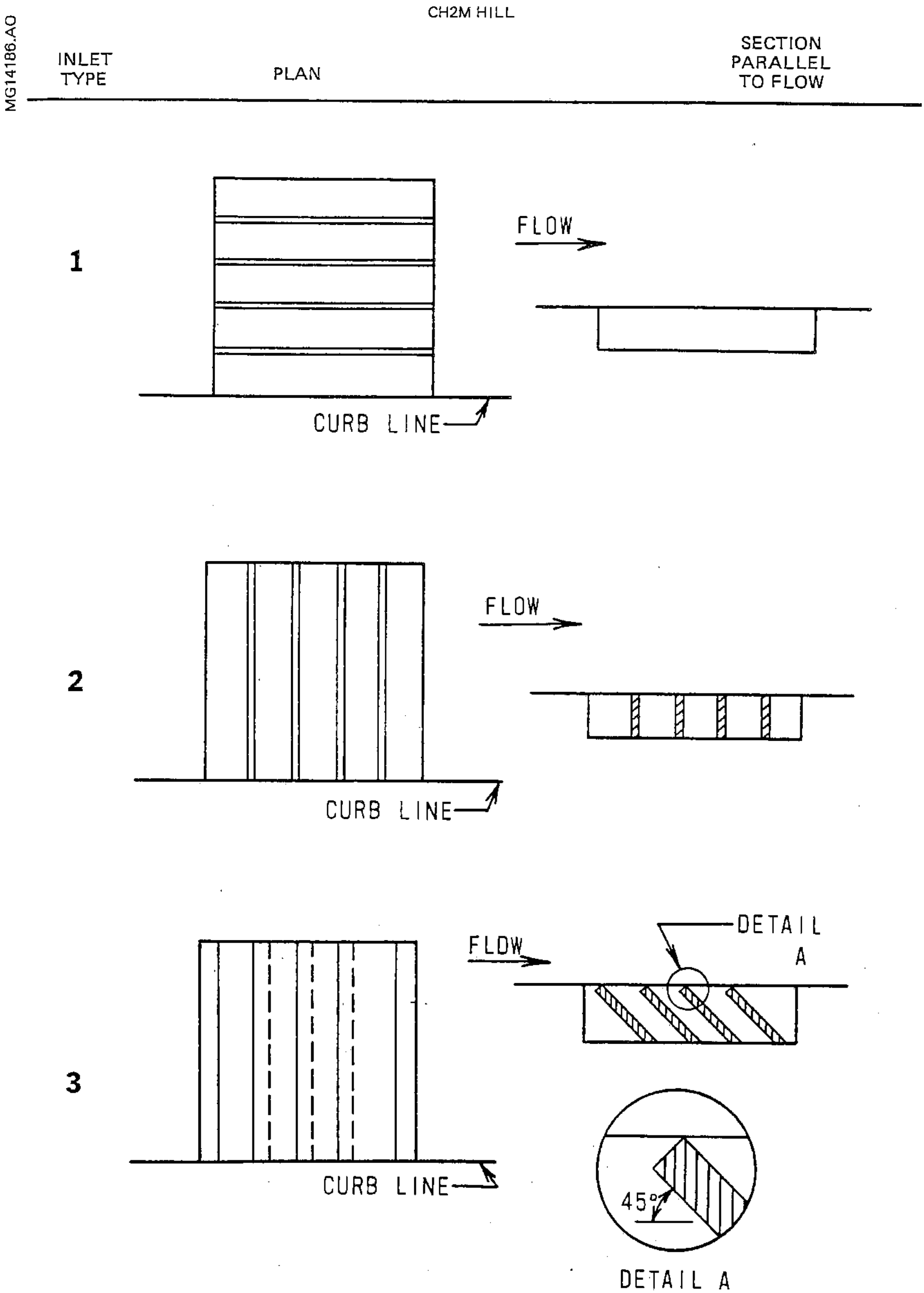

FIGURE 7-75. Six typical grate inlet configurations.

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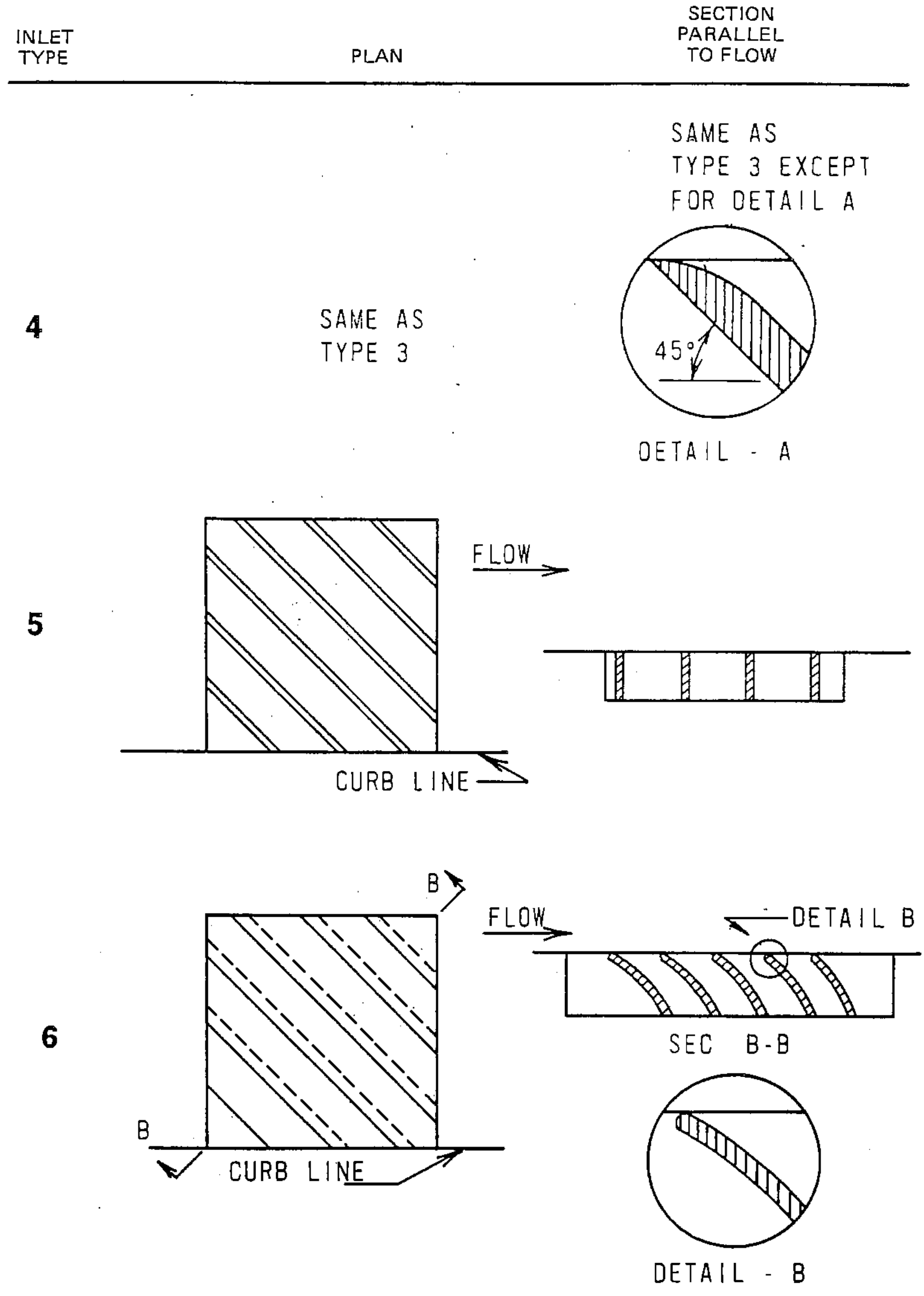


FIGURE 7-75. Six typical grate inlet configurations, continued.

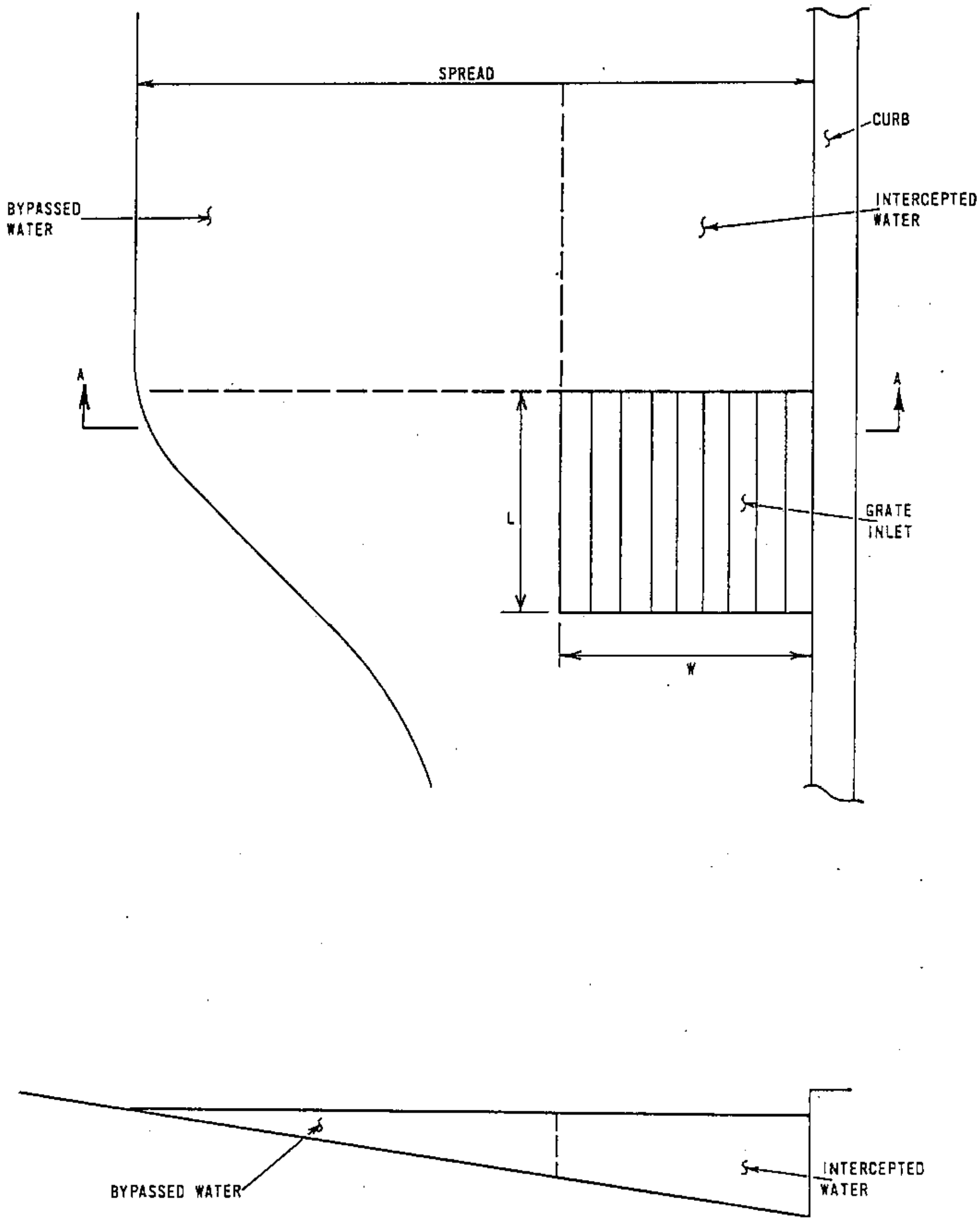


FIGURE 7-76. Intercept of an efficient grate inlet on a continuous grade.

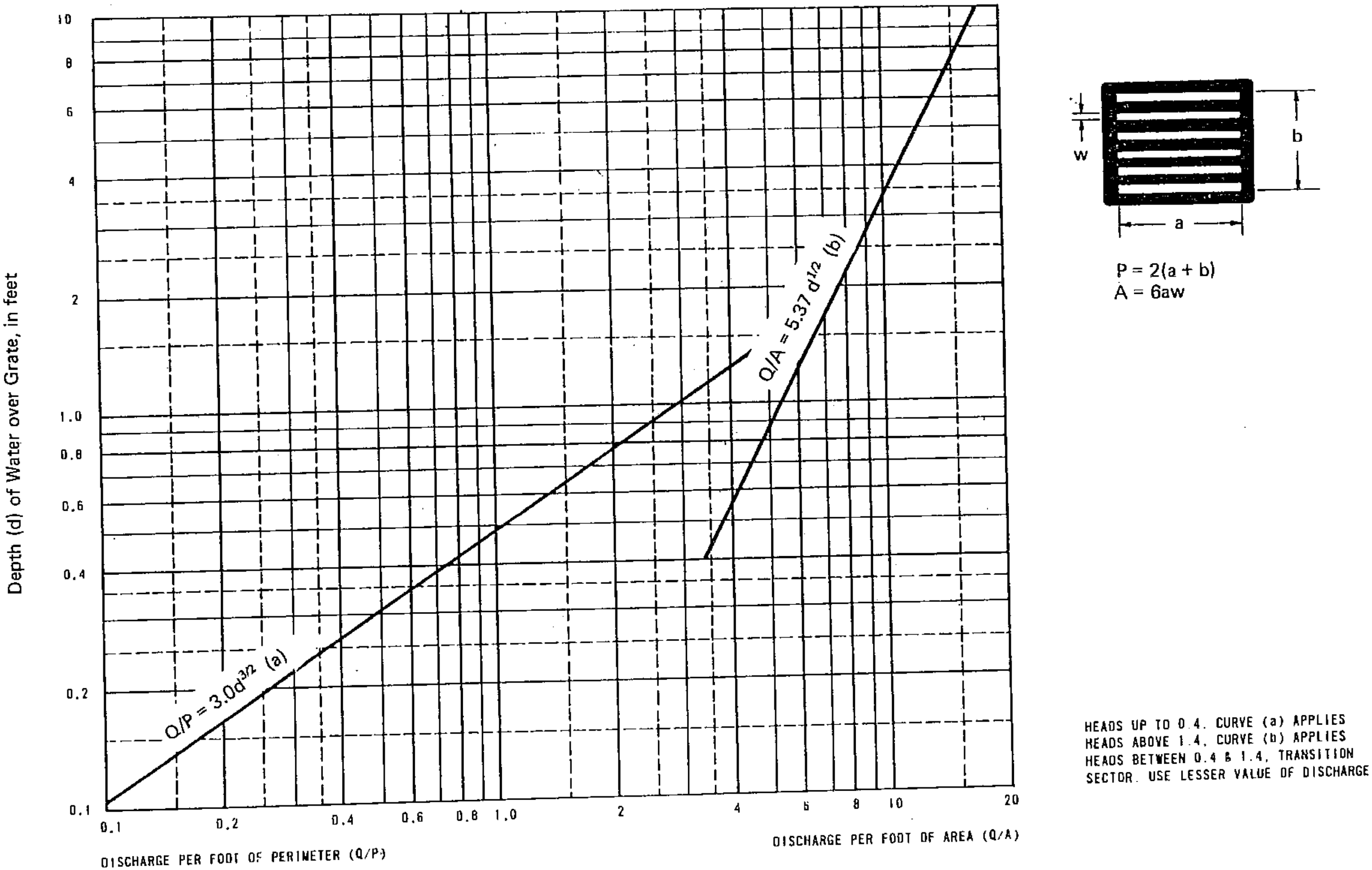
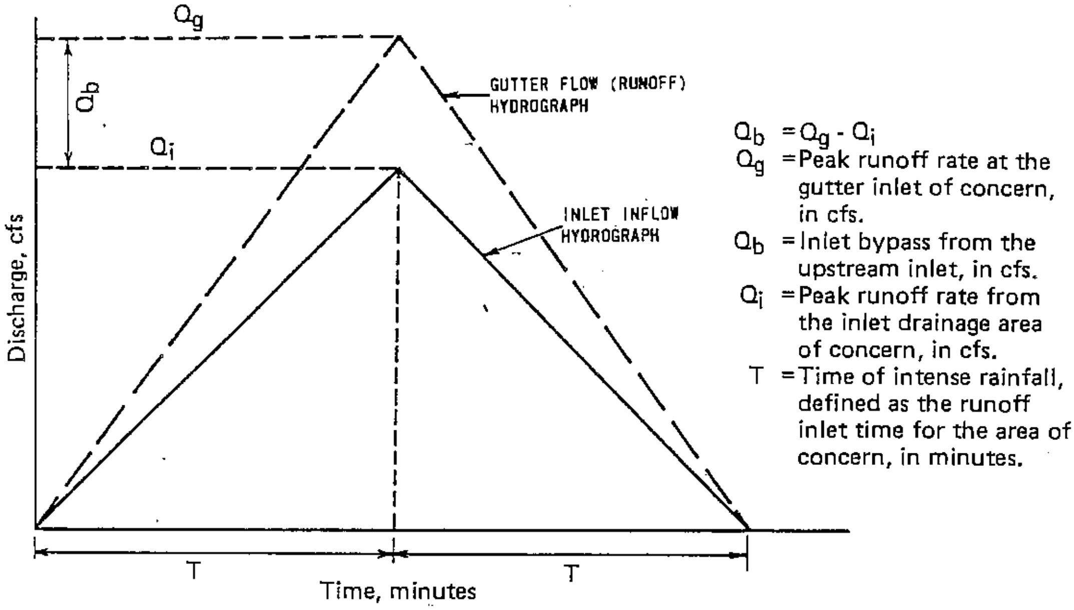
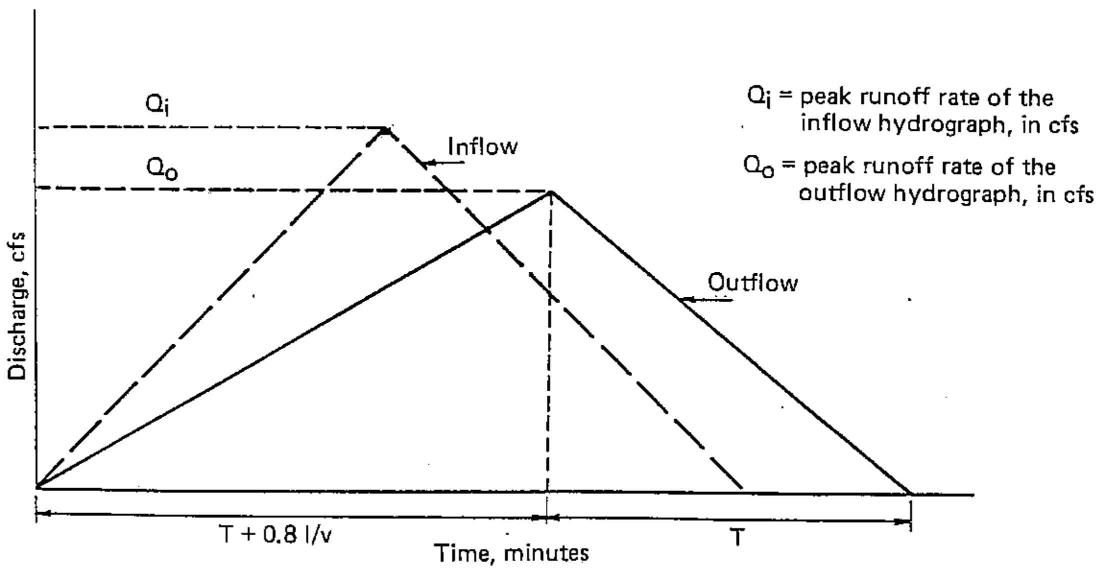


FIGURE 7-77. Capacity of grate inlets under sump conditions.



A) Inlet Hydrograph (Inflow)



B) Routed Inlet Hydrograph (Outflow)

FIGURE 7-78. Basic features of the inlet hydrograph.

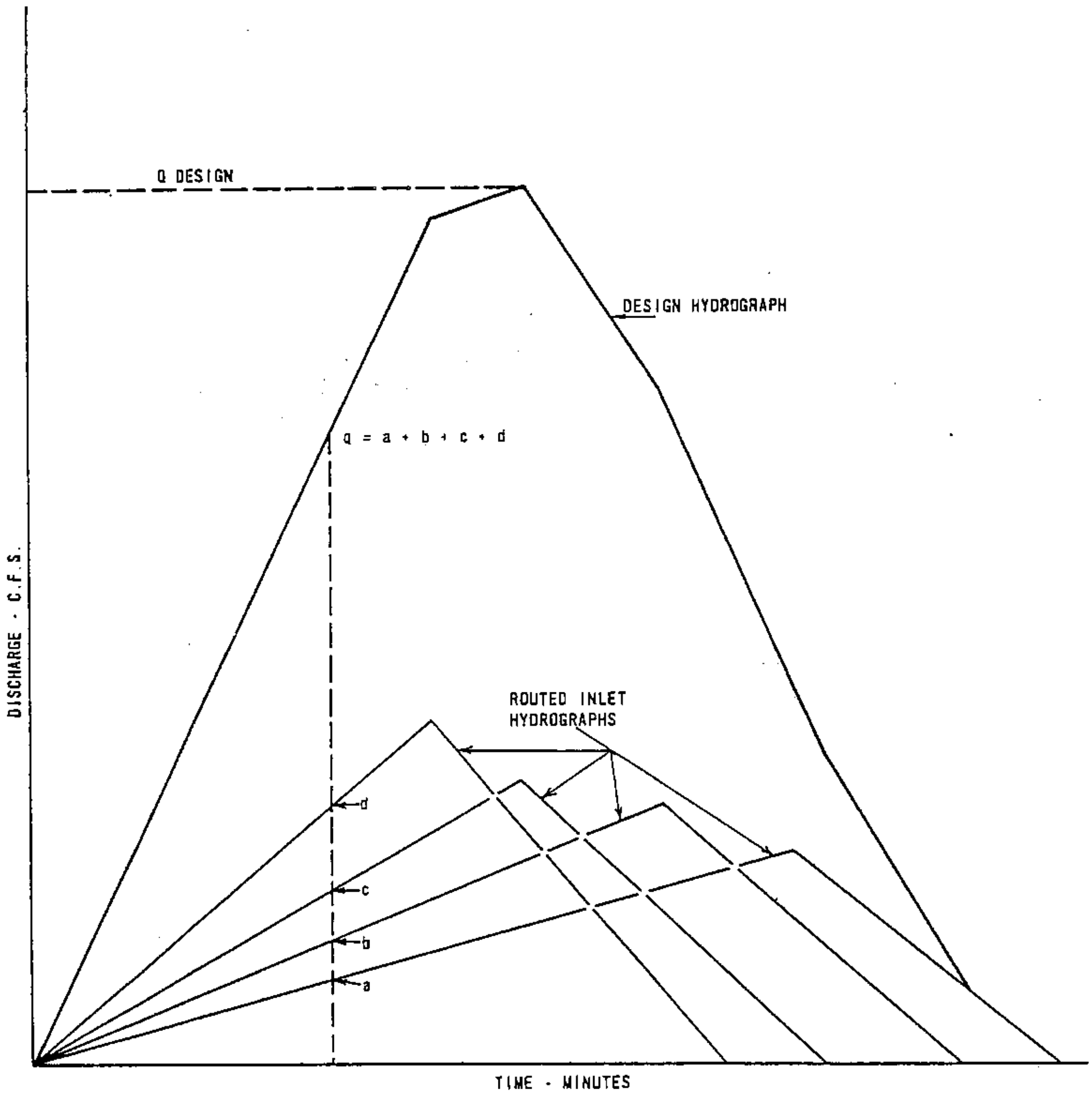


FIGURE 7-79. Synthesis of design hydrographs from routed inlet hydrographs.

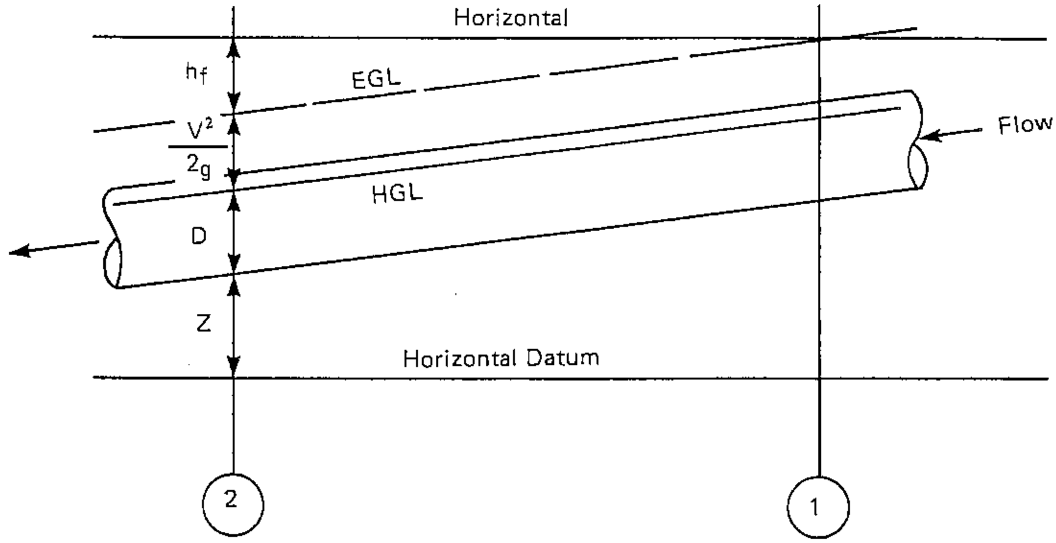


FIGURE 7-80. Open-channel flow in a closed conduit.

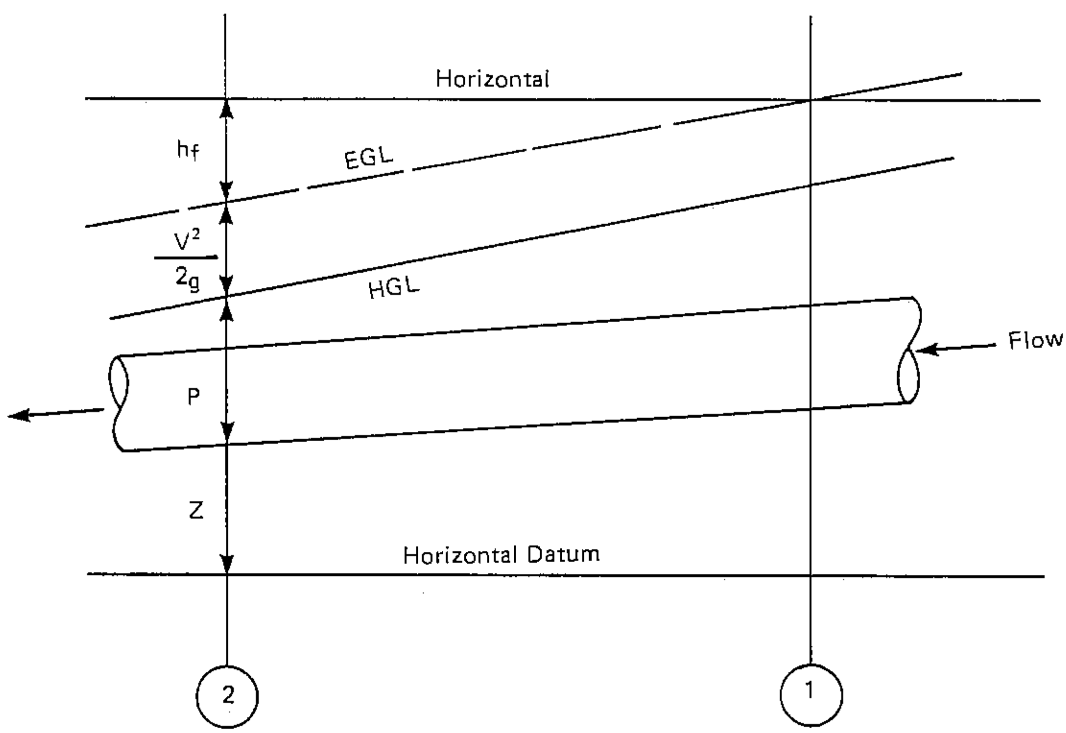


FIGURE 7-81. Pressure flow in a closed conduit.

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

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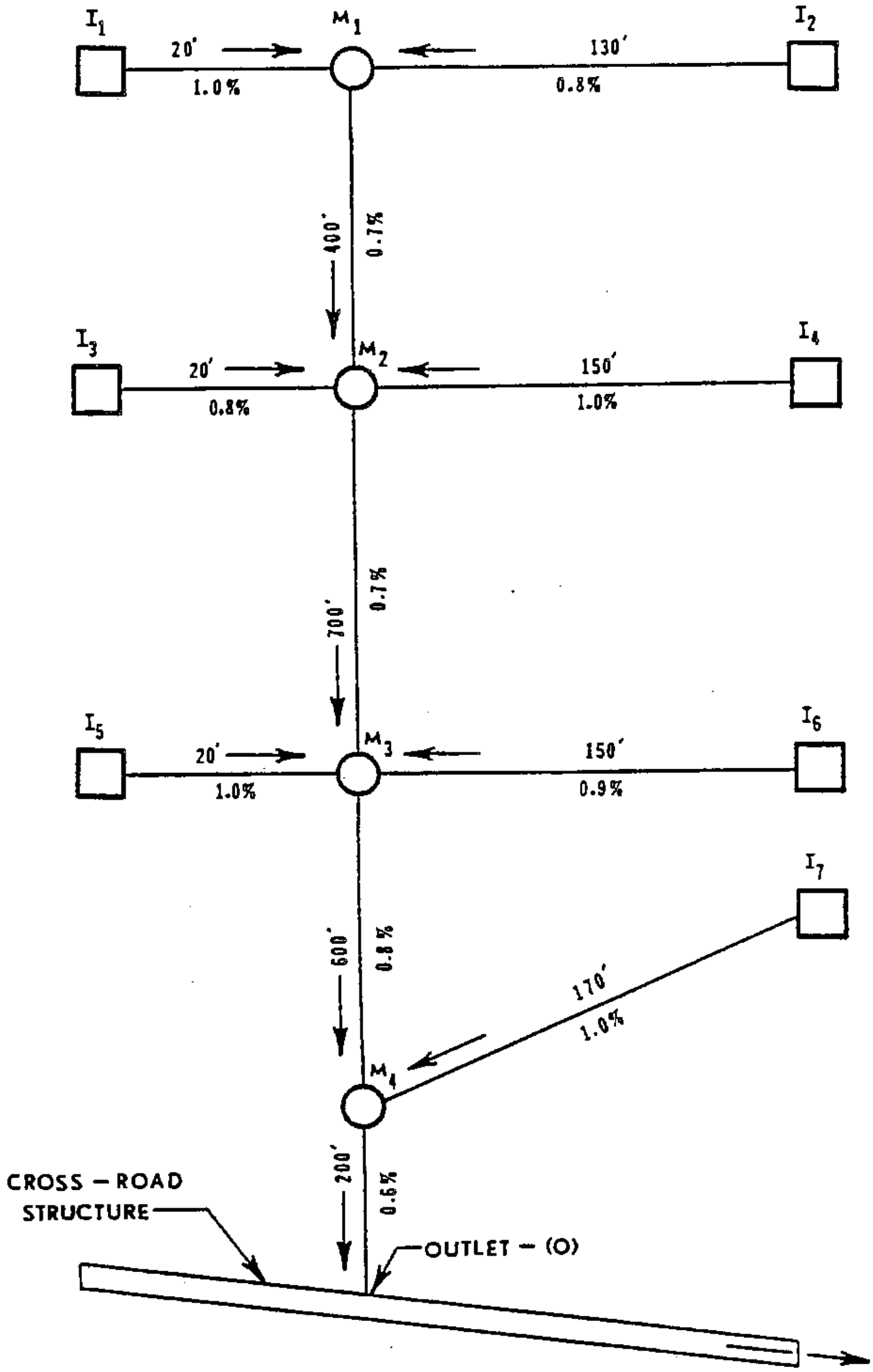

FIGURE 7-82. Example problem for storm sewer system layout.

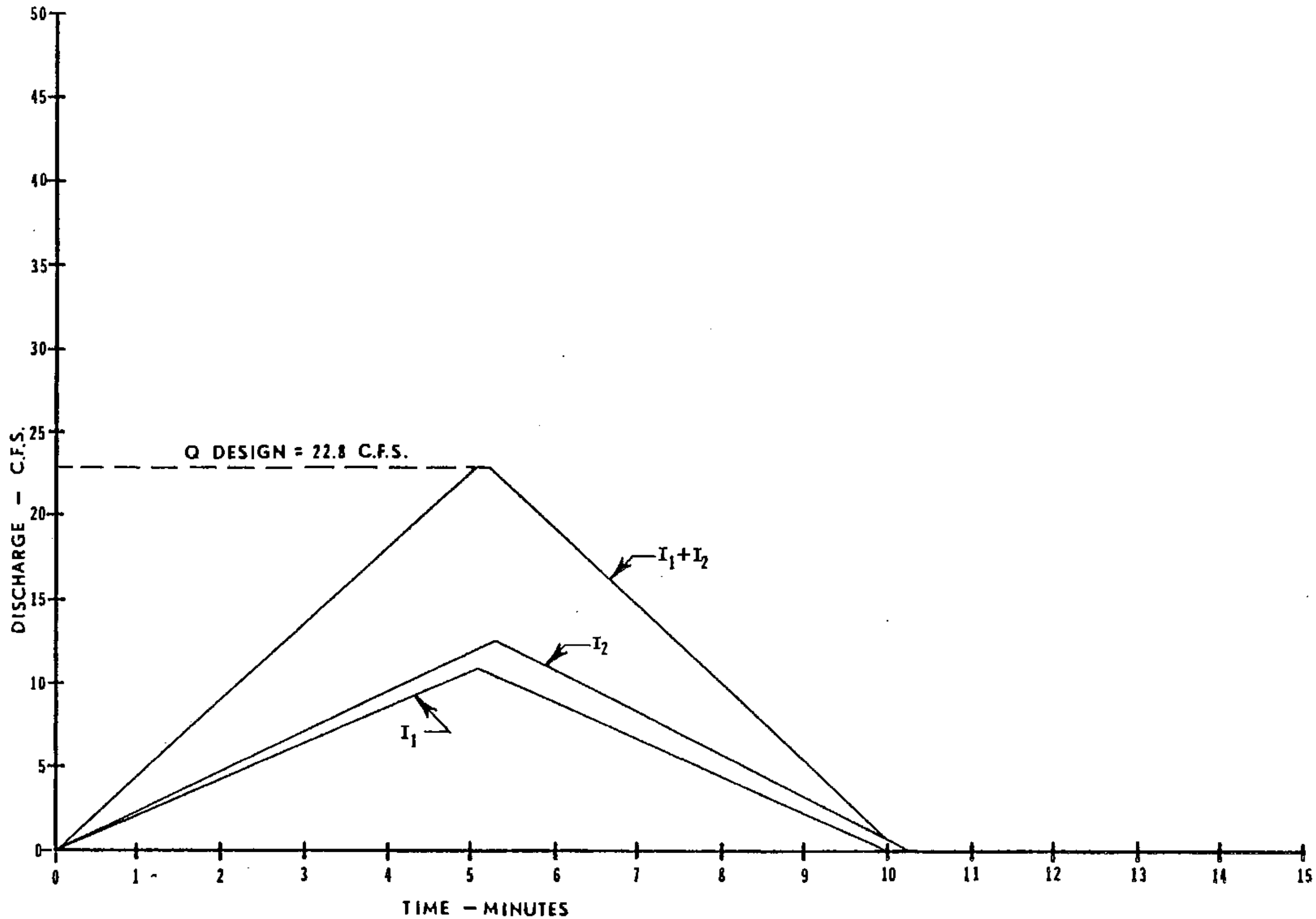

FIGURE 7-83. Inlet hydrograph at manhole 1.

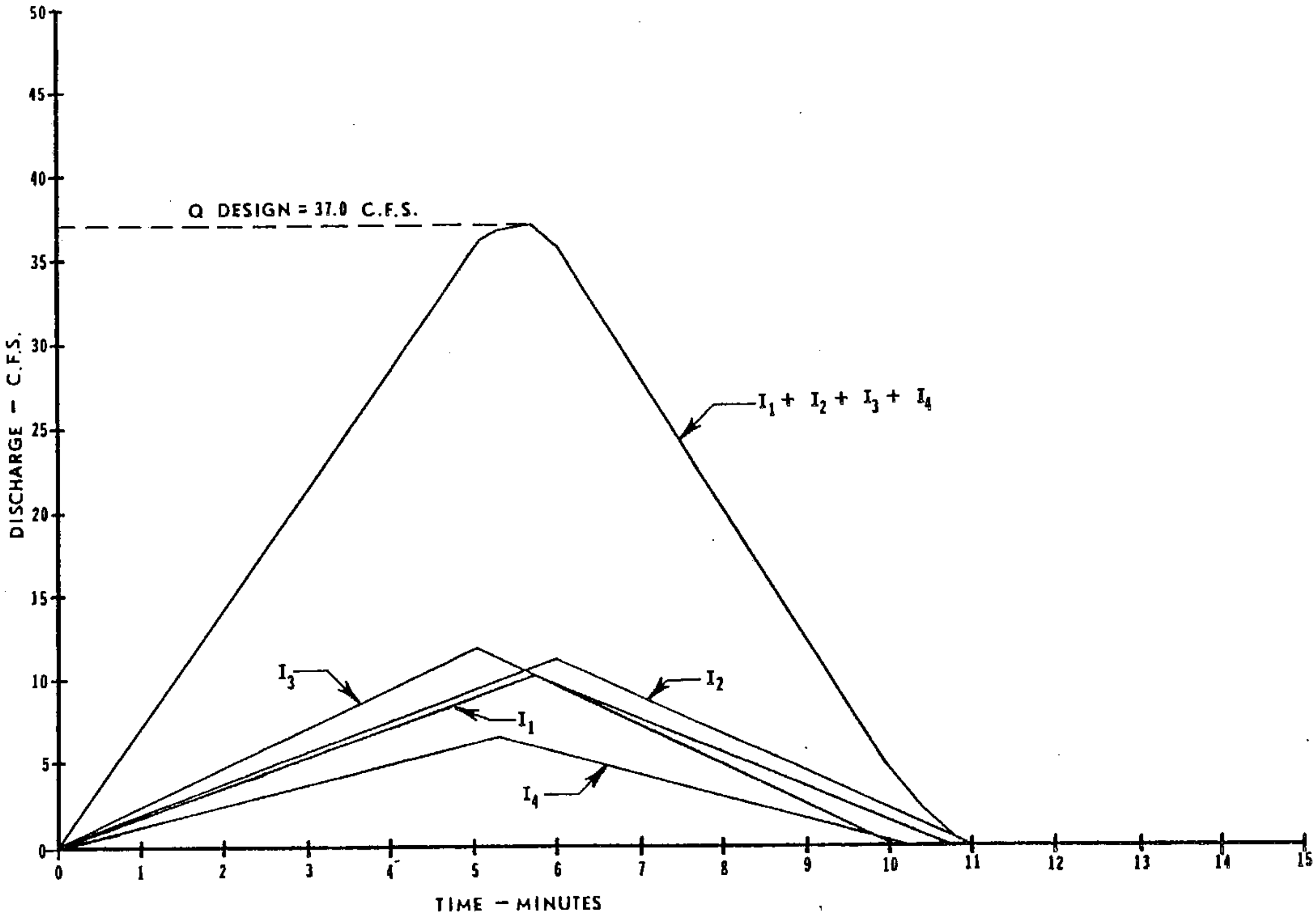

FIGURE 7-84. Inlet hydrograph at manhole 2.

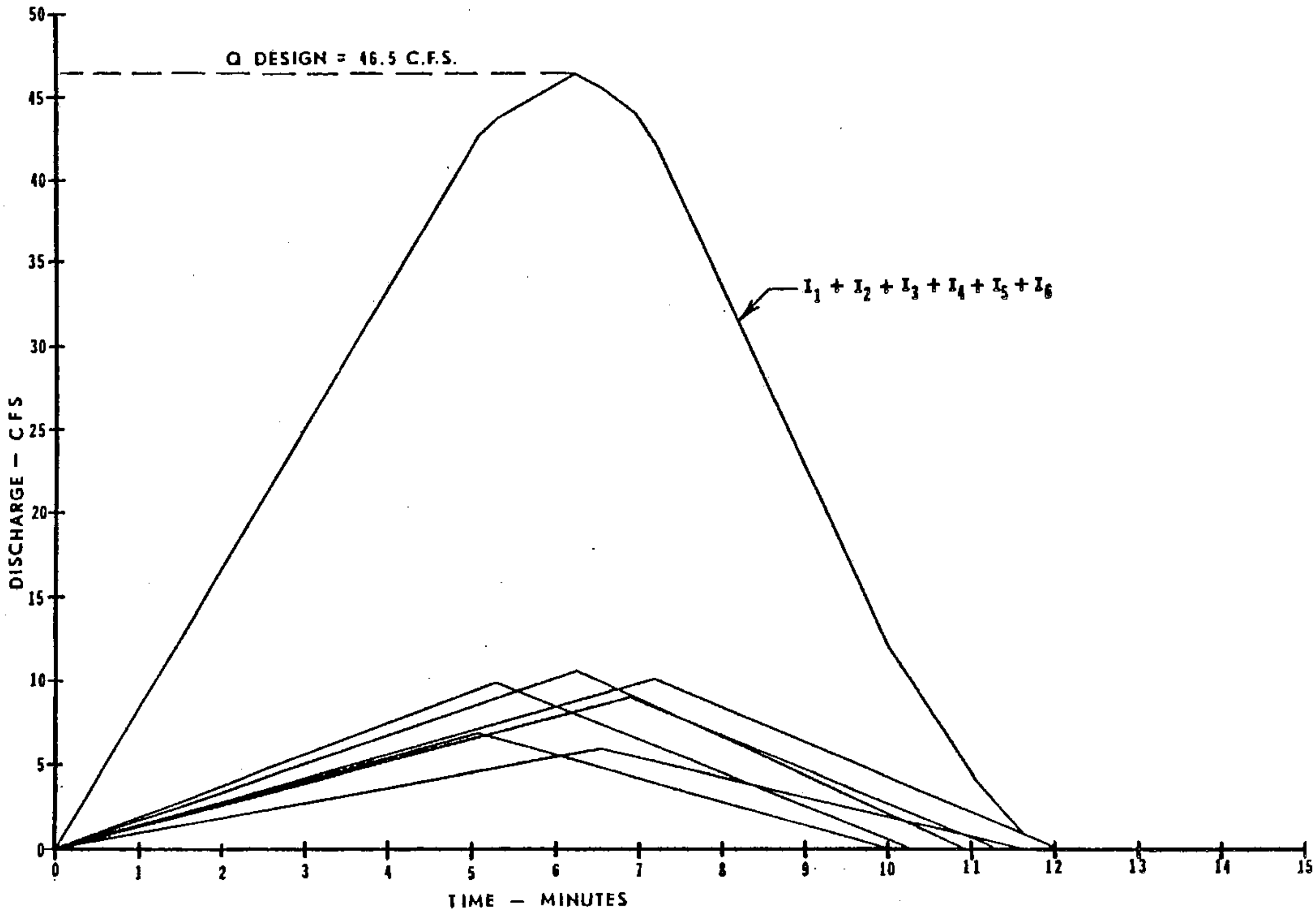

FIGURE 7-85. Inlet hydrograph at manhole 3.

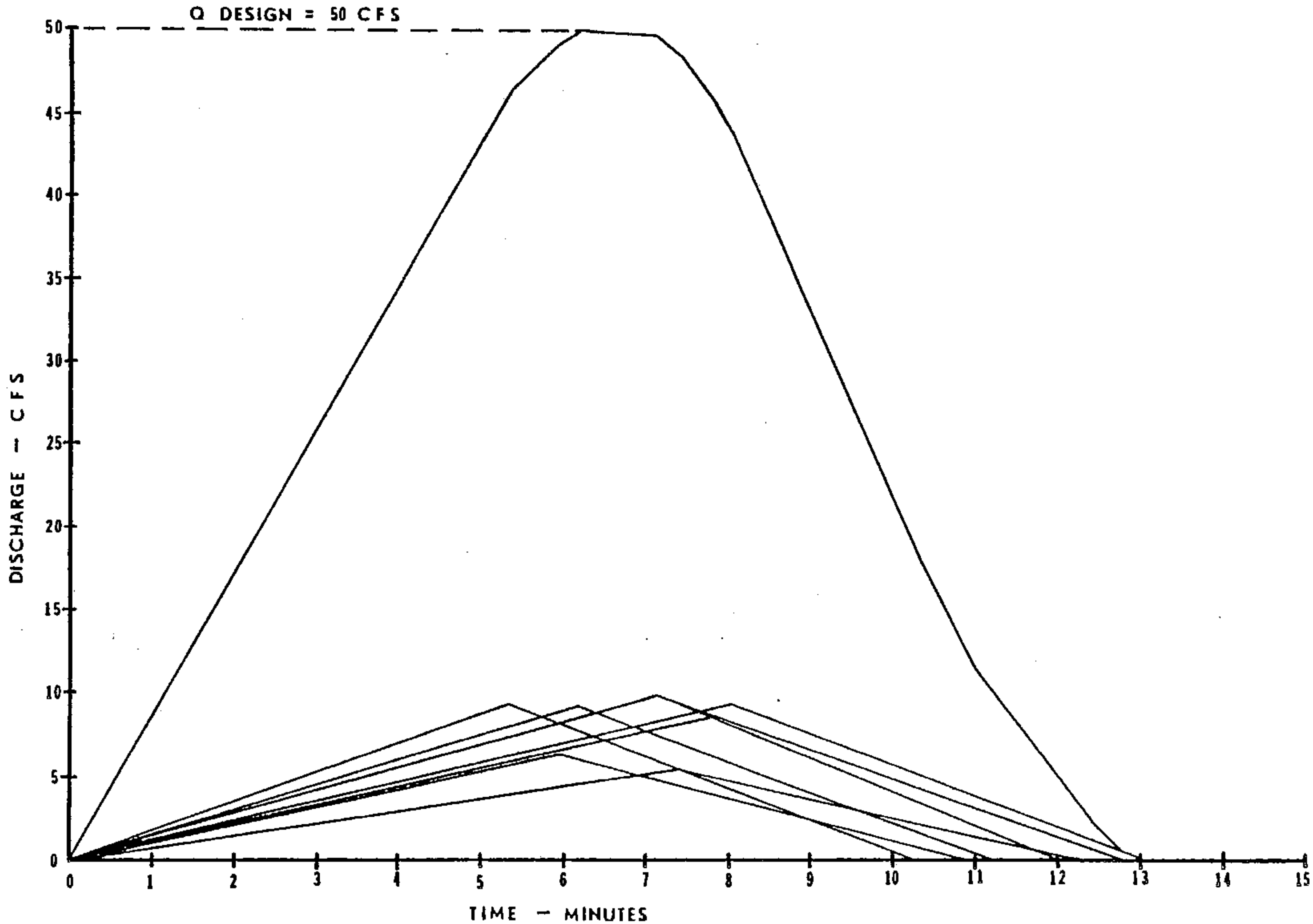

FIGURE 7-86. Inlet hydrograph at manhole 4.

SECTION 1.0 INTRODUCTION

A natural watershed generally features temporary stormwater storage widely distributed in small-volume components throughout the watershed. This natural temporary storage is usually reduced when development occurs. If the reduction is significant (e.g., flooding is increased) onsite stormwater storage measures may be required. Types of stormwater storage that are appropriate in the Montgomery urban area are identified in the first section of this chapter. Detailed information for conducting a storage reservoir routing is then presented. Chapter 8 concludes with a general hydraulic design procedure for stormwater storage systems.

SECTION 2.0 TYPES OF STORAGE

Three types of stormwater storage are considered in this section: runoff control Best Management Practices (BMP's), detention storage, and retention storage.

2.1 Runoff Control BMP's

The basic principle of providing stormwater storage by runoff control BMP's is to view stormwater as an asset to be utilized onsite, rather than as a liability to be rapidly transported from the area. In general, vegetated drainage swales (i.e., green areas) are maintained along natural drainage contours, and lakes or ponds (i.e., blue areas) are provided to temporarily or permanently store stormwater onsite. This philosophy of stormwater management is often called "blue-green" storage, and was proposed by Jones (1967).

Runoff control BMP's which can be utilized to replace natural onsite storage lost by development include porous pavement, concrete grid pavements, rooftop ponding, parking lot ponding, and percolation storage.

2.1.1 Porous Pavement. Although porous pavement has limited usefulness in Montgomery, where soil conditions limit infiltration rates, research and demonstration project installations of porous pavement have proven that it can be considered as a viable alternative to conventional pavement. A porous asphaltic concrete containing 5.5 percent asphalt by weight and aggregate graded to allow a water flow of 76 inches per hour has been found to be optimal for most applications. The design of porous asphalt concrete roadways depends primarily on the load-bearing capacity of the subgrade, the expected traffic volume, and the reservoir capacity of the surface and base. For most conditions, Thelen et al., (1972) found that the cost of conventional pavement with storm sewers was higher than the cost of an equivalent porous asphaltic concrete installation. Detailed information related to design are published by Thelen et al. (1972).

A list of typical applications for porous pavement is presented in Table 8-1. This table includes the loads, speeds, and frequencies of traverse estimated as acceptable traffic for each application. The first three applications of Table 8-1 deal with human traffic and the next two with passenger vehicle and light truck traffic. Three others include heavy truck loads, and the last two deal with airport traffic. The first five applications are subjected to relatively low traffic loads and therefore are more feasible at sites where a wet soil may be poor in load-bearing capacity.

A porous pavement parking lot was installed at The Woodlands, Texas (Characklis et al., 1979). Its performance was compared to a conventional, dense pavement parking lot, with the following conclusions:

1. Porous paving may be used effectively to store and release stormwater which would otherwise cause erosion or flash flooding. The quality of the released water is generally better than that of runoff from standard paving.

2. Unacceptable lead concentrations suggest that stored water under porous pavement should be prevented from contacting drinking water supplies.

3. Porous pavement is comparable to conventional paving in terms of driveability and safety. Mud and dirt from construction activities in the vicinity can clog porous paving.

4. Periodic maintenance of the paving should be performed by brush sweepers with vacuum, followed by high pressure water washing.

5. A more durable porous pavement can be produced by using lower penetration or stiffer asphalt than that used at The Woodlands.

<u>2.1.2 Concrete Grid Pavement</u>. Concrete grid pavements have been used extensively in Europe and are currently available from manufacturers throughout the U.S. A concrete grid pavement provides open area in the concrete to establish grass cover within the pavement structure. Thus, they are probably best suited to parking lot surfaces. A diagram of

selected lattice grid and castellated grid pavers are presented on Figure 8-1 and 8-2, respectively. As with porous pavement, concrete grid pavement will have limited usefulness in Montgomery, where soil conditions limit infiltration rates.

The results of recent research reported by Day (1980) indicate that the ability of a concrete grid paver to absorb and detain stormwater tends to be a function of surface geometry. The castellated surface geometry was found to absorb and detain stormwater better than the lattice geometry. Thus the runoff coefficient is lower for castellated pavers than for lattice pavers. A comparison of runoff coefficients for these two concrete grid pavers and other urban surfaces is reported on Figure 8-3.

2.1.3 Rooftop Ponding. The structural capability of the roof system must be considered when designing a temporary rooftop ponding system. A 3-inch water depth is equivalent to a load of 15.6 pounds per square foot; overflow mechanisms should be provided so that there is no danger of overloading a building structure during major storms. Special considerations of roof watertightness may be necessary since storage may be effective only if the water is detained for a significant period of time. Many flat roofs already pond significant amounts of water, although they may not have been designed for this purpose. Existing rooftop ponding should be considered when evaluating drainage conditions in established urban areas.

2.1.4 Parking Lot Ponding. Parking lot ponding should be arranged so that pedestrians can reach their destinations without walking through ponded water. The ponding should be designed for those portions of the parking area farthest from the destination or to overflow parking areas. In addition, only a portion of the total area should be designed for ponding so that a reasonable parking area remains available for use.

The maximum depth of ponding allowed during the design event can vary by location. A 7-inch design depth is not unreasonable where access to parked vehicles will not be impaired. Provision for emergency overflow during storms greater than the design storm should be provided. Periodic cleaning of outlet drains is required for the parking lot storage to be effective.

2.1.5 Percolation Storage. Runoff which is discharged into trenches that are designed as graded percolation filters can provide recharge to a surface ground-water aquifer if the aquifer standing level is below the trench elevation. Percolation tests must be run on the stratum at the bottom of the proposed trench. An inflow rate which is greater than the percolation rate will result in temporary storage in the voids of the filter material.

Two important design considerations for percolation storage are (1) the potential effects of clogging in the voids, and (2) the reduced load-carrying capacity of pavement subbase in a saturated condition. In addition, provisions for emergency overflow during storms greater than the design storm should be included.

Percolation storage can also be provided if open joints or perforations are placed in a storm sewer system. Factors discussed above should be evaluated to identify appropriate conditions for storm sewer exfiltration.

2.2 Storage Basins

Stormwater storage basins can be installed to reduce peak runoff rates, aid in the recharge of ground water, and reduce downstream flooding, stream erosion, and sedimentation. Storage basins can be designed to provide temporary or permanent storage of stormwater. Temporary stormwater storage is commonly called "detention storage," while permanent stormwater storage is commonly called "retention storage." A schematic diagram of these two types of stormwater storage is presented on Figure 8-4. Important factors to consider when designing either detention or retention storage basins are briefly discussed below.

2.2.1 Detention Storage. A detention storage basin is designed to completely drain after a storm has passed (see Figure 8-4, Part A). The attenuation of a peak inflow rate can be determined only by performing a reservoir routing. In general, a detention basin is designed by determining the spillway combination which reduces the peak flow to a desired value given the basin size and design storm return period. Details concerning reservoir routing are presented in Section 3.0 of this chapter. Theoretically, detention storage should provide a peak flow reduction which is a maximum for the basin size available. In practice, this maximum can be obtained only if the basin is dry prior to the storm. When the time between large storms is small, the peak flow reduction may be decreased. In addition, the timing of peak flows is changed such that downstream peaking conditions could be aggravated. It is therefore important for the hydraulic impact of a storage basin to be properly planned such that the intended functions are accomplished. Improperly located storage may create new flooding problems or aggravate others.

2.2.2 Retention Storage. A retention storage basin is designed to have a permanent pool of standing water. Therefore, for a given storage basin size, the peak flow reduction caused by retention storage will be less than the reduction caused if the basin were designed to provide detention storage. This characteristic of retention storage is illustrated on Figure 8-4, Part B.

Retention basins can allow multiple uses for the basin, e.g., possibilities for boating, fishing, swimming, and establishing a wildlife habitat. In addition, property values in the area may be enhanced. In view of these multiple uses, proper maintenance and protection from health and safety hazards and positive control of visual appearance must be integral parts of storage design.

A potential retention basin site should be evaluated to ensure that the water balance is favorable. This involves conducting soil percolation tests and locating sufficient natural or artificial inflow to replace evaporation and percolation losses. Additional maintenance considerations include the potential for eutrophication, the effect of dry spells, siltation, and erosion.

The peak flow reduction characteristics of a retention basin are determined using the same procedures discussed above for detention basins. For example, a reservoir routing may be performed to size the basin spillways such that a desired peak outflow is obtained. For a given basin size the peak flow reduction obtained from a retention basin is theoretically less than that obtained from a detention basin (see Figure 8-4).

SECTION 3.0 RESERVOIR ROUTING

For the purposes of this manual, the primary function of a stormwater storage system is to reduce the peak flow of a hydrograph to a desired value. To determine the peak flow reduction obtained by a stormwater storage system, a reservoir routing procedure is required. Three approaches to reservoir routing are presented in this section. The first, the Modified Rational Method, is applicable only to drainage ~~storage~~ areas less than 20 acres. The second is a graphical procedure, developed by the SCS, which provides a quick preliminary routing for watersheds greater than 20 acres. The third is a hydrologic routing procedure called the "Storage Indication Method," which is to be used in the final design of any basin with a watershed area greater than 20 acres.

3.1 Modified Rational Method (areas less than 20 acres)

As discussed in Subsection 5.2 of Chapter 5, the Rational Method was developed for the sole purpose of predicting peak flow rates from small watersheds. In practice, if the watershed area tributary to a storage basin is less than 20 acres, the Rational Method can be manipulated to provide a reasonable estimate of the maximum storage volume required to obtain a desired peak flow reduction. This manipulated form, commonly called the Modified Rational Method, accounts for the fact that storms with durations greater than the time of concentration for a basin produce a larger volume of runoff with reduced peak flows. The correct sizing of a stormwater retention basin should be based on the storm duration which has the largest volume difference between the inflow and outflow hydrographs.

Once a design storm return period has been selected, a family of runoff hydrographs can be developed using the Rational Method according to the procedure described in Subsection 5.2.2 of Chapter 5. A family of Rational Method hydrographs is shown on Figure 5-15. The rising and falling limb of each of these hydrographs are equal to the time of concentration for that watershed. The volume of runoff or area under each of these hydrographs is calculated using the following equation:

$$V = 0.5\ H\ (B_1 + B_2) \qquad (8\text{-}1)$$

where

V = volume of a trapezoidal hydrograph (see Figure 5-15)

H = peak flow or height of the trapezoidal hydrograph, in cfs

B_1 = bottom time base for the trapezoidal hydrograph, in seconds (segment AD on Figure 5-15)

B_2 = top time base for the trapezoidal hydrograph, in seconds (segment XY on Figure 5-15)

Once an allowable peak flow rate from the storage basin has been determined, the largest volume difference between the inflow and outflow hydrographs can be determined using equation 8-1. The allowable release rate may be the peak flow from the site prior to development or the peak flow which does not cause flooding at a particular downstream control section. If the post-development drainage area is less than the pre-development conditions, the pre-development area should be used to determine the maximum allowable release rate.

For hydrologic flood routing, the maximum release rate will always occur at the point where the outflow hydrograph crosses the receding limb of the inflow hydrograph. Therefore, the maximum release rate is placed at point B on the falling limb of the inflow hydrograph shown on Figure 5-15. By assuming a constant release rate past this point, the critical storage volume is found by determining the maximum difference between the inflow and outflow hydrographs. This procedure is illustrated in Table 8-2 for the hydrographs presented on Figure 5-15. In this case the critical storage volume was found to be 25,760 ft^3 for a 30-minute storm.

3.2 SCS Preliminary Method

For watershed areas greater than 20 acres a hydologic routing procedure should be utilized for designing a stormwater storage system. A quick graphical method published by the SCS (1975) can be used for preliminary design purposes when the watershed area is greater than 20 acres. This method is based on application of the storage indication method for numerous field situations. The preliminary SCS method can analyze the peak release rate from principal spillways only. Emergency spillway flow is not considered.

Two figures are utilized in the SCS preliminary routing method. The first relates the volume of runoff, or storage basin inflow, to the storage volume required to obtain a range of release rates in cfs per square mile of drainage area per inch of runoff (csm) (see Figure 8-5 at the end of this chapter). It is applicable only for weir outflow structures with release rates up to 150 csm and for pipe outflow structures with release rates up to 300 csm.

The second figure of the SCS preliminary method is used when the release rate exceeds 150 csm for weir outflow structures and 300 csm for pipe outflow structures. This figure relates the peak outflow to inflow ratio of the basin to the storage to runoff volume ratio (see Figure 8-6 at the end of this chapter).

The accuracy of the curves on Figures 8-5 and 8-6 depends on the relationship between the storage available, the inflow volume, and the shape of the inflow hydrograph. When only a small volume is available for detention storage, the shape of the outflow hydrograph is very sensitive to the rate of rise of the inflow hydrograph. Conversely, when a large detention storage volume is available, the shape of the inflow hydrograph has little effect on the outflow hydrograph. Therefore, parameters such as runoff curve number and time of concentration, which affect the rate of rise for a hydrograph, become significant parameters in analyzing stormwater storage.

The peak inflow rate is not a factor for determining detention storage requirements using Figure 8-5. As shown on Figure 8-5, the ratio of storage volume to runoff volume is relatively high. Therefore, the inflow peak is not a significant parameter. Figure 8-5 is usually accurate within 5 percent for release rates under 100 csm and within 10 percent for release rates over 100 csm.

The ratio of peaks to volumes is presented on Figure 8-6. For these conditions, hydrograph shape parameters are important. If the runoff curve number is less than 65, combined with short watershed t_c values, the volume ratio read from the curve will be up to 25 percent too high. For curve numbers over 85 with long watershed t_c values, the volume ratio will be up to 25 percent too low.

The steps for performing the SCS preliminary reservoir routing are as follows:

1. Determine the basic watershed parameters (DA, CN, t_c, etc.).
2. Determine the volume of runoff and peak rate of flow from the watershed.
3. Set the desired rate of outflow from the structure.
4. Determine the required volume of storage from the appropriate figure (8-5 or 8-6).
5. Proportion the storage structure so that the design outflow rate and maximum storage occur at the same stage.
6. Size the emergency spillway.

In general, the SCS preliminary method should be used only to develop input data for the storage-indication method, presented in Section 3.3. Note that in Steps 3 and 4, the storage volume could be set and the resulting rate of outflow determined from Figures 8-5 and 8-6. For structures with drainage areas over 2,000 acres and for events of less than 2-year frequency, other reservoir routing procedures should be used. Example 8-1 at the end of this section illustrates the SCS procedure.

3.3 Storage-Indication Method

In order to utilize the storage-indication method the following three basic relationships must be established:

1. Inflow Hydrograph

2. Stage-Storage Curve

3. Stage-Discharge Curve

Development of each of these relationships is briefly described below, followed by a detailed discussion of routing computations and a reservoir routing design procedure.

3.3.1 Inflow Hydrograph. Procedures for developing synthetic runoff hydrographs were presented in Subsection 5.5 of Chapter 5. The reservoir routing calculations cannot be performed until an inflow hydrograph has been developed using appropriate site-specific data. Therefore, the development of an inflow hydrograph can be considered the first step in the storage-indication reservoir routing method.

3.3.2 Stage-Storage Curve. A stage-storage curve defines the relationship between the depth of storage and volume in the detention basin. An example stage-storage curve is shown in Figure 8-7. The data for this curve were developed using a topographic map and the double-end area method to estimate the storage volume. The double-end area method is expressed mathematically as follows:

$$V_{1,2} = \frac{A_1 + A_2}{2} d \qquad (8\text{-}2)$$

where

$V_{1,2}$ = storage volume, in ft^3, between elevations 1 and 2

A_1 = surface area at elevation 1, in ft^2

A_2 = surface area at elevation 2, in ft^2

d = change in elevation between points 1 and 2, in ft

3.3.3 Stage-Discharge Curve. A stage-discharge curve defines the relationship between the storage depth and the discharge from a storage basin. As illustrated on Figure 8-4, Part B, a typical stormwater storage basin has two spillways : principal and emergency. The principal spillway is usually designed with a capacity sufficient to convey the design storm without allowing flow to enter the emergency spillway. A pipe is generally used for the principal spillway. Since this pipe operates hydraulically in a manner identical to a culvert through an embankment, a stage-discharge curve for the principal spillway can be developed using the culvert nomographs presented in Section 3.0 of Chapter 7.

The emergency spillway is generally sized to provide a bypass for stormwater during a storm which exceeds the design capacity of a reservoir. Selecting a magnitude for sizing the emergency spillway should depend on the threat to downstream life and property, should the detention storage embankment fail. A broad-crested weir is generally the type of structure utilized for an emergency spillway. The stage-discharge curve of a broad-crested weir is expressed mathematically as follows:

$$Q = C\ L\ H^{3/2} \tag{8-3}$$

where

Q = discharge, in cfs

C = weir coefficient

L = length of the weir, in feet

H = height or head of water above the weir elevation, in feet

A typical value of the weir coefficient for a broad-crested weir is 3.0. Detailed information for determining specific values of the weir coefficient for various weir configurations is presented by Brater and King (1976).

In cases where culvert and broad-crested weir hydraulic relationships are not appropriate, more detailed information on stage-discharge relationships should be obtained. Two good sources for this information include the hydraulics handbook by Brater and King (1976) and the design text on small dams by the U.S. Department of the Interior, Bureau of Reclamation (1973). An example stage-discharge curve is presented on Figure 8-8.

3.3.4 Routing Computations. As discused in Section 6.0 of Chapter 5, hydrologic flood routing techniques are all based on the continuity equation, expressed as equation 5-35. A finite difference approximation to equation 5-35 can be expressed mathematically as follows:

$$\left[\frac{I_1 + I_2}{2}\right] - \left[\frac{O_1 + O_2}{2}\right] = \frac{S_2 - S_1}{\Delta t} \tag{8-4}$$

where

I_1 and I_2 = inflow rates at times 1 and 2, respectively, in cfs

O_1 and O_2 = outflow rates at times 1 and 2, respectively, in cfs

S_1 and S_2 = Storage volumes at times 1 and 2, respectively, in ft^3

Δt = time change between periods 1 and 2, in seconds

By rearranging equation 8-4, the storage-indication method utilizes the following equation to perform a reservoir routing:

$$S_2 + \frac{O_2}{2}\Delta t = \left[S_1 - \frac{O_1}{2}\Delta t\right] + \left[\frac{I_1 + I_2}{2}\Delta t\right] \qquad (8\text{-}5)$$

The only unknown of equation 8-5 for any time increment is the left-hand side of the equation. To simplify calculations a pair of storage characteristics curves can be developed which provide a direct determination of O_2 and S_2 given the value of the right-hand side of the equation. The storage characteristics curves are developed using appropriate storage-discharge data for the basin and spillway configuration being analyzed. An example tabulation of storage, discharge, $S - O/2\ \Delta t$, and $S + O/2\ \Delta t$ is presented in Table 8-3. The resulting storage characteristics curves are shown on Figure 8-9, which plots $S \pm O/2\ \Delta t$ on the ordinate and stage or depth of storage on the abscissa.

The following computations are required to peform a reservoir routing by the storage-indication method:

1. 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, $S_1 - O_1/2\ \Delta t$ can be determined from the appropriate storage characteristics curve on Figure 8-8.

2. Determine the value of $S_2 + O_2/2\ \Delta t$ by adding

   $$\frac{I_1 + I_2}{2}\Delta t \quad \text{and} \quad S_1 - \frac{O_1}{2}\Delta t$$

   which were determined in Step 1.

3. Enter the appropriate storage characteristics curves at the value of $S_2 + O_2/2\ \Delta t$ determined in Step 2 and read off a new depth of storage, H_2.

4. Determine the value of O_2 which corresponds to a stage of H_2 determined in Step 3, using the stage-discharge curve, Figure 8-7.

5. Repeat Steps 1 through 4 by setting new values of I_1, O_1, S_1 and H_1 equal to the previous I_2, O_2, S_2, and H_2 and using a new I_2 value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

A routing time period, Δt, is generally selected to provide at least five points on the rising limb of the inflow hydrograph.

3.4 Example Problems

Example 8-1. SCS Preliminary Reservoir Routing Computations

Given the following data, use the SCS preliminary reservoir routing method to size a storage basin.

Drainage area = 75 acres

Q_{25} (existing) = 180 cfs

Q_{25} (future) = 360 cfs

Future runoff = 3.4 inches

Principal spillway is to be a pipe flow structure.

1. For this example 180 cfs is the desired outflow and 360 cfs is the discharge into the reservoir for future conditions. The inflow runoff is 3.4 inches.

2. $Q_0 = \frac{(180)(640)}{(75)} = 1{,}536$ csm

   Since Q_0 is greater than 300 csm, use Figure 8-6.

   Compute $\frac{Q_0}{Q_i} = \frac{180}{360} = 0.5$

with $\frac{Q_o}{Q_i} = 0.5$ enter Figure 8-6 on the abscissa, and

find $\frac{V_s}{V_r} = 0.28$

since $V_r = 3.4$ inches,

$$V_s = (0.28)(3.4) = 0.95 \text{ inches}$$

$$V_s = \frac{(0.95)(75)}{(12)} = 5.9 \text{ acre-feet}$$

3. The crest of the emergency spillway should be set at the elevation which provides 5.9 acre-feet of storage, and the principal spillway should be sized to carry 180 cfs at this elevation.

4. Check the preliminary storage size using the storage indication method, and then size the emergency spillway for at least the 100-year design storm.

Example 8-2. Reservoir Routing Using the Storage-Indication Method

The stage-storage and stage-discharge curves presented on Figures 8-7 and 8-8, respectively, provide a peak outflow of 180 cfs for 5.9 acre-feet of storage as required for example 8-1. Given the inflow hydrograph tabulated in columns 1 and 2 below, find the outflow using the storage-indication method.

1. Develop storage characteristics curves using Figures 8-7 and 8-8. These computations are tabulated in Table 8-3.

2. Using the data tabulated in column 2 on the next page, calculate:

$$\frac{(I_1 = I_2)\Delta t}{2}$$

and tabulate these values in column 3 below.

3. Given that $S_1 - \frac{O_1}{2} \Delta t = 0.05$ acre-ft for $H_1 = 0$ ft, find $S_2 = \frac{O_2}{2} \Delta t$ by summing 0.05 + 0.01 (column 5 value plus column 3 value) and tabulate 0.06 acre-ft in column 6).

4. Enter the $S = \frac{O}{2}\Delta t$ storage characteristics curve on Figure 8-9 and read the stage at the value of 0.06 acre-ft. This value is found to be 100.10 ft and is tabulated in column 7 below.

5. Using the stage of 100.10 ft found in Step 4, enter the stage-discharge curve (Figure 8-8) and find the discharge corresponding to that stage. In this case 0 is approximately 1 cfs and is tabulated in column 8 below.

6. Assign the value of H_2 to H_1, find a new value of $S_1 - \frac{O_1}{2}\Delta t$ from Figure 8-9, and repeat the calculations for Steps 3, 4, and 5 above. Continue repeating these calculations until the entire inflow hydrograph has been routed through the storage basin.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time (min)	Inflow (cfs)	$\frac{(I_1 = I_2)\Delta t}{2}$ (acre-ft)	H_1 (ft)	$S_1 - \frac{O_1}{2}\Delta t$ (acre-ft)	$S_2 + \frac{O_2}{2}\Delta t$ (acre-ft)	H_2 (ft)	0 (cfs)
0	0						
10	2	0.01	0	0.05	0.06	100.10	1
20	27	0.20	100.10	0.06	0.26	101.10	16
30	130	1.08	101.10	0.21	1.29	102.20	41
40	300	2.96	102.20	0.61	3.57	104.10	100
50	360	4.55	104.10	2.20	6.75	105.60	175
60	289	4.47	105.60	4.40	8.87	106.25	217
70	194	3.33	106.25	5.80	9.13	106.30	220
80	133	2.25	106.30	5.90	8.15	106.05	205
90	91	1.54	106.05	5.30	6.84	105.65	177
100	61	1.05	105.65	4.50	5.55	105.10	147
110	37	0.67	105.10	3.60	4.27	104.50	116
120	20	0.39	104.50	2.70	3.09	103.80	87
130	11	0.21	103.80	1.90	2.11	103.05	64
140	5	0.11	103.05	1.18	1.30	102.25	43
150	1	0.04	102.25	0.63	0.67	101.40	22
160	0	0	101.40	0.35	0.35	100.70	10

SECTION 4.0 DESIGN PROCEDURE

Three interdependent design variables must be considered when a stormwater detention basin is sized:

1. Design storm return period
2. Peak outflow from the basin
3. Storage capacity of the basin

Once any two of these design variables are known, the third is automatically fixed. Two classes of design problems are generally encountered which have the common objective of lowering the peak outflow rate to an acceptable value. In one type of design problem, the basin storage capacity and design storm return period are known and the outlet works must be sized to obtain the minimum peak outflow from the basin. This is referred to as the "outlet works design problem" in this manual. The other typical design problem must determine the storage volume required to obtain the desired peak outflow rate for a given design storm. This is referred to as the "storage volume design problem" in this manual. Since the solution to each of these design problems must be obtained by a trial and error procedure, preliminary design procedures for each of these problems are presented below.

4.1 Outlet Works Preliminary Design

For a given basin size and design storm return period, the peak flow reduction is fixed by the requirement that the basin does not overflow during passage of the design storm. The solution to this problem requires a trial and error solution since only the stage-storage curve is known explicitly. A preliminary estimate of the potential peak rate reduction for a given storage volume can be obtained from the following equation developed by Wycoff and Singh (1976).

$$\frac{Q_o}{Q_i} = 1 - 0.712 \left(\frac{V_s}{V_r}\right)^{1.328} \left(\frac{t_b}{t_p}\right)^{0.546} \qquad (8\text{-}6)$$

where

Q_o = outflow peak flow, in cfs

Q_i = inflow peak flow, in cfs

V_s = volume of storage, in inches

V_r = volume of runoff, in inches

t_b = time base of the inflow hydrograph, in hours, determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak

t_p = time to peak of the inflow hydrograph, in hours

An alternative to equation 8-6 is the SCS Preliminary Reservoir Routing Method presented in Subsection 3.2 of this chapter.

The final design of a peak outflow problem requires application of the storage-indication method as described in Subsections 3.3 and 4.3.

4.2 Storage Volume Preliminary Design

For this problem only the design storm return period and peak outflow from the detention basin are known. In this case, a trial and error solution is required to obtain the storage volume required. A preliminary estimate of the storage volume required can be obtained from the following equation developed by Wycoff and Singh (1976):

$$\frac{V_s}{V_r} = \frac{1.291\left(1 - \frac{Q_o}{Q_i}\right)^{0.753}}{\left(\frac{t_b}{t_p}\right)^{0.411}} \qquad (8\text{-}7)$$

where all terms are as defined above for equation 8-5.

An alternative to equation 8-7 is the SCS Preliminary Reservoir Routing Method presented in Subsection 3.2 of this chapter.

The final design of a storage volume problem requires the application of the storage-indication method as described in Subsections 3.3 and 4.3.

4.3 Design Computations

As indicated earlier in this section, three relationships must be known to perform reservoir routing computations. Since the duration of the critical storm for the specified design storm return period is not known, the critical inflow hydrograph is generally an unknown relationship. In addition, most design problems require that either the stage-storage or stage-discharge curves be determined. Therefore, most detention storage design problems have two unknown relationships and a trial and error solution process is required. In general, the design computations for a stormwater detention basin should follow the following steps (Mein 1980):

1. Compute design inflow hydrographs for a range of storm durations at the desired return period. For preliminary calculations, only inflow stormwater volumes are required.

2. Route each inflow hydrograph or preliminary volume through detention storage and determine basin dimensions using the storage-indication method, or an appropriate preliminary solution process.

   a. Outlet Works Design Problem. For each storm duration of Step 1, adjust the diameter and/or number of principal spillway pipes until the available storage volume just fills during the design storm and the minimum peak outflow is obtained.

   b. Storage Volume Design Problem. For each storm duration of Step 1, adjust the basin storage capacity for a given principal spillway configuration such that the desired peak outflow is obtained.

3. Design the detention basin for the storm duration with the highest peak outflow identified in Step 2.

   a. Outlet Works Design Problem. Size the principal spillway for the worst case in Step 2a.

   b. Storage Volume Design Problem. Size the basin storage capacity for the worst case in Step 2b.

4. Size the emergency spillway.

5. Determine downstream effects.

Since this design process requires a significant number of reservoir routings, a computer will greatly aid in conducting the final design computations.

SECTION 5.0 REFERENCES

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LIST OF EXAMPLE PROBLEMS--CHAPTER 8

LIST OF SYMBOLS--CHAPTER 8

V = volume of a trapezoidal hydrograph

H = peak flow or height of the trapezoidal hydrograph, in cfs

B_1 = bottom time base for the trapezoidal hydrograph, in seconds

B_2 = top time base for the trapezoidal hydrograph, in seconds

$V_{1,2}$ = storage volume, in ft3, between elevations 1 and 2

A_1 = surface area at elevation 1, in ft2

A_2 = surface area at elevation 2, in ft2

d = change in elevation between points 1 and 2, in feet

Q = discharge, in cfs

C = weir coefficient

L = length of weir, in feet

H = height or head of water above weir evalation, in feet

I_1 and I_2 = inflow rates at times 1 and 2, respectively, in cfs

O_1 and O_2 = outflow rates at times 1 and 2, respectively, in cfs

S_1 and S_2 = storage volumes at times 1 and 2, respectively, in ft3

Wt = time change between periods 1 and 2, in seconds

Q_0 = outflow peak flow, in cfs

Q_i = inflow peak flow, in cfs

V_s = volume of storage, in inches

V_r = volume of runoff, in inches

t_b = time base of the inflow hydrograph, in hours, determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak

t_p = time to peak of the inflow hydrograph, in hours

Table 8-1
APPLICATIONS OF POROUS PAVEMENT

Pavement Applications	Estimated Loads Maximum[a] Pounds per Square inch	foot	yard	Speed High-average mph	Frequency Passages per minute	Special Consideration
1. Sidewalk	30	300	900	6	10	Appearance
2. Public Square	30	300	1,500	4	20	Appearance
3. Playground	25	250	750	8	20	Resilience-injury
4. Parking Lot	60	2,000	2,000	10	5	Shopping carts, color
5. Residential Street (restricted traffic)	40	1,200	1,600	30	10	Plantings alongside, color
6. Service Road	90	7,000	15,000	15	5	Litter
7. Business Street	70	5,000	10,000	30	10	Traffic Stripes
8. Highway	90	7,000	15,000	65	26	Wet Skid
9. Taxi Apron	200	28,000	60,000	6	3	Fuel and oil spills
10. Airport Landing	200	28,000	60,000	300	3	Wet skid and hydroplaning

Source: Thelen et al. (1972)

[a]Not including impact

Table 8-2
EXAMPLE RESERVOIR ROUTING CALCULATIONS
FOR THE MODIFIED RATIONAL METHOD

Storm Duration (minutes)	Inflow Hydrograph[a]				Outflow Hydrograph[a]				Storage Volume Required (ft^3)
	H (cfs)	B_1 (sec)	B_2 (sec)	Volume (ft^3)	H (cfs)	B_1 (sec)	B_2 (sec)	Volume (ft^3)	
8	45.4	960	0	21,790	12.5	960	0	6,240	15,550
10	43.2	1,080	120	25,920	12.5	1,080	120	7,500	18,420
15	37.4	1,380	420	33,660	12.5	1,380	420	11,250	22,410
20	32.4	1,680	720	38,880	12.5	1,680	690	14,810	24,070
30	26.6	2,280	1,320	47,880	12.5	2,280	1,260	22,120	25,760
40	22.3	2,880	1,920	53,520	12.5	2,880	1,800	29,250	24,760

Note: Critical Storm Duration = 30 minutes
Critical Storage Volume = 25,760 ft^3

[a] Data are taken from Figure 5-15.

Table 8-3
EXAMPLE TABULATION OF STORAGE CHARACTERISTICS
CURVES FOR EXAMPLE PROBLEM 8-2

Stage (ft above MSL)	Storage[a] (acre-feet)	Discharge[b] (cfs)	Discharge[b] (acre-ft/hr)[d]	$S - \frac{O}{2}\Delta t$[c] (acre-ft)	$S + \frac{O}{2}\Delta t$[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	5.21	1.17	2.03
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 8-7.

[b] Obtained from Figure 8-8.

[c] Δt = 10 min = 0.167 hour.

[d] 1 cfs = 0.0826 acre-ft/hr.

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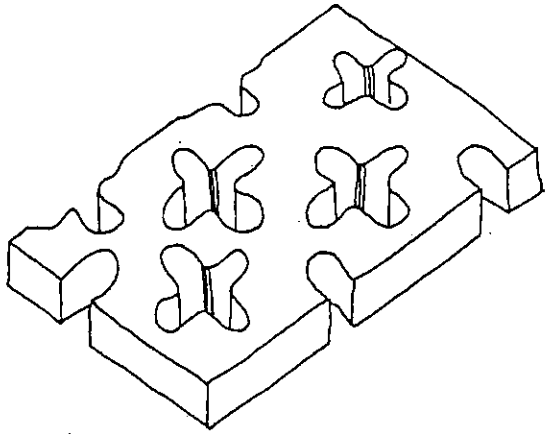
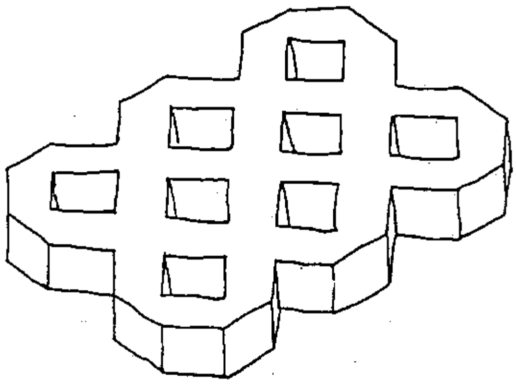
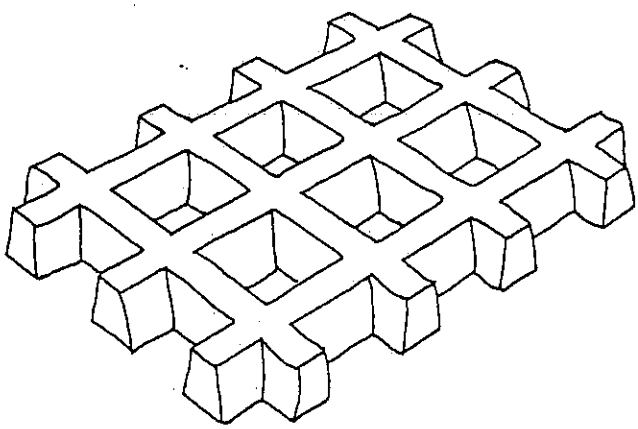


"Grasscrete" (Poured in Place)
by Bomanite Corp.



"Turfblock"
Paver Systems, Inc.
Wausau Tile



"Grasstone"
Boiardi Prods.

Source: Day (1980)

FIGURE 8-1. Lattice grid pavers.

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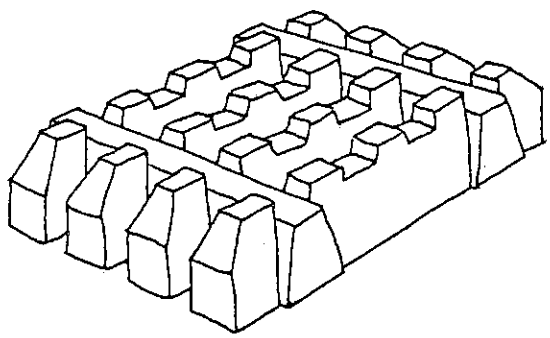
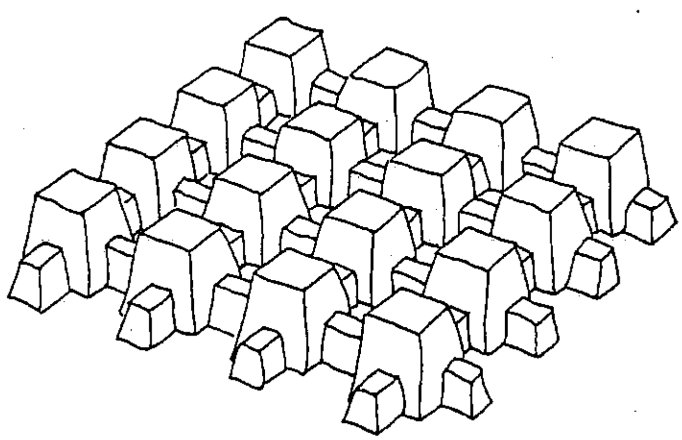


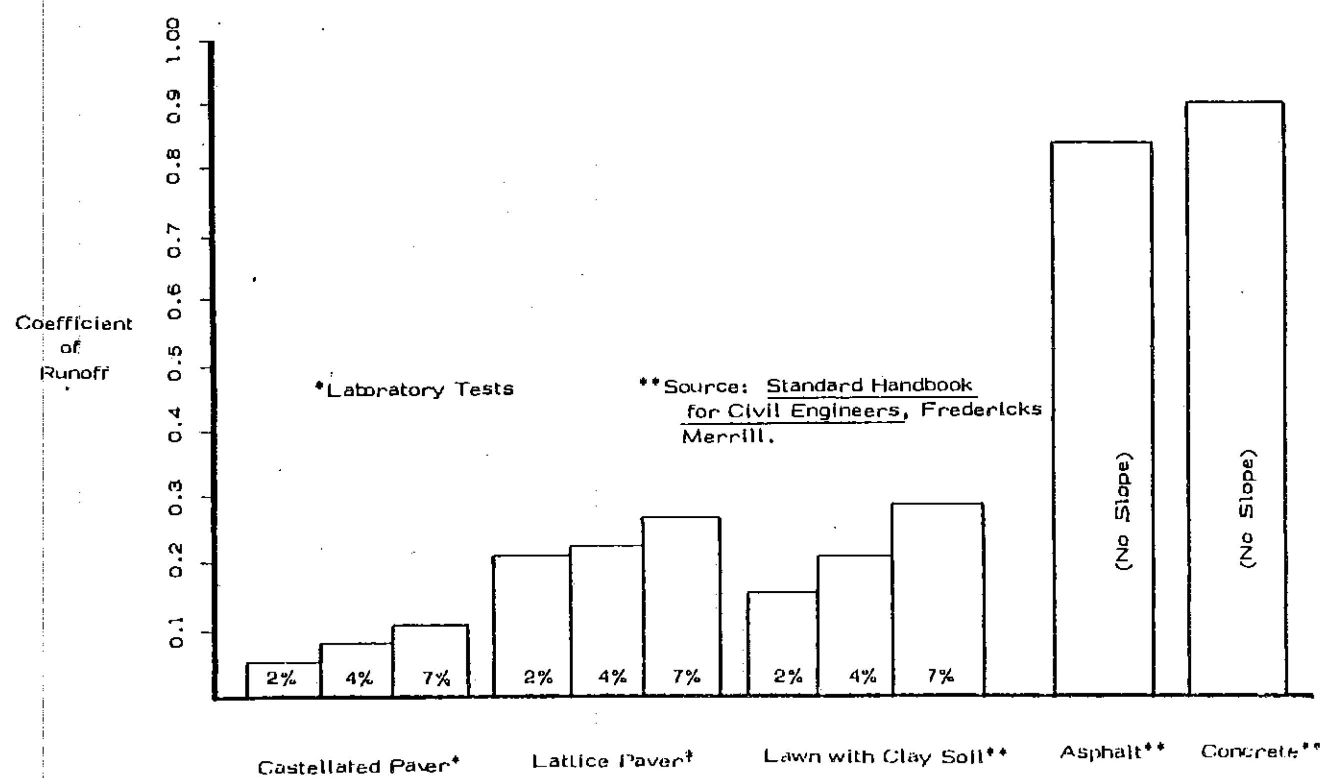
"Monoslab"
Grass Pavers, Ltd.



"Checker Block"
Hastings Co.

Source: Day (1980)

FIGURE 8-2. Castellated grid pavers.



Note: Percentages indicate surface slopes ranging from 2 to 7%.

Source: Day (1980)

FIGURE 8-3. Comparison of runoff coefficients of concrete grid pavements to other urban surfaces.

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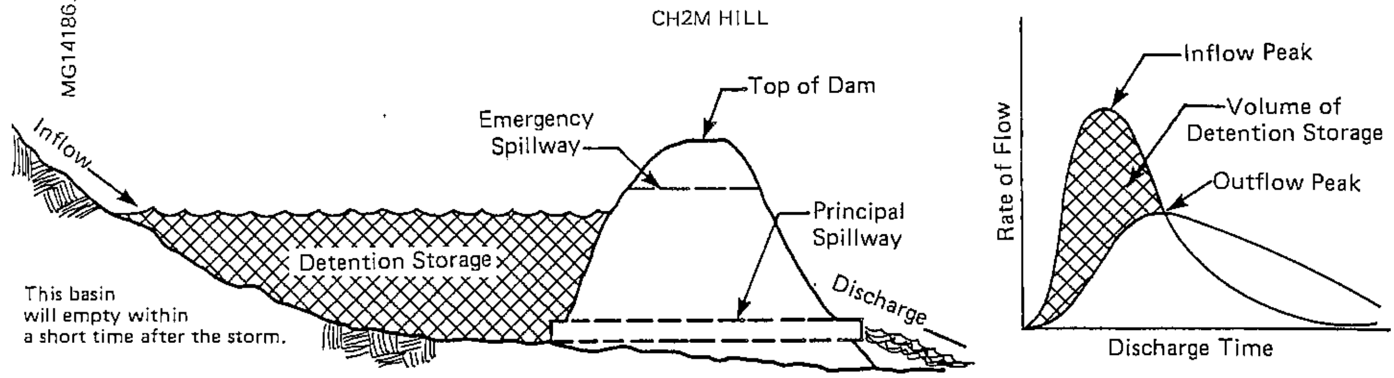
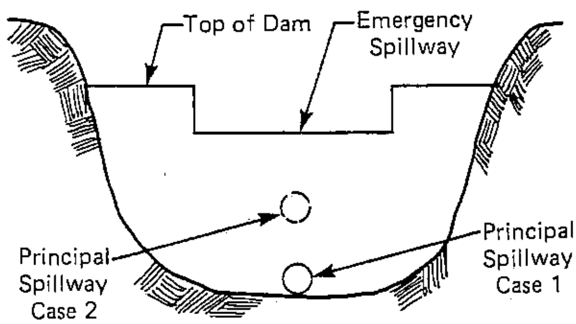
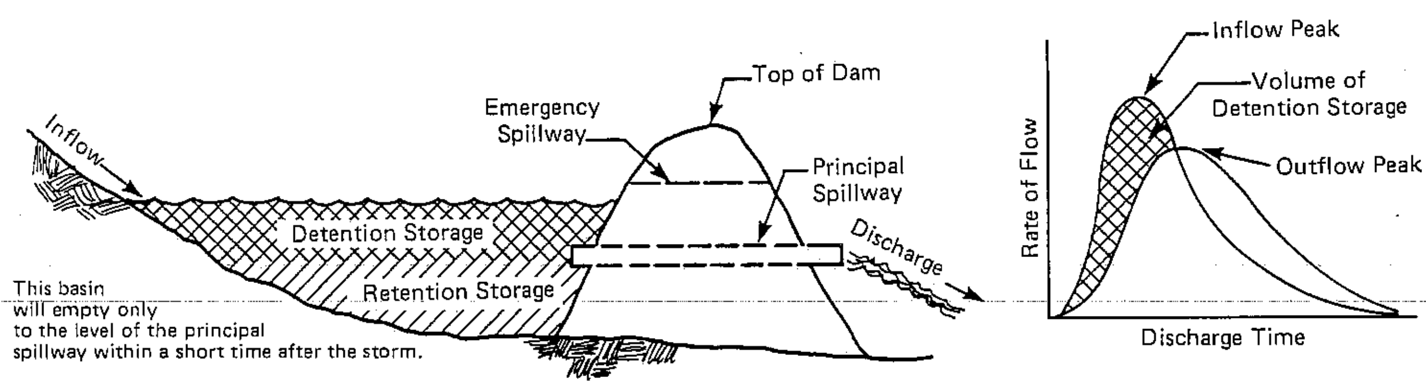


A) Case 1—Side View of a Detention or Temporary Storage Basin



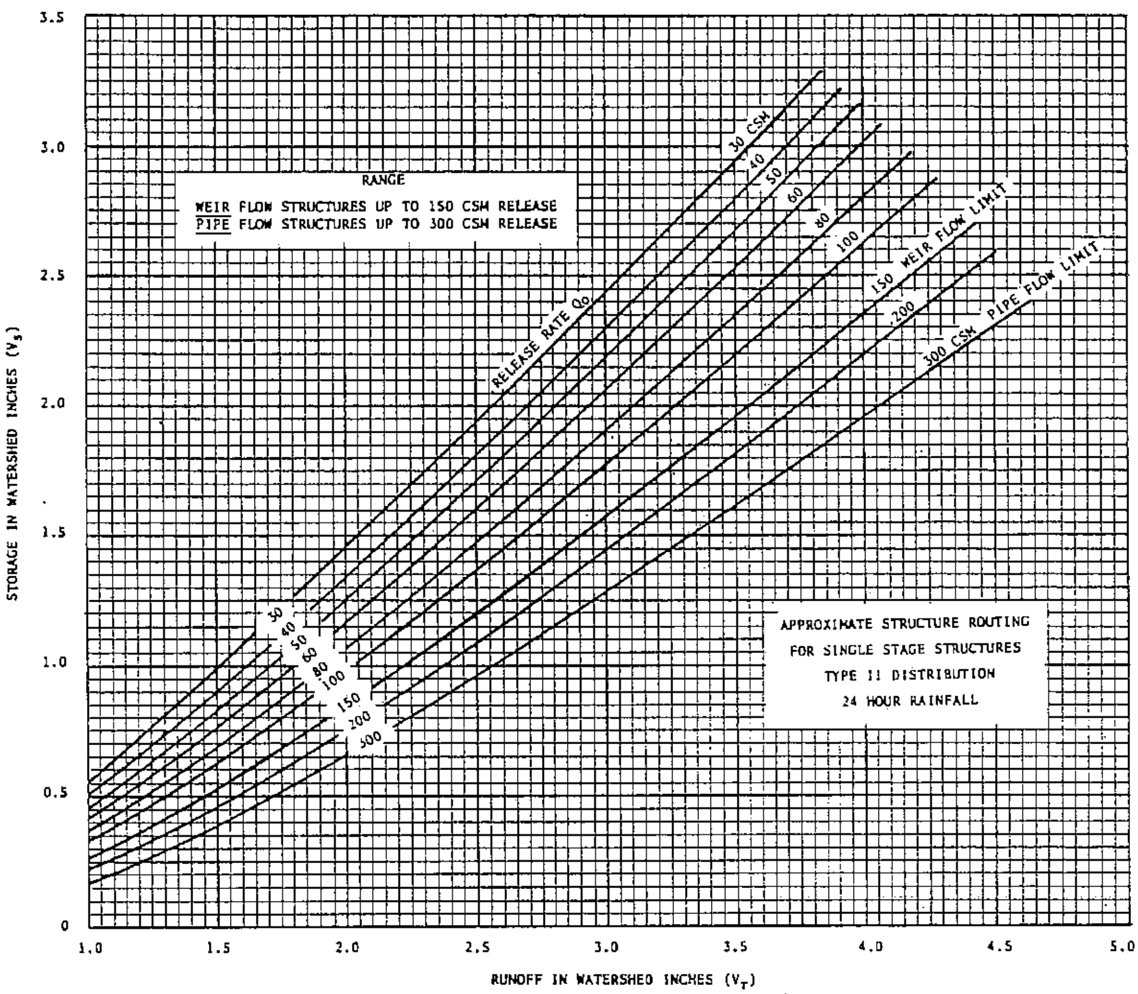
B) Front View

The purpose of a detention storage basin is to reduce the peak flows in a drainage system. By appropriate sizing of the principal spillway, floodwater is discharged from the basin at a safe rate, and the excess is held temporarily as flood storage. If part of the basin will be used as retention storagé, the ability to limit peak flows is reduced. The hydrographs to the right of each basin sketch illustrate the relative differences between outflow peaks for detention and retention storage systems.



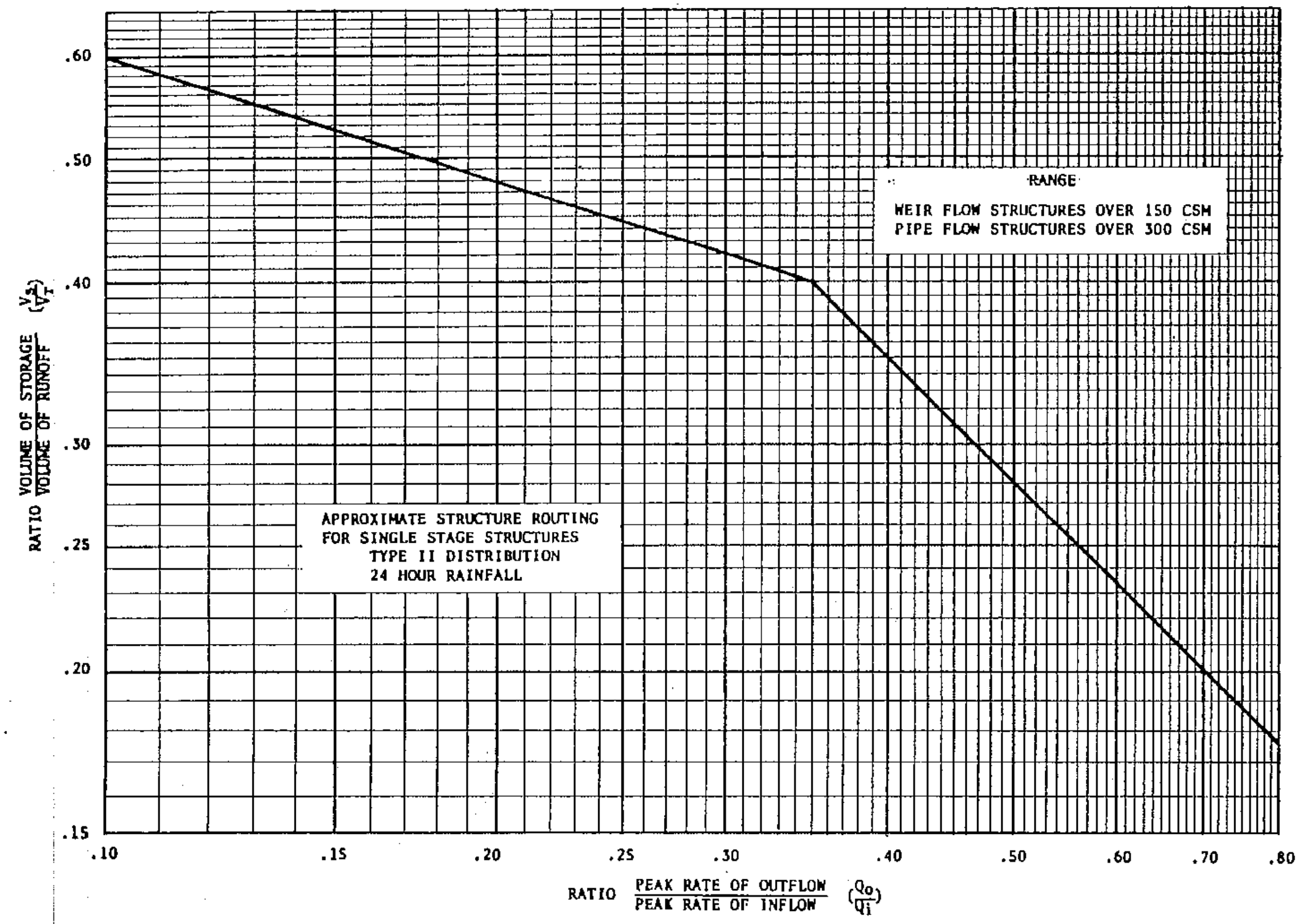
C) Case 2—Side View of a Retention or Permanent Storage Basin

FIGURE 8-4. Stormwater storage basins.



Source: USDA, SCS, TR-55 (1975)

FIGURE 8-5. SCS preliminary reservoir routing method for weir flow structures up to 150-csm release rate and pipe flow structures up to 300-csm release rate.



Source: USDA, SCS, TR-55 (1975)

FIGURE 8-6. SCS preliminary reservoir routing method for weir flow structures over 150-csm release rate and pipe flow structures over 300-csm release rate.

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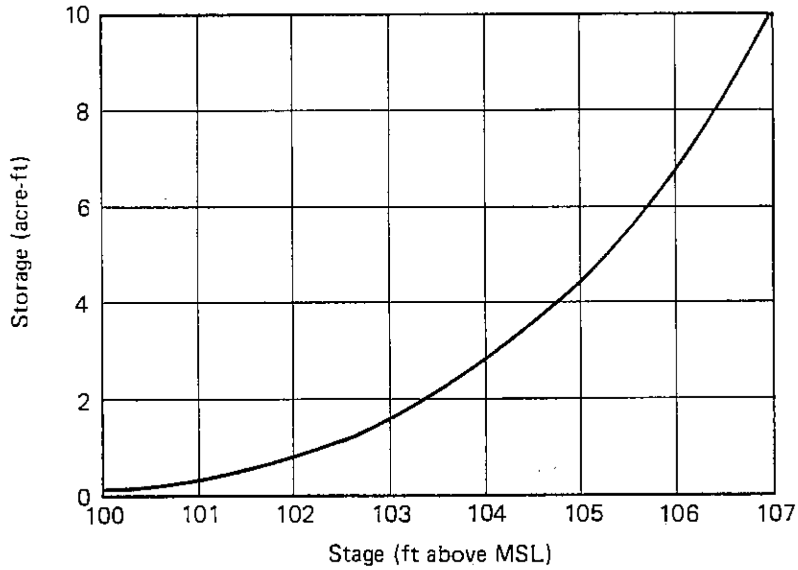

FIGURE 8-7. Example stage-storage curve used in example 8-2.

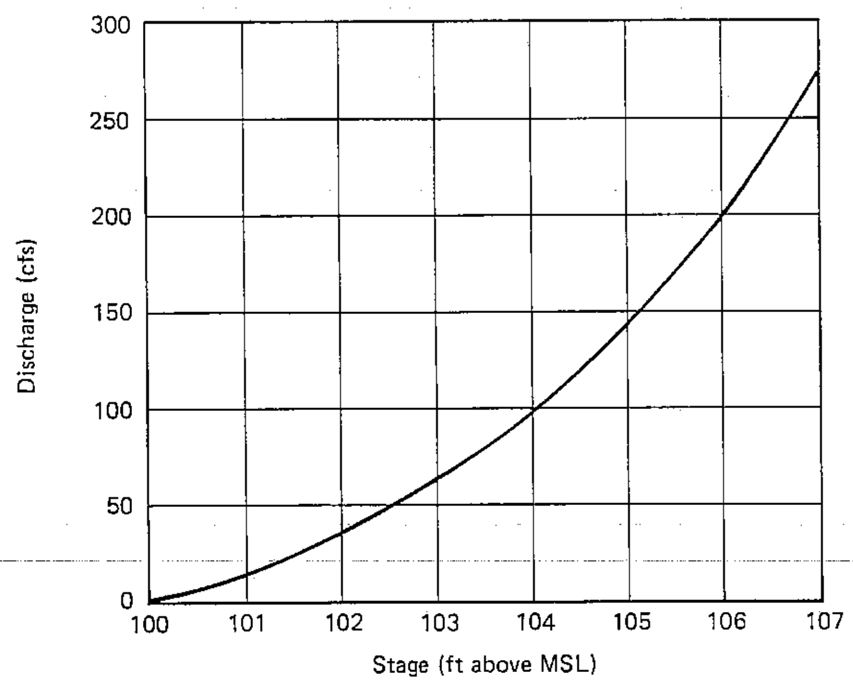

FIGURE 8-8. Example stage-discharge curve used in example 8-2.

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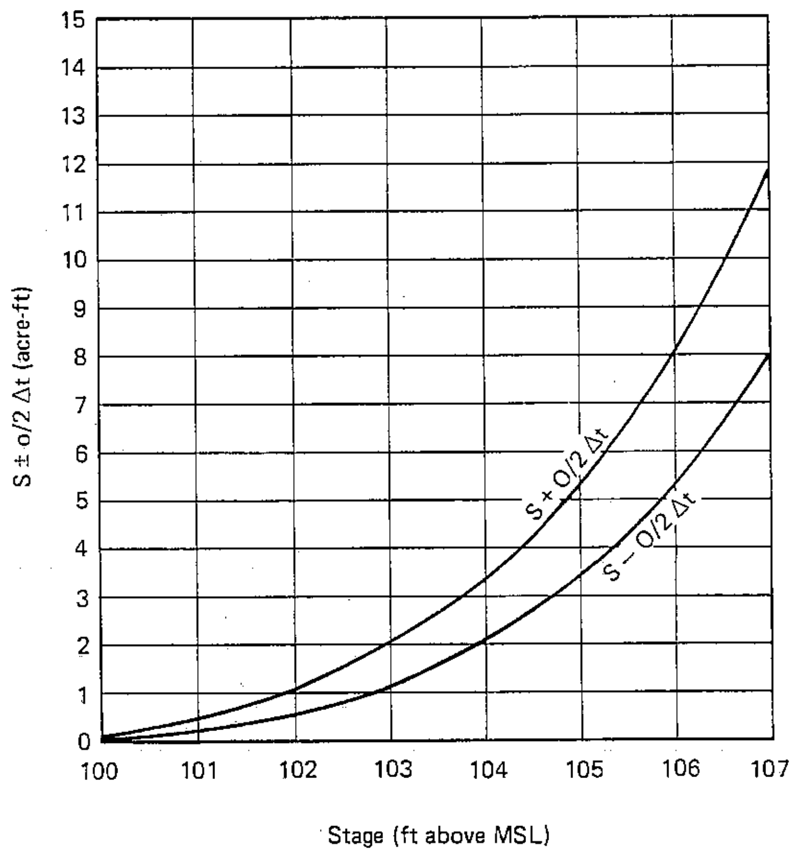

Note: Data obtained from Table 8-3.

FIGURE 8-9. Storage characteristics curve used in example 8-2.

SECTION 1.0 INTRODUCTION

In practice, the design of erosion and sediment control systems requires the application of common sense planning, scheduling, and control actions to minimize the adverse impacts of soil erosion, transport, and deposition. This chapter provides technical guidance to aid the designer in developing common sense erosion and sediment control measures for the typical urban stormwater system. In all cases, the average annual soil loss from a construction site in Montgomery shall be less than 15 tons/acre/year, with a maximum duration of exposed land of 3 months. In addition, the Standard Specifications of the Alabama Highway Department for temporary erosion control measures shall apply to the City of Montgomery.

SECTION 2.0 PRINCIPLES AND PRACTICES

Five basic, common sense principles govern the development and implementation of a sound erosion and sediment control plan:

1. Plan the development to fit the topography, soils, waterways, and natural vegetation at the site.

2. Expose the smallest practical area for the shortest possible time. For Montgomery, the maximum area which can be exposed at any given time is 17.5 acres.

3. Apply onsite erosion control measures to reduce the gross erosion from the site.

4. Utilize sediment control measures as a perimeter protection to prevent offsite damage.

5. Implement a thorough maintenance and follow-up program.

Planning of the erosion and sediment control strategies is a central theme which ties these five principles together in practice. This planning process requires that potential erosion and sediment control problems be identified prior to construction of the stormwater management system. A valuable tool for quantifying the critical portions of sites where potential problems exist is the Universal Soil Loss Equation (USLE), which was presented in Chapter 6. Planning and design information related to the effectiveness of selected erosion and sediment control alternatives is the subject of Chapter 9. Combining this effectiveness data with the procedures presented in Chapter 6, a basis for identifying appropriate erosion and sediment control strategies prior to construction can be established such that the average annual soil loss is less than 15 tons/acre/year.

The control practices considered in this chapter are classified as erosion control measures or sediment control measures (see principles 3 and 4 above). In general, erosion control practices are designed to prevent soil particles from being detached, whereas sediment control involves using practices that prevent the detached particles from leaving the site or from entering a receiving water. Sediment control measures are generally structural in nature, while erosion control measures can be either structural (such as diversions) or non-structural (such as mulches). A schematic diagram of the erosion process, with appropriate points of application for onsite erosion controls or sediment removal controls, is presented on Figure 9-1.

It is important to distinguish between temporary and permanent control measures. As the terms imply, temporary practices are those used for relatively short periods of time, while permanent practices are utilized for an indefinite period of time. In order to economize, the standards and specifications for temporary control measures can be reduced compared to those required for permanent control measures, which require individual designs in order to fit the practices to site-specific conditions.

A sediment and erosion control plan should identify the erosion control practices and sediment trapping facilities which are appropriate for the site conditions in question. In addition, the appropriate schedule of implementation should be identified. Particular attention is required for concentrated stormwater flows. Either concentrated stormwater flows should be avoided or the conveyance system should be protected sufficiently to prevent significant erosion. Sediment trapping devices are generally required at all points where stormwater leaves a site laden with sediment. The plan should identify permanent stormwater conveyance structures, final stabilized conditions of the site, provision for removing temporary control measures, stabilization of the site where temporary measures are removed, and maintenance requirements for any permanent measures.

SECTION 3.0 EROSION CONTROL

As noted in Section 2.0, erosion control practices are designed to prevent soil particles from becoming detached. Three general classifications of erosion control measures are presented in this section: (1) surface stabilization, (2) exposure scheduling, and (3) runoff control. Appropriate control measures for each classification are defined in sufficient detail to identify applicable conditions for employing each of these measures. Quantitative performance information for use with the USLE is presented, and a general discussion of design considerations is included when appropriate. All temporary erosion control systems shall conform to the requirements of the Alabama Highway Department Standard Specifications. According to Section 665 of these specifications, a contractor shall not expose more than 17.5 acres of erodible material for any separate major operation without prior approval of the Engineer. Requirements of the following temporary onsite erosion control measures shall apply:

Control Measure	Section
Riprap	610
Ground Preparation and Fertilizers for Erosion Control	651
Seeding	652
Sprigging	653
Solid Sodding	654
Mulching	656
Grassy Mulch	657
Hydro-Seeding and Mulching	658
Erosion Control Netting	659

3.1 Surface Stabilization

The soil cover factor, C, originally proposed for use in the USLE (equation 6-1) was defined as the ratio of soil loss from an area with specified cover and management conditions to the soil loss from an identical area in tilled continuous fallow. For the purposes of this manual, the original definition of C is applied to the surface stabilization factor, C_s, which is used in equation 6-3 to determine a single control practice factor, CP, which is utilized in the USLE. Since the basis for determining a soil stabilization factor is tilled soil in continuous fallow, freshly disked soil has a C_s value of 1.0. If the natural or cleared site surface conditions vary significantly from tilled continuous fallow, a C_s value other than 1.0 must be determined to establish baseline or existing soil loss conditions. Having established the baseline or existing conditions, the relative impact of various surface stabilization alternatives can be evaluated using the USLE with appropriate values of C_s.

Soil stabilization factors for natural or unprotected site conditions can be estimated from published data. For various types of bare soil conditions, C_s factors can be estimated from values reported in Table 9-1. For permanent pasture, rangeland, idle land, and grazed woodland, C_s factors can be estimated from values reported in Table 9-2. For undisturbed woodland, C_s factors can be estimated from values reported in Table 9-3.

Soil stabilization factors for selected erosion control alternatives are presented in the discussion which follows. Surface stabilization control measures which are considered include mulches, seeding and vegetation, chemical binders and tacks, and other materials. In addition, the stabilization of stormwater outlet structures is considered in this section.

<u>3.1.1 Mulches</u>. A mulch is a layer of plant residue or inorganic material applied to the soil surface for temporary soil stabilization and to help establish plant cover. Mulches are practical on graded or cleared areas for 6 months or less where seedings may not have a suitable growing season to produce an erosion-resistant cover. Final grading is not required prior to mulching; however, mulching may be applied after final grade is reached. Whenever structural erosion control features are used, they should be installed prior to mulching.

The important types of mulching material include straw or hay, crushed stone, and wood chips. Mulch surface stabilization factors, C_s, and length limits for selected application rates on construction sites are presented in Table 9-4. Design details for mulches are presented in Section 656 of the Alabama Highway Department Standard Specifications.

<u>3.1.2 Seeding and Vegetation</u>. Surface stabilizations by vegetation include temporary seeding, permanent seeding, sod, vines, shrubs, and trees. Design information for grass-lined open channels was presented in Chapter 7. Surface stabilization factors, C_s, for temporary and permanent seedings are presented in Table 9-5. Surface stabilization factors for mechanically prepared woodlands are presented in Table 9-6. Design details related to types of seeding and vegetation, application rates, site preparation, and fertilizer requirements can be found in Sections 651 and 652 of the Alabama Highway Department Standard Specifications.

<u>3.1.3 Chemical Binders and Tacks</u>. Synthetic binders and tacks are usually 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. If construction occurs at a time when seeding is not feasible, chemical binders and tacks provide a viable alternative for temporary protection. Surface stabilization, C_s, factors for selected chemical binders and tacks are presented in Table 9-5.

3.1.4 Other Materials. Other stabilization materials include nettings, plastic filter sheets, and riprap. Nettings of fiberglass, plastic, and paper yarn can be used to anchor straw, hay, wood chips, or grass and sod in drainageways and in other areas subject to concentrated runoff. Design details for erosion control nettings are presented in Section 659 of the Alabama Highway Department Standard Specifications. Plastic filter sheets consist of porous fabric woven from polypropylene monofilament yarns. They are lightweight, porous, strong, abrasion-resistant, and unaffected by salt-water. They are used as a replacement for graded sand filters beneath riprap and concrete structures placed in waterways. Their purpose is to prevent foundation soil particles from being drawn up through the structure by the hydraulic forces associated with concentrated and often turbulent runoff. Information related to stabilization with riprap appears in Chapter 7 and Section 610 of the Alabama Highway Department Standard Specifications.

3.1.5 Outlet Protection. Outlet protection consists of providing de-energizing devices and erosion-resistant channel sections between drainage outlets and stable existing down-stream channels. The channel sections may be rock-lined, vegetated, paved with concrete, or otherwise made 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, in order to convey the flow of water to a stable existing downstream channel, without causing erosion.

This practice applies to storm drain outlets, road culverts, paved channel outlets, etc., discharging into natural or constructed channels, which in turn discharge into existing streams or drainage systems. Analysis and 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.

Design procedures for outlet protection should be consistent with erosion resistance information presented in Chapter 7 for open channels. The design should include a plan view, profile, and cross section for each channel reach between the storm drain outlet and the existing publicly maintained system or natural stream channel. The velocity for the following should be indicated: (1) outlet (pipe, structure, or paved channel), (2) riprap or paved apron section, and (3) each successive channel reach from the end of the apron to the point of entry into the existing drainage system or natural stream channel. The plan should indicate the proposed method of stabilizing each channel reach, consistent with computed velocities. The velocity at the end of a structure or channel reach must not exceed the allowable velocity of the next downstream reach.

For the purposes of this manual, a channel reach is defined as a length of channel throughout which the hydraulic characteristics do not change. These include channel depth of flow, roughness, channel gradient, side slopes, bottom width, discharge rate, and velocity. A natural stream channel is defined as a naturally formed channel through which the storm runoff would have flowed (had there been no intervention by man) and which is capable of conveying the peak rate of runoff after development without eroding.

3.2 Exposure Scheduling

The sequence and duration of exposing cleared land to the erosion forces of rainfall can have a significant effect on the gross soil loss from a site. The impact of exposure scheduling on the gross soil loss from a site can be evaluated using the USLE (Chapter 6). This is accomplished using the monthly distribution of the rainfall erosion index for Montgomery, which is presented on Figure 6-2.

Exposure scheduling can be evaluated according to the following procedure. First, appropriate surface stabilization, C_s, factors are determined for the sequence of land covers to be evaluated. For example, a site may begin the year as an undisturbed woodland, followed by clearing for construction, temporary seeding, and then permanent seeding. Using the length of time and season during which each of these land cover types will exist, a weighted surface stabilization, C_s, factor can be calculated using the monthly distribution of the rainfall presented on Figure 6-1. This procedure is demonstrated in Table 9-7.

3.3 Runoff Control

3.3.1 Diversion Structures. Quantitative information related to the runoff control, C_r, factor presented in equation 6-3 is currently available only for diversion structures. However, this should not limit the usefulness of the USLE as a planning tool for runoff control, since diversion structures are the principal means of reducing slope lengths and thus gross erosion. Any structure which slows runoff or diverts it away from downslope areas can reduce erosion. Chen (1974) proposed that the impact of diversions on gross erosion be quantified using the following expression:

$$C_r = \frac{1}{(N + 1)^{1/2}} \qquad (9\text{-}1)$$

where

C_r = runoff control factor

N = number of diversions placed across a uniform slope

Diversions can be temporary or permanent structures. Types of diversion structures include dikes, swales, and channels.

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 disposal area. The diversion dike is the least durable diversion structure and, therefore, should only be used to provide protection for short periods of time and when relatively small amounts of runoff are to be handled. It is often used above a newly constructed fill and cut slope to prevent excessive erosion of the slope until more permanent drainage features are installed, or the slope is stabilized with vegetation. Where the ground slope is not steep, it is also used before graded slopes to divert sediment-laden runoff into sediment traps or basins. Once the slope is stabilized, the diversion dike is 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 trapping device, such as a sediment trap or sediment basin. The swale is left in place until the disturbed area is permanently stabilized. A diversion swale is also used to prevent stormwater from entering the disturbed area.

A diversion channel is a permanent or temporary drainageway constructed by excavating a shallow ditch along a hillside and building a soil dike along the downhill edge of the ditch with the excavated soil. In other words, it is a combination of a ditch and dike. Although diversion channels can be used in place of temporary structures, such as diversion dikes and swales, they are primarily used to provide more permanent runoff control on long slopes subject to heavy flow concentrations. In addition to being used to intercept and divert runoff on, or above, long graded spoil slopes, diversion channels can be used to intercept runoff flowing along a roadway and divert it across the roadway to a safe outslope disposal point.

3.3.2 Downdrain Structures. Downdrain structures are stabilized channels or pipes used to conduct concentrated runoff safely down a slope. They can be temporary or permanent structures. Such structures are often used to help dispose of water collected by diversion structures. Commonly used downdrain structures include the paved chute or flume and the pipe slope drain.

A paved chute or flume is a channel lined with bituminous concrete, portland cement, concrete, or comparable non-erodible material (such as grouted riprap), placed to extend from the top to the bottom of a slope. A paved chute or flume is used where a concentrated flow of surface runoff must be conveyed down a slope without causing erosion. For temporary structures, the maximum allowable drainage area is 36 acres.

A pipe slope drain consists of either a rigid pipe or flexible tubing, together with a prefabricated entrance section, and it is temporarily placed to extend from the top to the bottom of a slope. Like paved chutes and flumes, pipe slope drains are used where a concentrated flow of surface runoff must be conveyed down a slope without causing erosion. The maximum allowable drainage area is 5 acres.

<u>3.3.3 Level Spreaders</u>. A level spreader is an 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 non-erosive velocities onto undisturbed areas stabilized by existing vegetation. The level spreader is used only in those situations where the spreader can be constructed on undisturbed soil, where the area directly below the level lip is stabilized by existing vegetation, where the drainage area above the spreader is stabilized by existing vegetation, and where the water will not be reconcentrated immediately below the point of discharge.

<u>3.3.4 Check Dams</u>. A check dam is a structure used to stabilize the grade or to control head cutting in natural or artificial channels. Check dams are used to reduce or prevent excessive erosion by reducing velocities in watercourses or by providing partially lined channel sections or structures that can withstand high flow velocities. Check dams are used where the capability of earth and/or vegetative measures is exceeded in the safe handling of water at permissible velocities, where excessive grade or overall conditions occur, or where water is to be lowered from one elevation to another.

<u>3.4 Example Problems</u>

<u>Example 9-1</u>. Surface Stabilization Control Practice Factor

An idle tract of land with a 50 percent canopy of tall grass-like weeds and 40 percent ground cover is to be undisturbed except for scraping between April 1 and August 1. The soil series is Catalpa with a slope length of 200 feet and a slope of 1 percent. Determine a CP factor with surface stabilization control practices which will keep the average annual soil loss below 15 tons/acre/year. Assume that temporary and permanent seeding are applied after August 1.

1. From Table 9-2, the idle land has a C_s value of 0.07. From Table 9-1, undisturbed land with scraping has a maximum C_s value of 1.30. From Table 9-5, temporary seeding has a C_s value of 0.40 and permanent seeding has a C_s value of 0.05.

2. Using the exposure scheduling procedure presented in Table 9-7, the following table is developed:

Time Period	Surface Cover	C_s Factor	Fraction of Annual R During Time Period	Weighted C_s Factor
1/01 to 4/01	Idle Weeds	0.07	0.20	0.014
4/01 to 6/01	Scraped Site	1.30	0.18	0.234
6/01 to 8/01	Temporary Seed	0.40	0.31	0.124
8/01 to 12/31	Permanent Seed	0.05	0.31	0.016
				Σ = 0.388

3. From Table 4-4, k = 0.28. From Figure 6-3, LS = 0.16. Therefore, according to equation 6-1:

$$A = (350)(0.28)(0.16)(0.388) = 6.8 \text{ tons/acre/year}$$

4. Since A is less than 15 tons/acre/year, a CP factor of 0.38 is adequate.

Example 9-2. Runoff Control Practice Factor

For the data presented in example 3-1, find the number of diversions required to provide a level of erosion control comparable to that provided with surface stabilization practices.

1. Find the CP value required for runoff control.

$$0.388 - 0.014 = 0.374$$

$$(C_r)(0.80) = 0.374$$

$$C_r = \frac{0.374}{0.080} = 0.47$$

2. Using equation 9-1:

$$(N + 1)^{\frac{1}{2}} = \frac{1}{C_r} = \frac{1}{0.47}$$

$$N + 1 = \left(\frac{1}{0.47}\right)^2$$

$$N = \left(\frac{1}{0.47}\right)^2 - 1 = 3.5 \text{ diversion structures}$$

SECTION 4.0 SEDIMENT CONTROL

As noted in Section 2.0, sediment control practices are designed to prevent detached soil particles from leaving a particular site. Three general types of sediment control measures are defined and briefly discussed in this section: straw bale dikes, sediment traps, and sediment basins. Sediment control at construction entrances is considered as well.

It should be noted at the outset that straw bale dikes and sediment traps are temporary, stop-gap measures. They should be applied only when other alternatives prove unfeasible.

4.1 Straw Bale Dikes

A straw bale dike is a temporary barrier constructed of straw bales installed across or at the toe of a slope. The life expectancy of a straw bale dike is 3 months or less. The purpose of a straw bale dike is to intercept and detain small amounts of sediment from unprotected areas of limited extent.

The straw bale dike is used only where:

1. No other practice is feasible.

2. There is no concentration of water in a channel or other drainageway above the barrier.

3. Erosion would occur in the form of sheet and rill erosion.

4. Contributing drainage area is less than one-half acre and the length of slope above the dike is less than 100 feet.

Straw bales should be placed in a row with ends tightly abutting adjacent bales. Each bale should be embedded a minimum of 4 inches into the soil. In addition, bales should be securely anchored in place by stakes or re-bars driven through the bales.

4.2 Sediment Traps

A sediment trap is a small temporary basin formed by an excavation and/or an embankment to intercept sediment-laden runoff and to trap and retain the sediment. In so doing, drainageways, properties, and rights-of-way below the trap are protected from sedimentation. Sediment traps should be installed within the drainage system prior to leaving a construction site.

An earth outlet sediment trap consists of a basin formed by excavation and/or an embankment. The trap has a discharge point over or cut into natural ground. A pipe outlet sediment trap consists of a basin formed by an embankment or a combination of an embankment and excavation. The outlet for the trap is through a perforated riser and a pipe through the embankment.

4.3 Sediment Basins.

A sediment basin is constructed on a waterway to impound runoff coming from a cleared construction site. The pond is formed by placing an earthen dam across the waterway, by excavating a depression, or by a combination of the two. The purpose of a sediment basin is to remove sediment from runoff and thus protect drainageways, properties, and rights-of-way below the sediment basin from sedimentation. Sediment basins are installed below construction sites, on or adjacent to the major waterways. They act as a last line of defense against offsite sediment damage and are used to reduce the suspended solids concentration to acceptable levels.

A sediment basin should be constructed only under the following conditions:

1. Failure of the structure would not result in loss of life, damage to buildings, or interruption of service from public roads or utilities.

2. The basin is to be removed within a specified period of time.

Components of a typical sediment basin are illustrated on Figure 9-2. The storage volume of this sediment basin can be divided into three components: (1) a sediment storage volume for retaining trapped sediment, (2) a detention storage volume to hold the design storm for a duration sufficient to remove the desired amount of sediment, and (3) a flood storage volume to prevent overtopping and dam failure during rare storm events. Each of these storage components requires special design considerations.

4.3.1 Detention Storage. The detention storage volume is sized to hold the design storm for a duration sufficient to remove the desired amount of sediment. Having established a reasonable trap efficiency for the sediment basin, the surface area required to obtain that efficiency can be estimated by an equation presented by Camp (1945) as follows:

$$A = 1.2 \frac{EQ}{v_s} \tag{9-2}$$

where

A = surface area of a sediment detention basin, in ft^2

E = desired trap efficiency, expressed as a fraction

Q = design outflow rate from the basin, in cfs

v_s = average particle settling velocity, in fps

The average particle settling velocity, v_s, can be evaluated using Stokes law, which is expressed mathematically as follows:

$$v_s = \frac{1}{18} \frac{d^2 g}{\nu} (SG - 1) \tag{9-3}$$

where

d = average particle diameter, in feet

g = acceleration of gravity, 32.2 ft/sec^2

ν = kinematic viscosity, in ft^2/sec

SG = average specific gravity

Stokes law is valid only for spherical particle sizes and Reynolds numbers less than 0.5. Therefore, the factor of 1.2 in equation 9-2 is a safety factor to account for non-ideal settling. Values of kinematic viscosity for water at various temperatures are presented in Table 9-8.

The design flow rate from the pond is determined by routing the appropriate design inflow hydrograph through the principal spillway. Information related to reservoir routing procedures is presented in Chapter 8. The ratio of the design outflow rate, Q, to the surface area of the basin, Q/A, is known as the overflow rate or surface loading rate of the basin.

Ideal discrete particles which have settling velocities greater than the overflow rate will theoretically be completely removed by the sediment basin. Particles with settling velocities less the overflow rate will be removed in proportion to the ratio of their settling velocity divided by the overflow rate.

4.3.2 Sediment Storage. The sediment storage is sized to hold the sediment trapped by the basin for the desired design life of the basin. The sediment storage volume can be estimated by the following equation:

$$V_s = \frac{(1.2)\ E\ Y}{D_s(43,560)} \qquad (9\text{-}4)$$

where

V_s = sediment storage volume, in acre-ft

Y = sediment yield, in pounds (from Chapter 6)

D_s = bulk density of sediment, in pounds/ft^3 (see Table 6-1)

4.3.3 Flood Storage. The flood storage volume is determined by sizing an emergency spillway to prevent overtopping, which would cause dam failure during a rare storm event. The emergency spillway should be sized according to the procedures identified in Chapter 8, to pass the peak rate of runoff from the design storm less any reduction due to the principal spillway. Velocities in the exit channel of the emergency spillway should be in a non-erosive range for the type of lining used (see Chapter 7). A freeboard of 20 percent should be added to the embankment above the emergency spillway height.

4.3.4 Sediment Removal. Sediment basins require scheduled inspection and maintenance to function properly. They should be located in a manner which allows easy access and inspection. In addition to regular periodic maintenance, sediment basins should be inspected after all major storm events to ensure that spillway structures are not clogged and that the sediment storage volume has not been exceeded.

One rule of thumb is that a sediment basin should be cleaned out when it has reached 50 percent of its sediment storage capacity. A red pole can be placed in the sediment storage area to aid inspection. When the red pole disappears, the sediment should be removed. The sediment which is removed must be disposed of in a productive manner. The method of disposal should be clearly identified on the basin plans prior to construction.

4.3.5 Plans. Information related to detailed plans and specifications for sediment basins are published by the USDA (1975). Construction plans submitted to the City Engineering Department of Montgomery should include the following:

1. Specific location of the dam.
2. Plan view of dam, storage basin, and emergency spillway.
3. Cross section of dam, cross section of principal spillway, emergency spillway profile.
4. Details of pipe connections, riser to pipe connection, riser base, anti-seep collars, trash rack, and anti-vortex device.
5. Runoff calculations for design storm.
6. Storage computation:
    a. Detention storage.
    b. Sediment storage.
    c. Flood storage.
7. Level of sediment at which clean-out will be performed or the basin will be removed.

4.4 Construction Entrances

All points of access to a construction site should be stabilized to reduce or eliminate the tracking or flowing of sediment onto public rights-of-way. A stabilized pad of crushed stones should be installed on all entrances to construction sites for this purpose. Maintenance of such entrances may require periodic top dressing with additional stone as conditions demand or clean-out of any measures used to trap sediment.

In some cases, wheels of construction vehicles should be cleaned prior to leaving the construction site. When washing is required, it should be performed on a stabilized area (one with crushed stone) which drains into a sediment trap or basin.

4.5 Example Problems

Example 9.3. Sediment Basin Calculations

The peak design outflow rate for a sediment basin is estimated to be 100 cfs for a 25-year storm. Find the surface area required to trap 50 percent of the median particle size of

0.20 mm. Assume a specific gravity of 2.65, spherical particles, and a water temperature of 60°F. Also estimate the sediment storage volume required for average annual conditions.

1. Find the average particle settling velocity using equation 9-2.

$$d = \frac{0.20}{(25.4)(12)} = 0.00066 \text{ ft}$$

$$\nu = 1.217 \times 10^{-5} \text{ ft}^2/\text{sec}$$

$$V_s = \frac{1}{(18)} \frac{(0.00066)^2(32.2)}{(1.217 \times 10^{-5})} [2.65 - 1] = 0.105 \text{ ft/sec}$$

2. Find the basin surface area:

$$A = 1.2 \frac{(0.50)(100)}{(0.106)}$$

$$A = 566 \text{ ft}^2$$

3. Use the average annual sediment yield for the site considered in example 9-1 between April 1 and July 1. From Figure 6-2, 30 percent of the average annual rainfall index occurs between April 1 and July 1; therefore:

$$CP = (1.30)(0.30) = 0.39$$

$$A = (350)(0.28)(0.16)(0.39) = 6.12 \text{ tons/acre/3 months}$$

4. Find the sediment storage volume required using equation 9-3, assuming the exposed area is 17.5 acres:

$$Y = (6.12)(17.5)(2{,}000) = 214{,}200 \text{ lb}$$

$$V_s = \frac{(1.2)(0.50)(214{,}200)}{(100)(43{,}560)} = 0.0295 \text{ acre-foot}$$

SECTION 5.0 REFERENCES

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5. Wischmeier, W. H., and Smith, D. D. 1978. Predicting Rainfall Erosion Losses--A Guide to Conservation Planning. U.S. Department of Agriculture, Agriculture Handbook No. 537, U.S. Government Printing Office, Washington, D.C.

LIST OF EXAMPLE PROBLEMS--CHAPTER 9

LIST OF EQUATIONS--CHAPTER 9

		Page
(9-1)	$C_r = \frac{1}{(N + 1)^{1/2}}$	9 - 6
(9-2)	$A = 1.2 \frac{EQ}{v_s}$	9 - 12
(9-3)	$v_s = \frac{1}{18} \frac{d^2 g}{\nu} (SG - 1)$	9 - 12
(9-4)	$V_s = \frac{(1.2)\ E.\ Y}{D_s (43,560)}$	9 - 13

LIST OF SYMBOLS--CHAPTER 9

A = surface area of a sediment detention basin

C_r = runoff control factor

D_s = bulk density of sediment, in lb/ft^3

d = average particle diameter, in feet

E = desired trap efficiency, in percent

g = acceleration of gravity, 32.2 ft/sec^2

N = number of diversions placed across a uniform slope

Q = design outflow rate from the basin, in cfs

SG = average specific gravity

V_s = sediment storage volume, in acre-feet

v_s = average particle settling velocity, in fps

Y = sediment yield, in pounds

ν = kinematic viscosity, in ft^2/sec

Table 9-1
SURFACE STABILIZATION, C_s, FACTORS
FOR BARE SOIL CONDITIONS

Bare Soil Conditions	C_s Factor
Freshly disked to 6-8 inches	1.00
After one rain	0.89
Loose to 12 inches smooth	0.90
Loose to 12 inches rough	0.80
Compacted bulldozer scraped up and down	1.30
Same except root raked	1.20
Compacted bulldozer scraped across slope	1.20
Same except root raked across	0.90
Rough irregular tracked all directions	0.90
Seed and fertilize, fresh	0.64
Same after 6 months	0.54
Seed, fertilizer, and 12 months chemical	0.38
Not tilled algae crusted	0.01
Tilled algae crusted	0.02
Compacted fill	1.24-1.71
Undisturbed except scraped	0.66-1.30
Scarified only	0.76-1.31
Sawdust 2 inches deep, disked in	0.61

Source: National Cooperative Highway Research Program, Report 221. 1980.

Table 9-2
SURFACE STABILIZATION, C_s, FACTORS FOR PERMANENT PASTURE, RANGELAND, IDLE LAND, AND GRAZED WOODLAND[a]

Vegetal Canopy: Type and Height of Raised Canopy[b]	Canopy Cover[c] %	Type[d]	Cover that Contacts the Surface, Percent Ground Cover: 0	20	40	60	80	95-100
No appreciable canopy		G	.45	.20	.10	.042	.013	.003
		W	.45	.24	.15	.091	.043	.011
Canopy of tall weeds or short brush (20-in fall ht.)	25	G	.36	.17	.09	.038	.013	.003
		W	.36	.20	.13	.083	.041	.011
	50	G	.26	.13	.07	.035	.012	.003
		W	.26	.16	.11	.076	.039	.011
	75	G	.17	.10	.06	.032	.011	.003
		W	.17	.12	.09	.068	.038	.011
Appreciable brush or bushes (6-1/2-ft fall ht.)	25	G	.40	.18	.09	.040	.013	.003
		W	.40	.22	.14	.087	.042	.011
	50	G	.34	.16	.08	.038	.012	.003
		W	.34	.19	.13	.082	.041	.011
	75	G	.28	.14	.08	.036	.012	.003
		W	.28	.17	.12	.078	.040	.011
Trees but no appreciable low brush (13-ft fall ht.)	25	G	.42	.19	.10	.041	.013	.003
		W	.42	.23	.14	.089	.042	.011
	50	G	.39	.18	.09	.040	.013	.003
		W	.39	.21	.14	.087	.042	.011
	75	G	.36	.17	.09	.039	.012	.003
		W	.36	.20	.13	.084	.041	.011

Source: USDA--AH537 (1978).

[a]All values shown assume: (1) random distribution of mulch or vegetation, and (2) mulch of appreciable depth where it exists. Idle land refers to land with undisturbed profiles for a period of at least 3 consecutive years. Also to be used for burned forest land and forest land that has been harvested less than three years ago.

[b]Average fall height of waterdrops from canopy to soil surface.

[c]Portion of total area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).

[d]G: Cover at surface is grass, grasslike plants, decaying compacted duff or litter at least 2 inches deep. W: Cover at surface is mostly broadleaf herbaceous plants (such as weeds with little lateral root network near the surface) and/or undecayed residue.

Table 9-3
SURFACE STABILIZATION, C_s, FACTORS FOR UNDISTURBED WOODLAND

Effective Canopy[a] % of Area	Forest Litter[b] % of Area	C_s Factor[c]
100-75	100-90	.0001-.001
70-40	85-75	.002-.004
35-20	70-40	.003-.009

Source: USDA--AH537 (1978).

[a]Where effective litter cover is less than 40 percent or canopy cover is less than 20 percent, the area should be considered as grassland or idle land with C_s selected from Table 9-2. Where woodlands are being harvested, grazed or burned, use Table 9-2.

[b]Forest litter is assumed to be at least two inches deep over the percent ground surface area covered.

[c]The range in C_s values is due in part to the range in the percent area covered. In addition, the percent of effective canopy and its height has an effect. Low canopy is effective in reducing raindrop impact and in lowering the C_s factor. High canopy, over 13 meters, is not effective in reducing raindrop impact and will have no effect on the C_s value.

Table 9-4
MULCH SURFACE STABILIZATION, C_s, FACTORS AND LENGTH LIMITS FOR CONSTRUCTION SLOPES[a]

Type of Mulch	Mulch Rate (tons per acre)	Land Slope (percent)	Factor C_s	Length Limit[b] (ft)
None	0	all	1.0	--
Straw or hay, tied down by anchoring and tacking equipment[c]	1.0	1-5	0.20	200
	1.0	6-10	.20	100
	1.5	1-5	.12	300
	1.5	6-10	.12	150
	2.0	1-5	.06	400
	2.0	6-10	.06	200
	2.0	11-15	.07	150
	2.0	16-20	.11	100
	2.0	21-25	.14	75
	2.0	26-33	.17	50
	2.0	34-50	.20	35
Crushed stone, ¼ to 1½ in	135	<16	.05	200
	135	16-20	.05	150
	135	21-33	.05	100
	135	34-50	.05	75
	240	<21	.02	300
	240	21-33	.02	200
	240	34-50	.02	150
Wood chips	7	<16	.08	75
	7	16-20	.08	50
	12	<16	.05	150
	12	16-20	.05	100
	12	21-33	.05	75
	25	<16	.02	200
	25	16-20	.02	150
	25	21-33	.02	100
	25	34-50	.02	75

Source: USDA--AH537 (1978).

[a]Developed by interagency workshop group on the basis of field experience and limited research data.

[b]Maximum slope length for which the specified mulch rate is considered effective. When this limit is exceeded, either a higher application rate or mechanical shortening of the effective slope length is required.

[c]When the straw or hay mulch is not anchored to the soil, C_s values on moderate or steep slopes of soils having K values greater than 0.30 should be taken at double the values given in this table.

Table 9-5
SURFACE STABILIZATION, C_s, FACTORS FOR SELECTED METHODS OF SURFACE STABILIZATION

Surface Stabilization Method	C_s Factor
Asphalt Emulsion	
1,250 gallons/acre	0.02
1,210 gallons/acre	0.01-0.019
605 gallons/acre	0.14-0.57
302 gallons/acre	0.28-0.60
151 gallons/acre	0.65-0.70
Dust Binder	
605 gallons/acre	1.05
1,210 gallons/acre	0.29-0.78
Other Chemicals	
1,000 lb fiberglass roving with 60-150 gallons/acre	0.01-0.05
Aquatain	0.68
Aerospray 70, 10 percent cover	0.94
Curasol AE	0.30-0.48
Petroset SB	0.40-0.66
PVA	0.71-0.90
Terra-Tack	0.66
Wood fiber slurry[a], 1,000 lb/acre fresh	0.05
Wood fiber slurry[a], 1,400 lb/acre fresh	0.01-0.02
Wood fiber slurry[a], 3,500 lb/acre fresh	0.10
Seedings[b]	
Temporary, 0 to 60 days[c]	0.40
Temporary, after 60 days	0.05
Permanent, 0 to 60 days[c]	0.40
Permanent, 2 to 12 months	0.05
Permanent, after 12 months	0.01
Brush	0.35
Excelsior Blanket With Plastic Net	0.04-0.10

Source: National Cooperative Highway Research Program, Report 221 (1980).

[a] Wood fiber slurry is commonly referred to as hydromulch.

[b] Use minimum C_s values if plantings are performed with mulches.

[c] If dry weather occurs at planting and emergence is delayed, extend the 0-60 days to a period when rainfall normally occurs.

Table 9-6
SURFACE STABILIZATION, C_s, FACTORS FOR
MECHANICALLY PREPARED WOODLAND SITES

Percent of Soil Covered With Residue in Contact With Soil Surface	Soil Condition[a] and Weed Cover[b]							
	Excellent		Good		Fair		Poor	
	NC	WC	NC	WC	NC	WC	NC	WC
None								
A. Disked, raked or bedded[c,d]	.52	.20	.72	.27	.85	.32	.94	.36
B. Burned[e]	.25	.10	.26	.10	.31	.12	.45	.17
C. Drum chopped[e]	.16	.07	.17	.07	.20	.08	.29	.11
10% Cover								
A. Disked, raked or bedded[c,d]	.33	.15	.46	.20	.54	.24	.60	.26
B. Burned[e]	.23	.10	.24	.10	.26	.11	.36	.16
C. Drum chopped[e]	.15	.07	.16	.07	.17	.08	.23	.10
20% Cover								
A. Disked, raked or bedded[c,d]	.24	.12	.34	.17	.40	.20	.44	.22
B. Burned[e]	.19	.10	.19	.10	.21	.11	.27	.14
C. Drum chopped[e]	.12	.06	.12	.06	.14	.07	.18	.09
40% Cover								
A. Disked, raked or bedded[c,d]	.17	.11	.23	.14	.27	.17	.30	.19
B. Burned[e]	.14	.09	.14	.09	.15	.09	.17	.11
C. Drum chopped[e]	.09	.06	.09	.06	.10	.06	.11	.07
60% Cover								
A. Disked, raked or bedded[c,d]	.11	.08	.15	.11	.18	.14	.20	.15
B. Burned[e]	.08	.06	.09	.07	.10	.08	.11	.08
C. Drum chopped[e]	.06	.05	.06	.05	.07	.05	.07	.05

Table 9-6--Continued

Percent of Soil Covered With Residue in Contact With Soil Surface	Soil Condition[a] and Weed Cover[b]							
	Excellent		Good		Fair		Poor	
	NC	WC	NC	WC	NC	WC	NC	WC
80% Cover								
A. Disked, raked or bedded[c,d]	.05	.04	.07	.06	.09	.08	.10	.09
B. Burned[e]	.04	.04	.05	.04	.05	.04	.06	.05
C. Drum chopped[e]	.03	.03	.03	.03	.03	.03	.04	.04

Source: USDA--AH537 (1978).

[a]Excellent--Highly stable soil aggregates in topsoil with fine tree roots and litter mixed in.
Good--Moderately stable soil aggregates in topsoil or highly stable aggregates in subsoil (topsoil removed during raking), with only traces of litter mixed in.
Fair--Highly unstable soil aggregates in topsoil or moderately stable aggregates in subsoil, with no litter mixed in.
Poor--No topsoil, highly erodible soil aggregates in subsoil, with no litter mixed in.

[b]NC--No live vegetation
WC--75 percent cover of grass and weeds, having an average drop fall height of 20 inches. For intermediate percentages of cover, interpolate between columns.

[c]Multiply Item A values by following values to account for surface roughness:

Very rough, major effect on runoff and sediment storage, depressions greater than 6 inches	.40
Moderate	.65
Smooth, minor surface sediment storage, depressions of less than 2 inches	.90

[d]The C_s values for Item A are for the first year following treatment. For A-type sites 1 to 4 years old, multiply C_s value by .7 to account for aging. For sites 4 to 8 years old, use Table 9-2. For sites more than 8 years old, use Table 9-3.

[e]The C_s values for B and C areas are for the first 3 years following treatment. For sites treated 3 to 8 years ago, use Table 9-2. For sites treated more than 8 years ago, use Table 9-3.

Table 9-7
EXAMPLE CALCULATION OF THE SURFACE STABILIZATION, C_s, FACTOR FOR EXPOSURE SCHEDULING

Time Period	Surface Cover	C_s Factor	Fraction of Annual R During Time Period[a]	Weighted C_s Factor[b]
1/1-4/1	Undisturbed Woodland	.003	0.20	0.0006
4/1-6/1	Cleared Site	1.0	0.18	0.18
6/1-8/1	Temporary Seeding	0.40	0.31	0.12
8/1-12/31	Permanent Seeding	0.05	0.31	0.02

Note: Composite C_s for exposure scheduling is the sum of each weighted C_s factor or 0.32.

[a] Obtained from Figure 6-1 as the cumulative percentage at time 2 minus the cumulative percentage at time 1.

[b] Product of the C_s factor and the fraction of annual R during the specified time period.

Table 9-8
KINEMATIC VISCOSITY OF WATER
AT SELECTED TEMPERATURES

Temperature, °F	Kinematic Viscosity, ft^2/sec
32	1.931×10^{-5}
40	1.664×10^{-5}
50	1.410×10^{-5}
60	1.217×10^{-5}
70	1.059×10^{-5}
80	0.930×10^{-5}
90	0.826×10^{-5}
100	0.739×10^{-5}

Source: Metcalf and Eddy (1971).

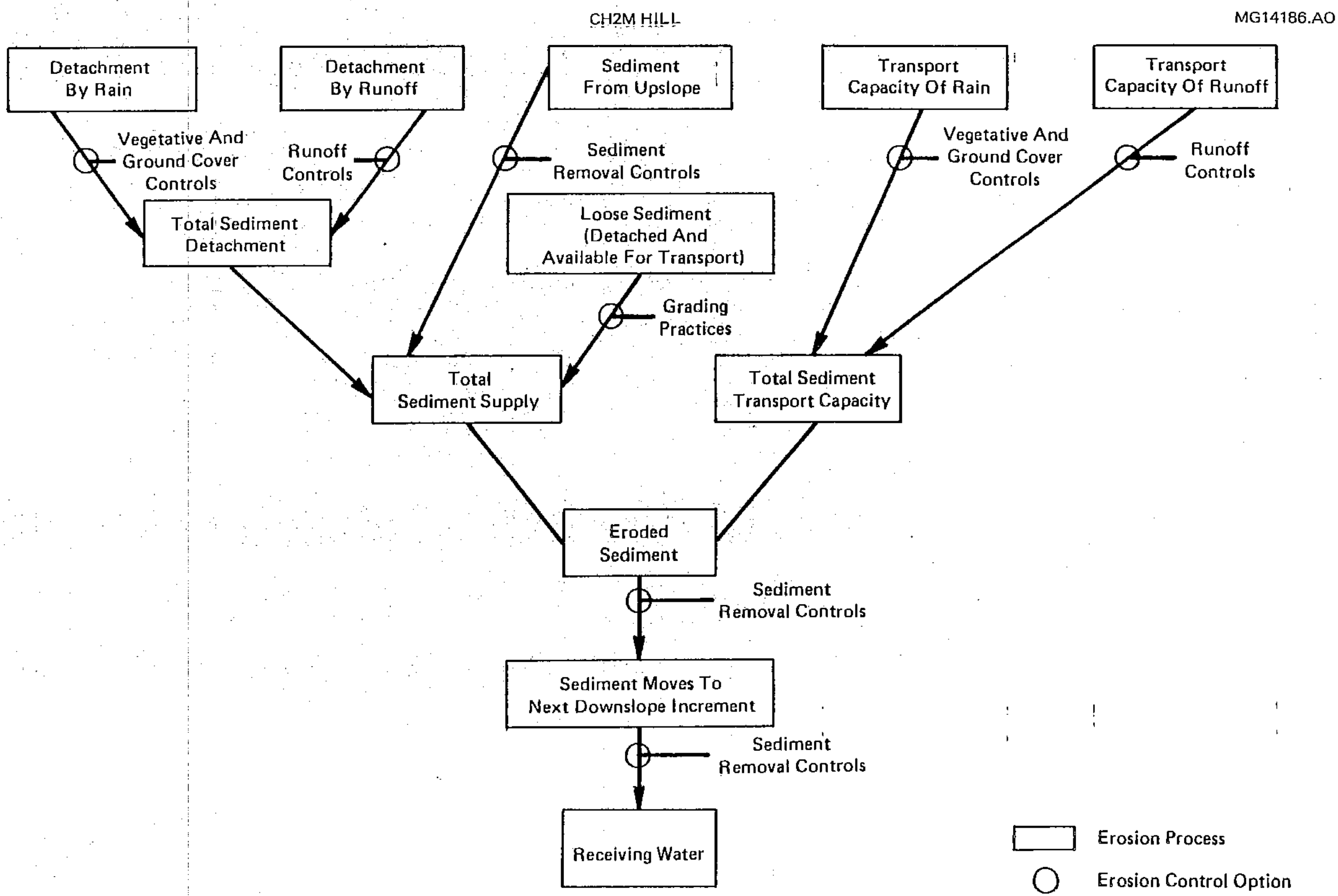

FIGURE 9-1. Erosion process and control schematic.

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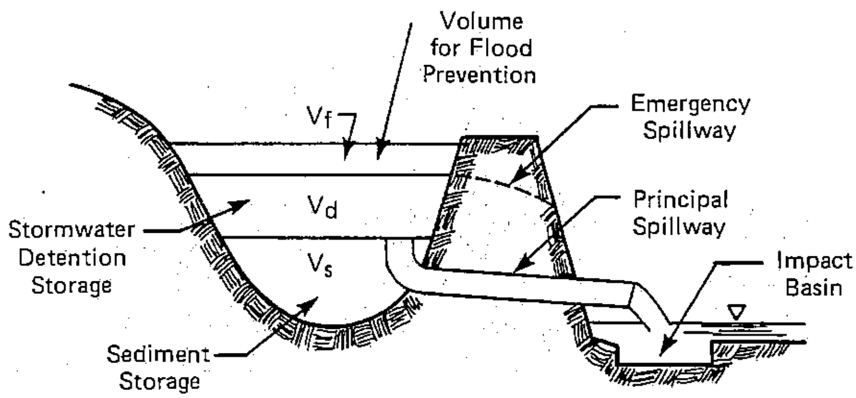


FIGURE 9-2. Components of a typical sediment basin.

SECTION 1.0 INTRODUCTION

Each year flooding causes considerable financial losses to businesses and residences in many metropolitan areas throughout the country. The City of Montgomery is no exception. For instance, during May of 1978, an intense thunderstorm caused citywide flooding with damage estimates exceeding $450,000.

A general rule to follow in urban stormwater management is to correct all possible downstream problems before correcting those upstream. If downstream flooding problems cannot be corrected, upstream peak flows must somehow be attenuated to relieve downstream problems. This chapter lists major methods of mitigating flood damage, discusses inherent advantages and disadvantages of each, and emphasizes those methods found particularly applicable to the Montgomery area.

SECTION 2.0 FLOOD-PLAIN RESOLUTIONS

Flood-plain management in Montgomery is guided by two resolutions which were adopted by the Board of Commissioners of the City of Montgomery on January 22, 1974. Copies of these resolutions are provided in Appendix C of this manual.

Resolution 38-74 deals with delineating areas with special flood hazards and assures the Federal Insurance Administration that provisions consistent with the criteria set forth in Section 1910 of the National Flood Insurance Program Regulations will be enforced for land use within areas having flood hazards. Resolution 39-74 defines the responsibilities of the Department of Inspection, the City Engineer, and the Water Works and Sanitary Sewer Board when plans and specifications for proposed construction are reviewed.

SECTION 3.0 MONTGOMERY FLOOD PLAINS

Established flood plains are located along major ditches, sloughs, and streams in Montgomery. In addition, the Alabama River flood plain has a dominant effect on the Montgomery area. Each of these flood plains has been delineated and is summarized in the <u>Flood Insurance Study for the City of Montgomery</u>. This study was recently completed by the U.S. Army Corps of Engineers and includes a series of flood boundary maps designed to assist Montgomery officials in developing sound flood-plain management strategies. One of the primary purposes of the National Flood Insurance Program is to encourage State and local governments to adopt sound flood-plain management programs.

3.1 Flood Boundaries

In order to provide a national standard without regional discrimination, the 100-year flood has been adopted by the Federal Insurance Administration as the base flood for purposes of flood-plain management measures. The 500-year flood is employed to indicate additional areas of flood risk in the community. For each stream studied in detail in Montgomery, the boundaries of the 100- and 500-year floods have been delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated using topographic maps at scales of 1:12,000, 1:24,000 and 1:62,500 with contour intervals of 10 feet, 10 feet, and 20 feet, respectively. The 1:24,000 and 1:63,500 maps were enlarged so that all work maps were at a scale of 1:13,000. In cases where the 100- and 500-year flood boundaries are close together, only the 100-year boundary has been shown. Flood boundaries for the 100- and 500-year floods are shown on the Flood Boundary and Floodway Maps. Small areas within the flood boundaries may lie above the flood elevations and not be subject to flooding; because of limitations of the map scale, and/or a lack of detailed topographic data, such areas are not shown.

3.2 Floodways

Encroachment on flood plains, such as artificial fill, reduces the flood-carrying capacity and increases flood heights, thus increasing flood hazards in areas beyond the encroachment itself. One aspect of flood-plain management involves balancing the economic gain from flood-plain development against the resulting increase in flood hazard. For purposes of the National Flood Insurance Program, the concept of a floodway is used as a tool to assist local communities in this aspect of flood-plain management. In accordance with this concept, the area of the 100-year flood is divided into a floodway and a floodway fringe. The floodway is the channel of a stream, plus any adjacent flood-plain areas, that must be kept free of encroachment so that the 100-year flood can be carried without substantial increases in flood heights. As minimum standards, the Federal Insurance Administration limits such increases in flood heights to 1.0 foot, provided that hazardous velocities are not produced.

The floodways presented in this study were computed on the basis of equal conveyance reduction from each side of the flood plain. The results of these computations are tabulated at selected cross sections for each stream segment for which a floodway is computed.

Because of the physical characteristics of the Alabama River (low overbank elevations and extremely wide bends), a floodway was not considered feasible and was not computed. A floodway was not computed for Sherwood Ditch, because it is considered to be already fully developed. No floodways were computed downstream of the first cross section for the tributaries to the Alabama and Tallapoosa Rivers due to total inundation of these tributaries by the Alabama and Tallapoosa Rivers.

As shown on the Flood Boundary and Floodway Map, the floodway boundaries were determined at cross sections; between cross sections, the boundaries were interpolated. In cases where the floodway and 100-year flood boundaries are close together, only the floodway boundary has been shown.

The area between the floodway and the boundary of the 100-year flood is termed the "floodway fringe." The floodway fringe thus encompasses the portion of the flood plain that could be completely obstructed without increasing the water-surface elevation of the 100-year flood more than 1.0 foot at any point. Typical relationships between the floodway and the floodway fringe and their significance to flood-plain development are shown on Figure 10-1.

3.3 Flood-Plain Policy

Land development within the vicinity of 100-year flood boundaries in Montgomery must be in accordance with the policy statement presented in Table 10-1. No building permit shall be issued for construction of residential, commercial, or industrial buildings within the flood plain of a projected 100-year storm until all requirements of these regulations are met and plans have been approved by the City Engineering Department.

All encroachments within the 100-year flood plain will require a grading, filling, erosion, and surface drainage plan to be approved by the City Engineering Department. If flood plain development is planned for a wetland area in or near navigable waters, it may be necessary to secure permits and approvals from the U.S. Army Corps of Engineers--Mobile District, U.S. Fish and Wildlife Service--Department of the Interior, and Alabama Water Improvement Commission (Water Quality Certificate), to name a few.

SECTION 4.0 FLOOD DAMAGE MITIGATION APPROACHES

In general, flood mitigation measures primarily address three different approaches to reducing economic and social losses due to flooding:

1. Modification of the extent of flooding.
2. Modification of the impact of flooding.
3. Use of land planning guidance, thus preventing structures which may be damaged by flooding from being located in areas where flooding occurs.

These general approaches are discussed in this section.

4.1 Flood Modification

Flood modification measures are generally structural features designed to reduce peak flows, confine peak flows, improve stormwater transport, or divert peak flows. Structural facilities used for meeting these objectives include channel and culvert improvements, stormwater detention basins, levees, and channel diversions.

Flood modification is expensive but is used extensively in Montgomery for correcting problems which are caused by development in or upstream of areas which are already subject to flooding. Caution must be used in designing and constructing flood modifications, as inappropriate or under-designed structures can create more problems in a major flood than no modification at all, especially if unwise use of flood-plain land is allowed following flood modifications. Levees can fail, detention basins can be full when a flood occurs, trash jammed against a bridge or culvert may cause unexpected stages, or floods greater than the design flood can occur. Some modifications may actually increase the floodcrest height downstream.

4.1.1 Channel and Culvert Improvements. The hydraulic capacity of a ditch or slough can be improved by removing brush and debris, dredging, lining with gabions or concrete, and/or straightening bends. The objective of these methods is to improve stormwater transport by decreasing friction, increasing depth, and/or increasing the channel slope by shortening the channel length. The effect of these improvements on flood heights can be computed by using established hydraulic procedures described in Chapter 7.

Channel improvements cannot be implemented in a random fashion. They must be considered as part of an overall plan for each watershed so that the new hydraulics of flood transport do not cause problems upstream or downstream of the improvement. In addition, consideration must be given to channel erosion; otherwise, the channel may begin to meander, causing downstream deposition of silt and clay which may reduce the capacity of downstream drainage facilities. The periodic removal of this silt and clay may represent a significant maintenance cost to the City.

An example illustrating why channel improvements should not be made indiscriminantly is shown on Figure 10-2. In this example, it can be seen that channelization of the entire reach of West End Ditch would have not only been expensive but would have also resulted in downstream peak flows too large to be contained by a reasonable size channel cross section. If this channelization had occurred without changes to downstream facilities, extreme and more frequently flooding would have resulted. Peak flows resulting from upstream channelization must be consistent with downstream capacities.

When developing subdivisions in new upstream areas, the preferable option, from a flood damage mitigation standpoint (if sufficient capacity exists), is to leave the natural channel alone. Improvements to the channel could drastically increase flows downstream by increasing velocities and decreasing the travel time of the peak flows.

The obvious advantage of implementing channel improvements for flood damage mitigation is that flood stages can be reduced by improving the hydraulic capacities of ditches, sloughs, and streams. In addition, the impact of channel improvements can be computed using established hydraulic methodologies, including those summarized in Chapter 7. Channel improvements can be used in combination with other flood modification methods and can be tailored to site-specific flooding problems.

There are a number of general disadvantages that should be understood before channel improvement programs are implemented. A careful, site-specific analysis of the improved channel hydraulics and its effect on flood routing is required for consistent design. As was shown earlier, channel improvements upstream may cause new or intensify existing flooding conditions downstream. Channel improvement programs are usually quite expensive to implement, and public funds used to construct these improvements are limited.

Culverts are used to convey stormwater under highways, streets, and driveways that cross waterways such as ditches and sloughs. These culverts play an important role in flood modification.

When an undersized culvert is placed in a channel, the excess water which cannot pass through the culvert builds up behind the culvert until it flows over the road and adjacent banks. Usually, the drainage pattern is such that the overflow does not readily flow back into the channel downstream of the crossing; rather, it sheet-flows across adjacent property, causing damage and disruption. The small culvert in essence acts as a channel block and in some cases increases the damage caused by the stormwater. This is not an isolated problem but occurs repeatedly throughout the City of Montgomery. The use of appropriately sized culverts yields savings both by reducing long-term flood damage and by avoiding the costs involved in replacing small culverts with larger ones.

Upstream culverts which are not adequately sized to carry the design flow have a significant effect on the downstream hydrology. Figure 10-3 shows the influence upstream ponding can have on peak runoff flows downstream. Provisions for upstream ponding are usually made using stormwater detention basins.

4.1.2 Stormwater Detention Basins. Temporary stormwater storage is commonly called "detention storage," in contrast to "retention storage," which is the function of basins; designed to have a permanent pool of standing water. The potential for peak flow reduction is likely to be lower with retention storage. A discussion of the differences between detention and retention storage is presented in Subsection 2.2 of Chapter 8. With proper sizing, the discharge from the retention basin can be designed to match the capacity of the downstream channel.

As discussed in Chapter 8, the location, size, and outlet for each basin must be selected on a site-specific basis depending on the storage characteristics of the basin and the nature of the flood problem to be solved. In general, the discharge capacity of the outlet for a detention basin should equal the maximum flow which the floodway channel can pass without causing damage. To be effective in attenuating peak flow, the basin storage capacity must equal the flow volume of the design flood less the volume of water released during the flood.

Detention basins can be effectively included in the design of new developments. Since they will drain soon after a major runoff event, the land can be used for other purposes such as parks, playgrounds, or other greenspace. Construction of detention basins should be accompanied by an education campaign so that the public understands how and why the basins work. The public may then find it easier to support the capital and operational expenditures necessary to build and maintain (cleaning out accumulated debris and sediment) the normally empty basins.

4.1.3 Levees or Flood Walls. Levees or flood walls are longitudinal dams erected on the flood plain parallel to and outside of the main channel. The objective of constructing a levee is to confine peak flood flows. In drainage basins which have steep gradients and small drainage areas, levees have limited usefulness for confining floods.

Levees are usually constructed of excavated soil. Because of their flat side slopes, levees of any substantial height require a large base width. As a result, land costs for levees in urban areas may be prohibitive. In such cases, concrete flood walls may be a better solution.

Foundation conditions and building materials for levees are rarely adequate thus, there is a danger of failure. There are many possible causes of levee failure, and few levees can be assumed totally safe during a flood. However, the short duration of peak flows in Montgomery makes them much more suitable than in many areas of the country. Regular inspections of levees should include inspection for evidence of bank caving, weak spots created by animals or vegetation, foundation settlement, bank sloughing, erosion around outlets of sewers and pipes, and all other sources of hazard. Levees, flood walls, and appurtenances should be monumented with horizontal and vertical survey markers so that ground movement can be documented as part of a safety program. Usually, levee or flood wall construction will result in higher flood stages along a flood plain unless reservoir or channel improvements are provided. Therefore, a careful consideration of physical, hydrologic, and hydraulic characteristics is required for adequate design.

4.1.4 Channel Diversions. A diversion structure routes floodwaters away from or around areas susceptible to damage. Opportunities for the construction of diversion floodways are limited by the topography and geology of the area and by the availability of low-value land which can be used for the diversion. Channel diversions must be evaluated on a site-by-site basis to determine the effect of a bypass on the stage downstream where the bypass rejoins the main channel. Bypasses must be properly designed to accommodate the peak flow and velocities for the site.

4.1.5 Vegetative Cover. The capacity for temporary storage of water through interception by the leaves and other portions of vegetal matter as well as storage in the soil can have a significant impact on surface runoff from small storms in Montgomery. The vegetal cover, in combination with soil, can function as a detention basin which captures a significant portion of rainfall from events with depth in the 1- to 3-inch/24-hour range. Runoff can then occur only when this storage capacity has been filled. This abstraction, the retention capacities of bush, pasture, and properly landscaped, low-density development, can have a significant positive effect on reducing runoff downstream during minor rainfall events.

4.2 Flood Impact Modification

Where development has occurred in floodprone areas and corrective action is not feasible for economic or other reasons, consideration should be given to modifying the impact of the flooding. The impact of flooding on existing developments can be reduced significantly if structural floodproofing of individual buildings is combined with responsible planning, public awareness, and flood insurance programs. In practice, the combination of these measures must be evaluated by a site-specific analysis of the magnitude and type of hazard which exists at each unit. It is likely that community meetings may be required to implement an effective program in Montgomery.

4.2.1 Floodproofing. Floodproofing is a process by which buildings are altered to become less vulnerable to flood damages. There are essentially two categories of flood-proofing: permanent and temporary. Permanent measures are those which once installed, remain in place and in effect for the life of the structure (e.g., a raised structure). Temporary measures include removable bulkheads for windows and doors, movement of goods and materials to higher elevations, and sandbagging of entrances. These, of course, are subject to human error and require advance warning of high flood waters.

4.2.1.1 Permanent Floodproofing--Permanent floodproofing can be accomplished by two major methods: (1) raising the damageable portion of the structure, or contents, above the anticipated flood level, and (2) preventing floodwaters from entering the structure.

Except for structures such as mobile homes and small wooden houses, it is usually impractical to consider elevating most existing structures. The elevation of structures is much better suited to planned development in an identified flood plain rather than as correction of existing problems. Montgomery uses this approach for new structures located in or near known flood plains. City policy is to require the developer or builder to set his building slab elevation at least 2 feet above the 100-year flood elevation or the known high water mark, whichever is available for the project site. This is a good example of preventative floodproofing.

Some success might be anticipated in preventing floodwaters from entering structures by constructing watertight walls or levees around the structures or by utilizing the outer walls of the structures themselves. A major problem with these walls is that they are usually not strong enough to resist the force or hydrostatic pressure of the water. The use of simple concrete block construction instead of steel rod and block or reinforced concrete construction with a good foundation is frequent and often results in structural failure of the walls.

For places with walls or levees, consideration should be given to providing a pump to remove water entering the walled area. Pumps can be used to remove rainfall which falls within the confines of the area protected by the flood wall. Using the walls of the structure itself to prevent waters from entering requires special considerations. Most structures are not designed to withstand hydrostatic pressure on the exterior walls. The principal reason most structures do not collapse during flooding is that water enters the structure, equalizing inside and outside pressures. If the objective is to prevent water from entering a structure, it is imperative that the structure be analyzed to ensure that it can withstand the anticipated pressures shown on Figure 10-4. The principal considerations for permanent structural floodproofing are: (1) the exterior walls must be impermeable, (2) all openings below the design flood level must be closed, and (3) the structure must be able to withstand anticipated hydrostatic pressure and buoyancy.

Other methods of floodproofing include flood shield closure at doors and windows and backflow devices on sewer and water lines. It should be noted that permanent floodproofing of new and old structures does not eliminate a flood hazard, but merely reduces the damage potential.

Because of the nature of floods in Montgomery, where the inundation from stream flooding is generally less than a few hours for most locations, floodproofing can be planned for effectiveness for just a short time period and utilities need not function during that period. At the same time, sewers should not back up into the structures, nor should the water supply become contaminated. Automatic backflow devices or hand valves should be installed on main water service lines and sanitary sewer systems that have openings below the regulatory flood elevation.

Within an existing structure or group of structures, damageable property can often be placed in a less damageable location or protected in place. Examples of such action may include the following: protecting water heaters, air conditioners, washers, dryers, and shop equipment by relocating them at a higher elevation; relocating commercial and industrial finished products, merchandise, and equipment to a higher floor or adjacent higher buildings; and anchoring all property, such as gas bottles, subject to movement by floodwaters.

In the construction of new structures in a flood hazard area or repairing existing structures, water-resistant materials and damage-reducing construction practices can be used to reduce the potential for damage. A list of other such practices and materials is presented in Table 10-2. However, it is important to reiterate that if the objective is to prevent water from entering a structure, it is imperative that the structure be designed to withstand the anticipated stresses, with a firmly anchored foundation to prevent flotation.

4.2.1.2 Temporary Floodproofing--Temporary floodproofing must be put in place each time it is needed. This requires reliable advanced warning, along with time and effort to install the material. Since flooding is often short-lived, these measures can be effective for certain applications. The greatest question in any potential flood situation is whether or not to stay in or adjacent to a structure and try to protect it.

Some of the materials which can be used for temporary floodproofing are:

1. Sand bags to block flow.
2. Plywood to cover windows and/or entrances.
3. Towels stuffed around doors to reduce leakage.
4. Wood or plastic to cover fences, creating a wall.
5. Wood beams to brace fences, doors, etc.
6. Concrete or wood blocks to raise furniture off the floor.

4.2.2 Response Planning. Response planning includes flood forecasting, disaster preparedness, and postflood recovery. Flash flood warning forecasting is incorporated as part of the weather report from the Weather Service Office at Dannelly Field. It is broadcast regularly on news programs in Montgomery and appears in the local papers as part of the weather forecast. Although these warnings usually come before periods of flooding, they are also issued when floods do not occur or when they occur only in parts of Alabama. The fact that there are many more warnings than there are floods tends to reduce public action and relax vigilance when the warnings are heard. It is, however, very doubtful that a better forecasting system can be devised.

Floods in Montgomery are usually preceded by very intense rains, which give further warning to occupants. An important part of response planning is disaster preparedness, which includes maintenance of vital services (e.g., energy, water, hospitals, sewage, and traffic control), identification of safe evacuation routes, temporary stations of safety and shelter, and the availability of manpower and equipment. Postflood recovery provides a unique opportunity for implementing long-term flood mitigation plans at existing developments since the issue is then fresh in everyone's mind.

4.2.3 Public Awareness. Public awareness of potential flood hazards and possible measures for reducing flood damage is an essential element of an effective stormwater management program. Although information is usually available in many forms and from many sources, the transfer of this information to government personnel and flood-plain residents is not possible without a captive audience which understands that a flood hazard exists. Some flood-plain residents may not believe that the hazard exists until floodwaters actually enter the house. The implementation of a flood damage mitigation program must include long-term public awareness and flood response plans, so that when the issue is critical, proper decisions can be made.

Future and existing developments will be affected by a public awareness program. The average Montgomery citizen needs to become more familiar with the existence and meaning of the City flood hazard maps. Methods of conveying this information may include community meetings, radio, television, and newspaper releases. Signs posted along designated regulatory floodways might also spur interest, especially in newly developing subdivisions. Program policies and guidelines should be subjected to public exposure and hearings so that the public has a more personal understanding of what must be done, e.g., the enforcement of building codes and the implementation of land use guidance.

4.2.4 Flood Insurance. Flood insurance of course does not reduce flood damage for either existing or future development, but does indemnify policy-holders for financial losses suffered during a flood. Insurance is a mechanism for spreading the cost of losses both over time and over a relatively large number of similarly exposed risks. The National Flood Insurance Program, in which Montgomery is participating, involves Federally subsidized flood insurance for existing property in flood hazard areas. It is provided to communities in return for their enactment and enforcement of flood-plain management regulations designed to reduce flood losses by regulation of new development in the designated flood hazard areas.

The program assists by transferring flood hazard information through its insurance rate structure and aids those people who built or bought in the flood plain before the flood hazard information was readily available. The indemnification has limitations in the magnitude and the type of damage covered. The deductible provisions in the policies also make the insurance less attractive for recovering losses on low-damage flooding, but it is an excellent bargain and service to those who need it.

In order to establish actuarial insurance rates, the Federal Insurance Administration has developed a process to transform the data from their <u>Flood Insurance System</u> into flood insurance criteria. This process includes the detailed determination of reaches, Flood Hazard Factors, and flood insurance zone designations for each flooding source study affecting the City of Montgomery.

<u>4.2.4.1 Reach Determinations</u>--Reaches are defined as lengths of watercourses having relatively the same flood hazard, based on the average weighted difference in water surface elevations between the 10- and 100-year floods. This difference does not have a variation greater than that indicated in the following table for more than 20 percent of the reach:

10- and 100-Year Floods	Variation
Less than 2 feet	0.5 foot
2 to 7 feet	1.0 foot
7.1 to 12 feet	2.0 feet
More than 12 feet	3.0 feet

The locations of the reaches determined for the flooding sources of Montgomery are shown on the Flood Profiles and are summarized in the <u>Flood Insurance Study for the City of Montgomery</u>.

<u>4.2.4.2 Flood Hazard Factors</u>--The Flood Hazard Factor (FHF) is the Federal Insurance Administration device used to correlate flood information with insurance rate tables. Correlations between property damage from floods and their FHF are used to set actuarial insurance premium rate tables based on FHFs from 005 to 200.

The FHF for a reach is the average weighted difference between the 10- and 100-year flood water surface elevations expressed to the nearest one-half foot, and shown as a three-digit code. For example, if the difference between water surface elevations of the 10- and 100-year floods is 0.7 foot, the FHF is 005; if the difference is 1.4 feet; the FHF is 015; if the difference is 5.0 feet, the FHF is 050. When the difference between the 10- and 100-year water surface elevations is greater than 10.0 feet, accuracy for the FHF is to the nearest foot.

<u>4.2.4.3 Flood Insurance Zones</u>--After the determination of reaches and their respective Flood Hazard Factors, the entire incorporated area of the City of Montgomery was divided into zones, each having a specific flood potential or hazard. Each zone was assigned one of the following flood insurance zone designations:

Zone A:	Special Flood Hazard Areas inundated by the 100-year flood, determined by approximate methods; no base flood elevations shown or Flood Hazard Factors determined.
Zones A2-A10 and A14:	Special Flood Hazard Areas inundated by the 100-year flood, determined by detailed methods; base flood elevations shown, and zones subdivided according to Flood Hazard Factors.
Zone B:	Areas between the Special Flood Hazard Areas and the limits of the 500-year flood, including areas of the 500-year flood plain that are protected from the 100-year flood by dike, levee, or other water control structure; also areas subject to certain types of 100-year shallow flooding where depths are less than 1.0 foot; and areas subject to 100-year flooding from sources with drainage areas less than 1 square mile. Zone B is not subdivided.
Zone C:	Areas of minimal floodings.

The flood elevation differences, Flood Hazard Factors, flood insurance zones, and base flood elevations for each flooding source studied in detail in the community are summarized in the Flood Insurance Study for the City of Montgomery.

4.2.4.4 Flood Insurance Rate Map Description--The Flood Insurance Rate Map for the City of Montgomery is, for insurance purposes, the principal result of the Flood Insurance Study. This map (published separately) contains the official delineation of flood insurance zones and base flood elevation lines. Base flood elevation lines show the location of the expected whole-foot-water-surface elevations of the base (100-year) flood. This map is developed in accordance with the latest flood insurance map preparation guidelines published by the Federal Insurance Administration.

4.2.5 Tax Adjustments. Tax adjustments can play a role both in influencing decisions about flood-plain occupancy and in providing relief to individuals. Specialized tax provisions could be used to encourage appropriate land use and discourage inappropriate land use.

4.3 Land Use Guidance

Through proper regulations and planning, the land use in flood-prone areas can be guided to reduce the opportunity for damage that could occur in the area. The measures used for land use guidance include regulations and planning policies.

4.3.1 Regulations. Available methods of regulation include flood-plain resolutions, subdivision regulations, and building codes. The City of Montgomery uses its subdivision regulations along with its zoning ordinance to guide land use practices. The flood-plain resolutions provided in Appendix C have the potential of being effective in mitigating future flood damages in Montgomery. However, the success of these resolutions will depend upon public awareness of flood hazards and the willingness of the government to abide by and enforce them.

SECTION 5.0 REFERENCES

1. CH2M HILL, 1979. West End Ditch Drainage Study. Prepared for the City of Montgomery.

2. Johnson, W. K., 1978. "Physical and Economic Feasibility of Nonstructural Flood-Plain Management Measures," The Hydrologic Engineering Center, Davis, California.

3. Wright-McLaughlin Engineers, 1969. Urban Storm Drainage Criteria Manual, Volumes I and II. Prepared for the Denver Regional Council of Governments.

Table 10-1
FLOOD-PLAIN POLICY FOR LAND DEVELOPMENT
WITHIN THE VICINITY OF 100-YEAR FLOOD
BOUNDARIES IN MONTGOMERY

1. The minimum elevation of any land developed shall be 0.5 foot above the projected water level of the 100-year flood.

2. The finished floor elevation of all structures shall be a minimum of 2 feet above the projected water level of the 100-year flood.

3. The elevation of the top of street curbs shall be a minimum of 1 foot above the projected flood level of the 100-year flood.

4. No development shall encroach the 100-year floodway.

5. All encroachments within the 100-year flood plain will require a grading, filling, erosion, and surface drainage plan to be approved by the City Engineering Department.

6. No encroachment in the flood plain shall increase the flood water heights by more than 1 foot.

7. No building permit shall be issued for construction of residential, commercial, or industrial buildings within the flood plain of a projected 100-year storm until all requirements of these regulations are met and plans have been approved by the City Engineering Department.

Table 10-2
CONSTRUCTION MATERIALS AND
PRACTICES TO REDUCE POTENTIAL
FLOOD DAMAGES

1. Overhead or watertight underground energy and telephone lines.

2. Large space for temporary elevated storage of contents during flood hazard.

3. Elevated main electrical distribution box.

4. Elevated electrical outlets.

5. Water-damage-resistant woodwork (cabinetry, etc.).

6. Anchored propane gas tanks.

7. Elevated outside vent for the discharge of air from clothes dryers.

8. Impermeable or damage-resistant thermal and acoustical insulation.

9. Water-resistant wall material: polyester epoxy paint, plastic tiles, treated wood beams, etc.

10. Outside drainpipe with valve at floor level for draining water trapped in the house.

11. Sewer gate valve to prevent backflow of sewage.

12. Sump pump for cleanup.

13. Extra-wide doors for rapid removal of furniture.

14. Water-damage-resistant carpeting.

15. Water-damage-resistant floor finish: linoleum, rubber, vinyl.

16. Anchorage to foundation to prevent flotation and/or overturning, especially for mobile homes.

Source: Johnson, 1978.

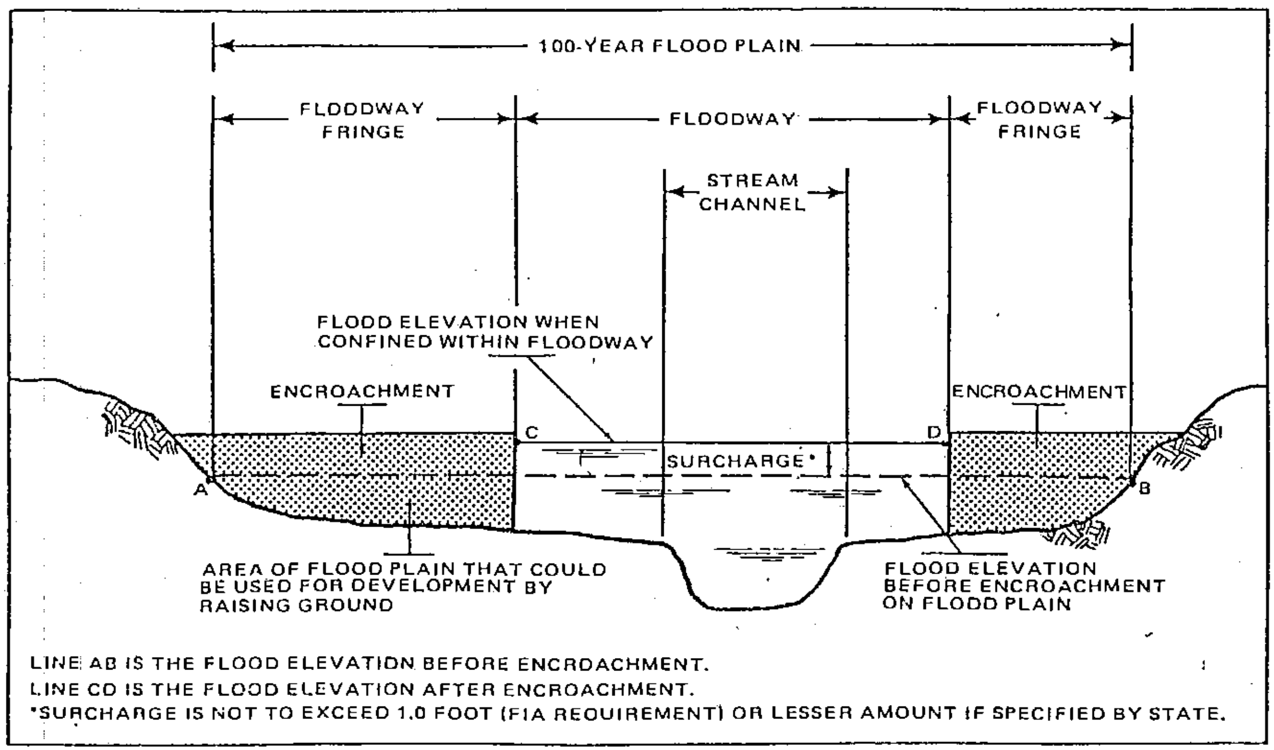


FIGURE 10-1. Floodway schematic used in flood insurance studies.

CH2M HILL MG14186.AO

SOURCE: WEST END DITCH DRAINAGE STUDY, CH2M HILL 1979

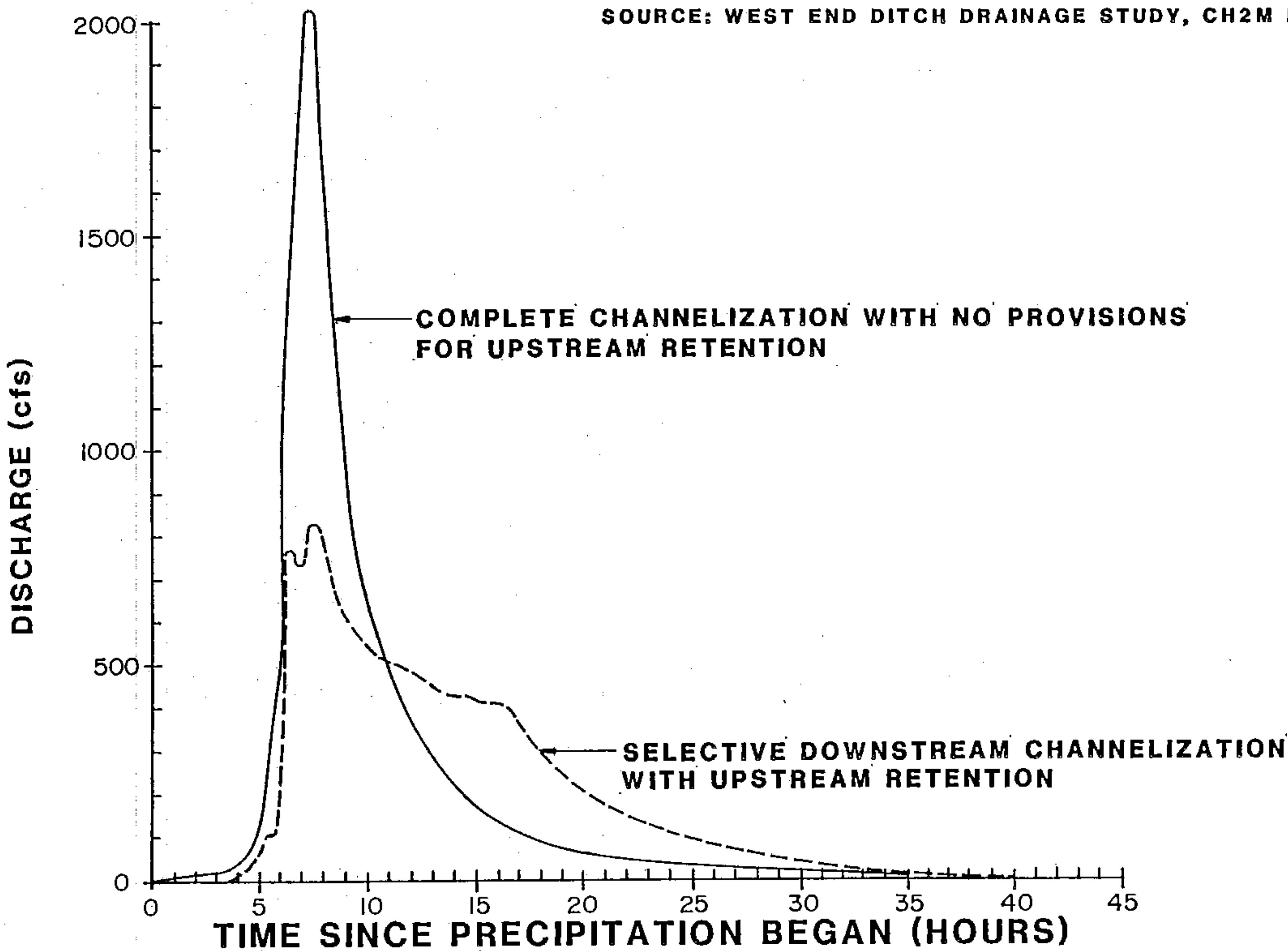


FIGURE 10-2. Effects of channelization on West End Ditch hydrology.

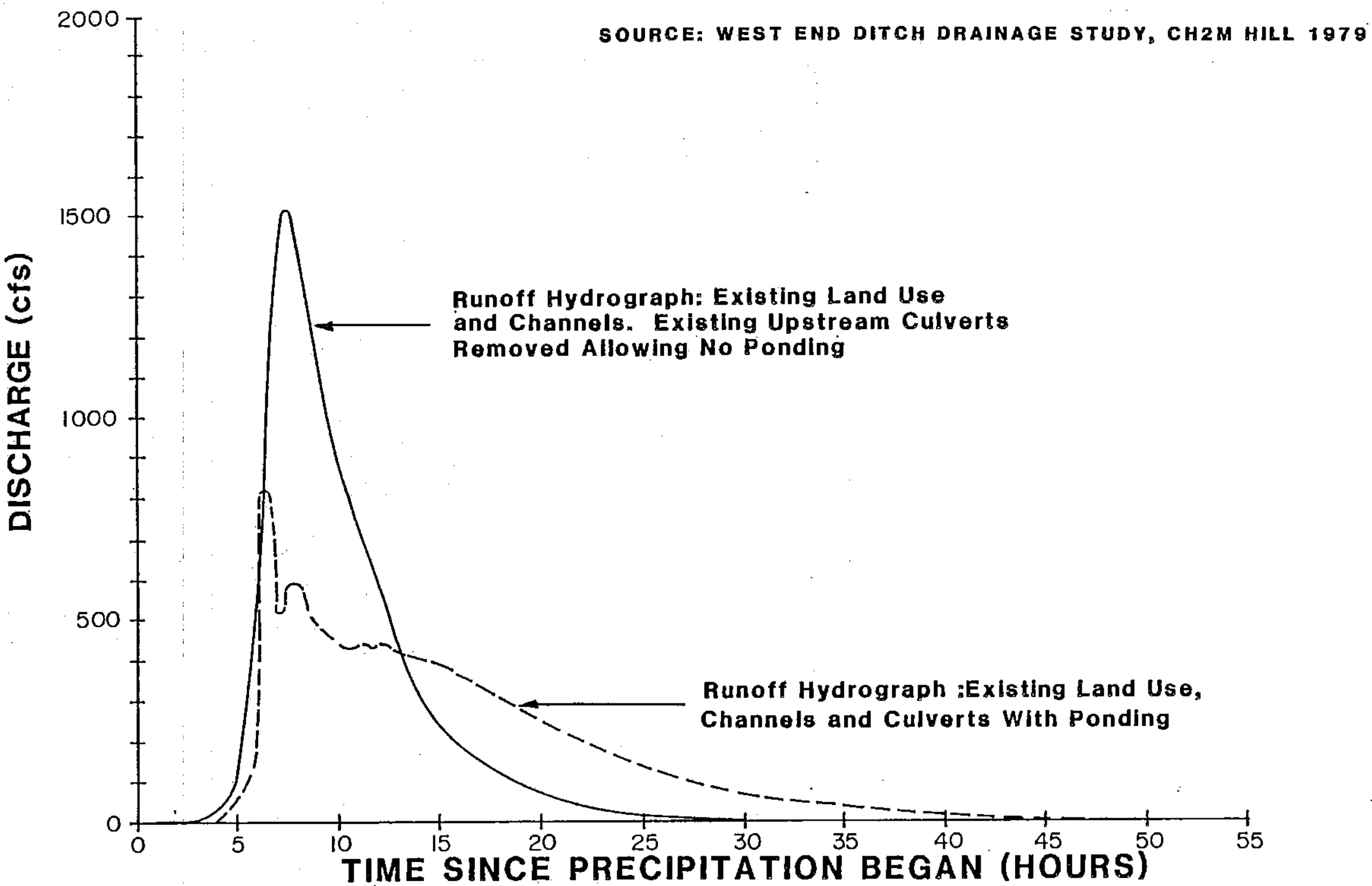
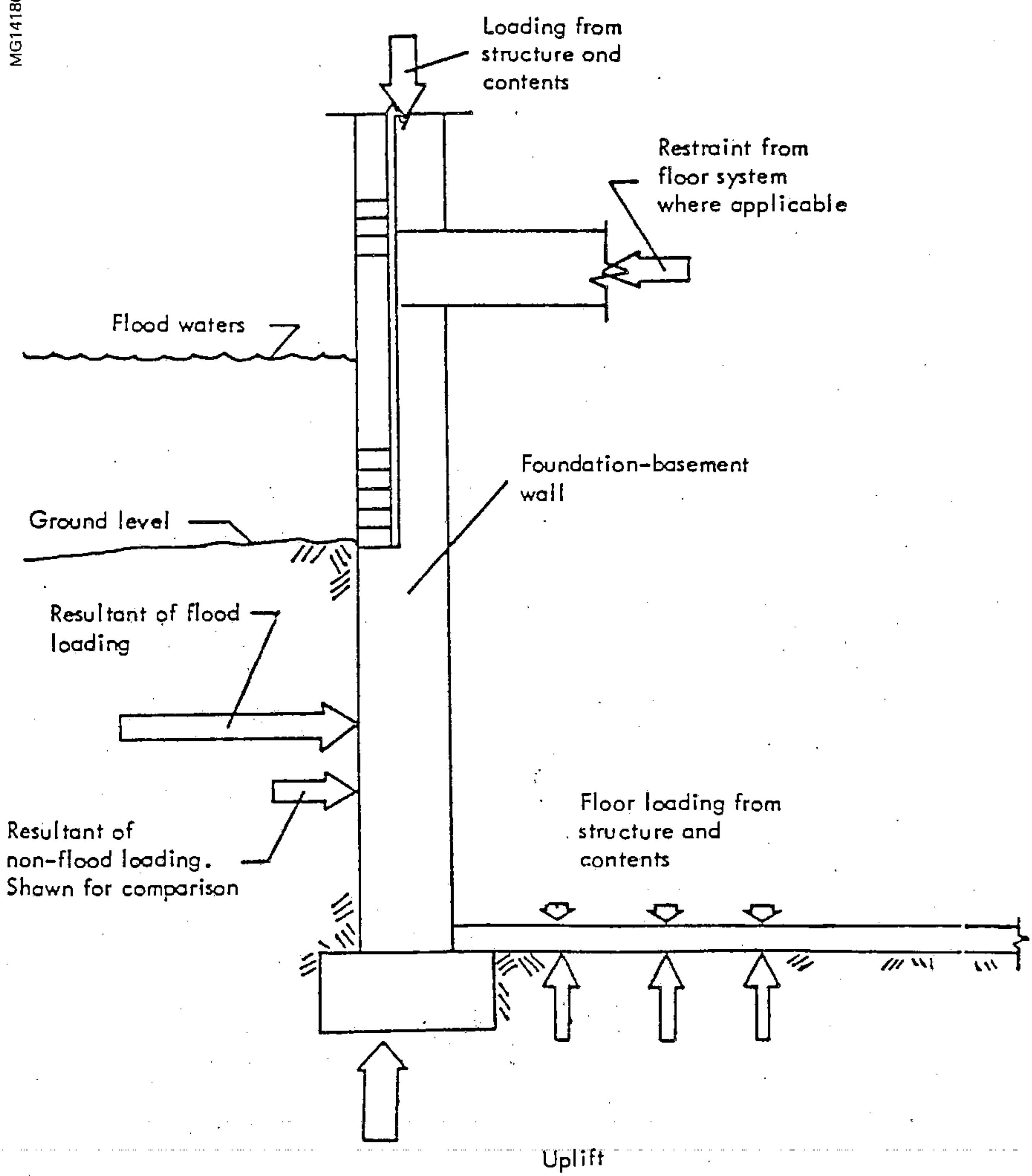


FIGURE 10-3. Effects of upstream ponding on West End Ditch hydrology.

MG14186.AO



Source: Wright-McLaughlin (1969).

FIGURE 10-4. General forces acting on a below-grade structure subjected to the hydrostatic pressures resulting from flood waters.

MG14186.AO

U.S. customary unit			SI	
Name	Abbreviation	Multiplier	Symbol	Name
acre	acre	0.405	ha	hectare
acre-foot	acre-ft	1,233.5	m^3	cubic metre
acre-inch	acre-in.	102.79	m^3	cubic metre
cubic foot	ft^3	28.32	L	litre
		0.0283	m^3	cubic metre
cubic feet per minute	ft^3/min	0.0283	m^3/min	cubic metres per minute
cubic feet per minute per 100 gallons	$ft^3/min\cdot 100$ gal	0.00747	$m^3/min\cdot 100$ L	cubic metres per minute per 100 litres
cubic feet per pound	ft^3/lb	62.4	L/kg	litres per kilogram
cubic feet per second	ft^3/s	28.32	L/s	litres per second
cubic feet per square foot per minute	$ft^3/ft^2\cdot min$	0.305	$m^3/m^2\cdot min$	cubic metres per square metre per minute
cubic inch	in.^{3}	16.39	cm^3	cubic centimetre
		0.0164	L	litre
cubic yard	yd^3	0.765	m^3	cubic metre
		764.6	L	litre
degrees Fahrenheit	°F	0.555 (°F-32)	°C	degrees Celsius
feet per minute	ft/min	0.00508	m/s	metres per second
feet per second	ft/s	0.305	m/s	metres per second
foot (feet)	ft	0.305	m	metre(s)
gallon(s)	gal	3.785	L	litre(s)
		3.785×10^{-3}	m^3	cubic metre
gallons per acre per day	gal/acre·d	9.353	L/ha·d	litres per hectare per day
gallons per capita per day	gal/capita·d	3.785	L/capita·d	litres per capita per day
gallons per day	gal/d	4.381×10^{-5}	L/s	litres per second
gallons per foot per minute	gal/ft·min	0.207	L/m·s	litres per metre per second
gallons per minute	gal/min	0.0631	L/s	litres per second
gallons per square foot	gal/ft^2	40.743	L/m^2	litres per square metre
gallons per square foot per day	$gal/ft^2\cdot d$	1.698×10^{-3}	$m^3/m^2\cdot h$	cubic metres per square metre per hour
		0.283	$m^3/ha\cdot min$	cubic metres per hectare per minute
gallons per square foot per minute	$gal/ft^2\cdot min$	2.445	$m^3/m^2\cdot h$	cubic metres per square metre per hour
		0.679	$L/m^2\cdot s$	litres per square metre per second
horsepower	hp	0.746	kW	kilowatts
inch(es)	in.	2.54	cm	centimetre
inches per hour	in./h	2.54	cm/h	centimetres per hour
mile	mi	1.609	km	kilometre
million gallons	Mgal	3.785	ML	megalitres (litres x 10^6)
		3785.0	m^3	cubic metres
million gallons per acre	Mgal/acre	8353	m^3/ha	cubic metres per hectare
million gallons per acre per day	Mgal/acre·d	0.039	$m^3/m^2\cdot h$	cubic metres per square metre per hour
million gallons per day	Mgal/d	43.808	L/s	litres per second
		0.0438	m^3/s	cubic metres per second
million gallons per square mile	$Mgal/mi^2$	1.461	ML/km^2	megalitres per square kilometre
		1461	m^3/km^2	cubic metres per square kilometre
parts per billion	ppb	1.0	mg/L	micrograms per litre
parts per million	ppm	1.0	mg/L	milligrams per litre
pound(s)	lb	0.454	kg	kilogram(s)
		453.6	g	gram(s)
pounds per acre per day	lb/acre·d	1.121	kg/ha·d	kilograms per hectare per day
pounds per cubic foot	lb/ft^3	16.018	kg/m^3	kilograms per cubic metre
pounds per 1000 cubic feet	lb/1000 ft^3	16.018	g/m^3	grams per cubic metre
		0.016	kg/m^3	kilograms per cubic metre
pounds per mile	lb/mi	0.282	kg/km	kilograms per kilometre
pounds per million gallons	lb/Mgal	0.120	mg/L	milligrams per litre
pounds per square foot	lb/ft^2	4.882×10^{-4}	kg/cm^2	kilograms per square centimetre
		4.882	kg/m^2	kilograms per square metre
pounds per 1000 square feet per day	lb/1000 $ft^2\cdot d$	4.882×10^{-3}	$kg/m^2\cdot d$	kilograms per square metre per day
pounds per square inch	lb/in.^{2}	0.0703	kg/cm^2	kilograms per square centimetre
square foot	ft^2	0.0929	m^2	square metre
square inch	in.^{2}	6.452	cm^2	square centimetre
square mile	mi^2	2.590	km^2	square kilometre
		259.0	ha	hectare
square yard	yd^2	0.836	m^2	square metre
standard cubic feet per minute	std ft^3/min	1.699	m^3/h	cubic metres per hour
ton (short)	ton (short)	0.907	Mg (or t)	megagram (metric tonne)
tons per acre	tons/acre	2240	kg/ha	kilograms per hectare
tons per square mile	$tons/mi^2$	3.503	kg/ha	kilograms per hectare
yard	yd	0.914	m	metre

APPENDIX A. Conversion factors U.S. Customary to SI (metric).

The purpose of Appendix B is to identify well-documented, readily available computer programs applicable to stormwater management problems. No user assistance is presented, nor is the list comprehensive. Appendix B does, however, briefly describe selected programs and identifies appropriate reference material for obtaining user assistance.

RUNOFF ESTIMATION PROGRAMS

TR-20:

Developed by the USDA, SCS (1973) using the hydrologic principles presented in NEH-4 (1972), TR-20 will develop a watershed hydrograph, route the hydrograph through reservoirs and channels, and combine hydrographs from various reaches. User information for TR-20 is documented in the following reports:

1. U.S. Department of Agriculture, Soil Conservation Service, 1973. "Computer Program for Project Formulation Hydrology," SCS TR-20, Washington, D.C.

2. U.S. Department of Agriculture, Soil Conservatic Service, 1972. "Hydrology," NEH-4, Washington, D.C.

HYMO:

Developed by the Agricultural Experiment Station at Texas A&M University (1973), HYMO is a problem-oriented computer language designed to transform rainfall data into runoff hydrographs. HYMO also has the capability to route these hydrographs through channels or reservoirs. User information for HYMO is documented in the following publications:

1. U.S. Department of Agriculture, Agricultural Research Service, 1973. "HYMO: Problem Oriented Computer Language for Hydrologic Modeling, User's Manual," ARS-S-9, Texas A&M University.

2. Williams, J. R. 1969. "Flood Routing with Variable Travel Time or Variable Storage Coefficients," Transactions of the ASAE, Vol. 12, No. 1, pp. 100-103.

HEC-1:

This program is one of a series of comprehensive programs developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers (1973). Each program of this series is intended to be a major computational aid for solving problems associated with a particular area of hydrologic engineering. HEC-1 can develop a single storm hydrograph, perform channel routing, and combine and route hydrographs. User information for HEC-1 is documented in the following publications:

1. U.S. Army Corps of Engineers, Hydrologic Engineering Center, 1973. "HEC-1 Flood Hydrograph Package, User's Manual," Generalized Computer Program 723-010, Davis, California.

2. U.S. Army Corps of Engineers, Hydrologic Engineering Center, 1977. "Flood Control System Component Optimization-HEC-1 Capability," Training Document No. 9, Davis, California.

3. U.S. Army Corps of Engineers, Hydrologic Engineering Center, 1979. "Introduction and Application of Kinematic Wave Routing Techniques Using HEC-1," Training Document No. 10, Davis, California.

STORM:

The original version of this program was completed in January, 1973, by Water Resources Engineers, Inc., of Walnut Creek, California, while under contract with the Hydrologic Engineering Center. STORM is a continuous simulation program developed to aid in the sizing of storage treatment facilities to control the quantity and quality of stormwater runoff and land surface erosion. Loads and concentrations of six basic water quality parameters are computed (i.e., suspended and settleable solids, biochemical oxygen demand, total nitrogen, orthophosphate, and total coliform). User information for STORM is documented in the following reports:

1. U.S. Army Corps of Engineers, Hydrologic Engineering Center, 1976. "Storage, Treatment, Overflow, Runoff Model, STORM, User's Manual," Computer Program 723-58-L7520, Davis, California.

2. U.S. Army Corps of Engineers, Hydrologic Engineering Center, 1977. "Guidelines for Calibration and Application of STORM," Training Document No. 8, Davis, California.

HYDRAULIC DESIGN PROGRAMS

ILLUDAS:

The Illinois Urban Drainage Area Simulator (ILLUDAS) is basically an adaptation of the British Road Research storm sewer design model to the peculiarities of the North American climatological conditions. It is a single-storm event model and is a fairly detailed design tool for sewer systems. User information for ILLUDAS is documented in the following report:

1. Terstriep, M. L. and Stall, J. B. 1974. "The Illinois Urban Drainage Area Simulator, ILLUDAS," Bulletin 58, Illinois State Water Survey, Urbana, Illinois.

SWMM:

This computer program was developed for the Environmental Protection Agency by Metcalf and Eddy, Inc., the University of Florida, and Water Resources Engineers, Inc. SWMM is a comprehensive single-event program which considers rainfall/runoff, storm sewer transport, storage and treatment, and receiving water impacts. For hydraulic design problems, the transport block can be used independently of other components to the program. Sewer hydraulics are performed mathematically using a nonlinear kinematic wave equation. User information for SWMM is documented in the following report:

1. Huber, W. C. et al., 1975. "Storm Water Management Model, User's Manual, Version II," EPA-670/2-75-017, Washington, D.C.

ISS:

The Illinois Storm Sewer (ISS) computer model is a highly accurate model considering the unsteady and backwater effects in sewers as well as the effects of junctions and manholes. User information for ISS is documented in the following report:

1. Sevuk, A. S. et al., 1973. "Illinois Storm Sewer System Simulation Model: User's Guide," Research Report No. 73, Water Resources Center, University of Illinois at Urbana-Champaign, Illinois.

CULVERT DESIGN PROGRAMS:

Three computer programs for the hydraulic design of culverts are available from the U.S. Department of Transportation. Each of these programs is based on appropriate hydraulic design charts discussed in

Section 3.0 of Chapter 7. User information for these culvert design programs is documented in the following reports:

1. U.S. Department of Transportation, Federal Highway Administration, 1962. "A Fortran Program for the Hydraulic Design of Circular Culverts," Hydraulic Engineering Circular No. 7, Washington, D.C.

2. U.S. Department of Transportation, Federal Highway Administration, 1968. "Hydralic Analysis of Pipe-Arch Culverts," BPR Program HY-2, Washington, D.C.

3. U.S. Department of Transportation, Federal Highway Administration, 1969. "Hydralic Analysis of Box Culverts," BPR Program HY-3, Washington, D.C.

<u>WATER SURFACE PROFILES</u>

<u>HEC-2</u>:

This computer program computes and plots the water surface profile for river channels of any cross section for either subcritical or supercritical flow conditions. The effects of various hydraulic structures such as bridges, culverts, weirs, embankments, and dams may be considered. The principal use of the program is for determining profiles for various frequency floods for both natural and modified conditions. The latter may include channel improvements, levees, and floodways. User information for HEC-2 is documented in the following report:

1. U.S. Army Corps of Engineers, Hydrologic Engineering Center, 1973. "HEC-2, Water Surface Profiles, User's Manual." Computer Program 723-X6-L202A.

<u>WSP2</u>:

Developed by the USDA, SCS (1973), WSP2 is designed to aid in the determination of flood characteristics for a given set of physical conditions. It will compute 15 water surface profiles for up to a total of 50 reach and road sections. User information for WSP2 is documented in the following report:

1. U.S. Department of Agriculture, Soil Conservation Service, 1973. "WSP2-Water Surface Profile Program, User's Guide," USDA/DF-73-021a, Washington, D.C.

SEDIMENT YIELD

CREAMS:

CREAMS is a physically based, daily simulation model that estimates runoff, erosion/sediment transport, plant nutrient, and pesticide yield from field-sized areas. The erosion component maintains elements of the USLE, but includes sediment transport capacity for overland flow.

A channel erosion/deposition feature of the model permits consideration of concentrated flow within a field. Impoundments are also considered. Although this program is designed specifically for agricultural areas, it may be applicable to developing areas of Montgomery. User information for CREAMS is documented in the following report:

1. U.S. Department of Agriculture, 1980. "CREAMS, A Field Scale Model for Chemicals, Runoff, and Erosion from Agricultural Management Systems." Conservation Research Report No. 26, Washington, D.C.

Additional computer programs which have USLE based erosion/deposition features include STORM and HYMO.

SEDIMENT RETENTION BASINS

DEPOSITS:

This computer program was prepared for the Institute for Mining and Minerals Research by the Department of Agricultural Engineering at the University of Kentucky (Ward et al., 1979). DEPOSITS (DEposition Performance of Sediment In Trap Structures) is a conceptual model which describes the sediment transport and deposition process in a reservoir as a function of the basin geometry, inflow hydrograph, the inflow sedimentograph, the sediment characteristics, the outlet spillway design, and the hydraulic behavior of the flow within the basin. The model may be used to determine basin trap efficiency, loss in storage due to sediment accumulations, and effluent suspended sediment concentrations. User information for DEPOSITS is documented in the following report:

1. Ward, A. et al., 1979. "The DEPOSITS Sedimentation Pond Design Manual," University of Kentucky, Lexington, Kentucky.

PROGRAMMABLE CALCULATORS

Programmable calculators can be useful aids for performing hydrologic and hydraulic computations. However, programs for such calculations are, generally, poorly documented, and user information is difficult to obtain. Therefore, limited information could be identified for this component of Appendix B.

University of Iowa Manual:

The Iowa Institute of Hydraulic Research at the University of Iowa has developed a manual entitled "Hydrologic and Hydraulic Computations on Small Programmable Calculators," (Croley, 1977). The purpose of this manual is to provide programs for many basic, typical problems of water resources engineering in the areas of hydrology and hydraulics. The approach taken in presenting such programs is to allow the user to combine personal programming needs with previously programmed computations. Thus, personal programming time can be reduced. The manual contains a brief description of the problem and solution theory, as well as algorithms for solution for every problem considered. The reference for this manual is as follows:

1. Croley, T. E. 1977. "Hydrologic and Hydraulic Computations on Small Programmable Calculators," Iowa Institute of Hydraulic Research, University of Iowa, Iowa City, Iowa.

Storm Sewer Design Using the Rational Method:

This program was developed for a TI-59 calculator and was published at a workshop on storm sewer system design (Voorhees, 1978). A detailed flow chart showing the procedure of the program calculator operations, user interactive steps, and user decisions is presented. The program consists of 450 steps and requires 21 storage registers. Input format is listed on TI-59 program record sheets. This program is published in the following report:

1. Voorhees, M. L. 1978. "TI-59 Calculator Program for Storm Sewer Design Using Rational Method," published in "Workshop Notes on Storm Sewer Design," edited by B. C. Yen, Department of Civil Engineering, University of Illinois, Urbana, Illinois.

Hydrograph Synthesis by the HNV-SBUH Method:

This program was developed for an HP-97 or H-67 programmable calculator by B. L. Golding (1980). A step-by-step description of the basic HNV-SBUH method is presented along with user coding instructions and model validation data. This program is published in the following report:

1. Golding, B. L. 1980. "Hydrograph Synthesis by the HNV-SBUH Method Utilizing a Programmable Calculator," published in "Proceedings Stormwater Management Model (SWMM) User's Group Meeting June 19-20, 1980,' EPA-600/9-80-064, U.S. EPA Environmental Research Laboratory, Athens, Georgia.

Columbus, Georgia SCS Hydrograph and Flood Routing Programs:

The City Engineering staff of Columbus, Georgia, developed two programs for an HP 41-C. The first uses the SCS curve number approach to develop a runoff hydrograph, and the second performs a storage reservoir routing. A listing of those programs is available; however, very limited user documentation has been developed.

RESOLUTION
38-74

WHEREAS, certain areas of the City of Montgomery are subject to periodic flooding from Catoma Creek, the Alabama River, and the Tallapoosa River; causing serious damage to properties within these areas; and

WHEREAS, relief is available in the form of Federally subsidized flood insurance as authorized by the National Flood Insurance Act of 1968; and

WHEREAS, it is the intent of this Board of Commissioners to require the recognition and evaluation of flood hazards in all official actions relating to land use in areas having special flood hazards; and

WHEREAS, this body has the legal authority to adopt land use and control measures to reduce future flood losses pursuant to Title 37 of the Code of Alabama, 1940.

NOW, THEREFORE, BE IT RESOLVED, THAT This Board of Commissioners hereby:

1. Assures the Federal Insurance Administration that it will enact as necessary, and maintain in force for those areas having flood hazards, adequate land use and control measures with effective enforcement provisions consistent with the criteria set forth in Section 1910 of the National Flood Insurance Program Regulations; and

2. Vests the Planning and Development Department with the responsibility, authority, and means to:

   A. Delineate or assist the Administrator, at his request, in delineating the limits of the areas having special flood hazards on available local maps of sufficient scale to identify the location of building sites.

   B. Provide such information as the Administrator may request concerning present uses and occupancy of the flood plain.

C. Cooperate with Federal, State, and local agencies and private firms which undertake to study, survey, map, and identify flood-plain areas, and cooperate with neighboring communities with respect to management of adjoining flood plain areas in order to prevent aggravation of existing hazards.

D. Submit on the anniversary date of the community's initial eligibility an annual report to the Administrator on the progress made during the past year within the community in the development and implementation of flood plain management measures.

3. Appoints the Department of Inspections to maintain for public inspection and to furnish upon request to a record of elevations (in relation to mean sea level) to the lowest flood (including basement) of all new or substantially improved structures located in the special flood hazard areas. If the lowest floor is below grade on one or more sides, the elevation of the floor immediately above must also be recorded.

4. Agrees to take such other official action as may be reasonably necessary to carry out the objectives of the program.

RESOLUTION
39-74

WHEREAS, the City of Montgomery has adopted and is enforcing its Zoning Ordinance, Subdivision Regulations, and the Southern Standard Building Code; and

WHEREAS, Section 2, Article II of the Zoning Ordinance prohibits any person, firm, or corporation from erecting, constructing, enlarging, altering, repairing, improving, moving or demolishing any building or structure without first obtaining a separate building permit for each building or structure from the Building Department; and

WHEREAS, the Building Department must examine all plans and specifications for the proposed construction when application is made to it for a building permit.

NOW, THEREFORE, BE IT RESOLVED BY THE BOARD OF COMMISSIONERS OF THE CITY OF MONTGOMERY, as follows:

1. That the Department of Inspections shall review all building permit applications for new construction or substantial improvements to determine whether proposed building sites will be reasonably safe from flooding. If a proposed building site is in a location that has a flood hazard, any proposed new construction or substantial improvement (including prefabricated and mobile homes) must (1) be designed (or modified) and anchored to prevent flotation, collapse, or lateral movement of the structure, (2) use construction materials and utility equipment that are resistant to flood damage, and (3) use construction methods and practices that will minimize flood damage.

2. That the City Engineer shall review subdivision proposals and other proposed new developments to ensure that (1) all such proposals are consistent with the need to minimize flood damage, (2) all public utilities and facilities, such as sewer, gas, electrical, and water systems are located, elevated, and constructed to minimize or eliminate flood damage, and (3) adequate drainage is provided so as to reduce exposure to flood hazards.

3. That the Water Works and Sanitary Sewer Board shall require new or replacement water supply systems and/or sanitary sewage systems to be designed to minimize or eliminate infiltration of flood waters into the systems and discharges from the systems into flood waters, and require onsite waste disposal systems to be located so as to avoid impairment of them or contamination from them during flooding.