STORMWATER DESIGN MANUAL

MID-OHIO REGIONAL PLANNING COMMISSION. . . . . . . .COLUMBUS, OHIO

STORMWATER DESIGN MANUAL

Prepared By

Burgess & Niple, Limited
5085 Reed Road
Columbus, Ohio 43220

For

Mid-Ohio Regional Planning Commission
285 East Main Street
Columbus, Ohio 43215

June, 1977

MID-OHIO REGIONAL PLANNING COMMISSION

OFFICERS AND EXECUTIVE COMMITTEE

Mrs. Lou Briggs, Worthington, Chairman
Mr. Robert Parkinson, Columbus, Vice Chairman
Mr. Roger Higgins, Groveport, Secretary

Reverend Jacob J. Ashburn, Franklin County
David P. Barker, Franklin County
Kenneth Browning, Township Trustee
Harold Cooper, Franklin County Com.
Paul J. Falco, Marble Cliff
N. Jack Huddle, Columbus
Hal Hyrne, Upper Arlington
Gerald Kingsmore, Grove City

COMMISSION MEMBERS

Philip F. Allen, Whitehall
William H. Anderson, Franklin County
John Belt, Gahanna
Edward Bischoff, Jr., Grove City
William T. Bownas, Minerva Park
Mrs. Barbara Brandt, Bexley
John W. Breen, Gahanna
C. Ray Buck, Grandview Heights
Arthur Day, Canal Winchester
Michael Dorrian, Franklin County Com.
Richard W. Eschliman, Franklin County
Larry Ford, Worthington
Wilbur Gantz, Franklin County
Vaughn Hairston, Urbancrest
Clifford Hall, Franklin County
Hunter Hopson, Hilliard
Donald Hummer, Reynoldsburg
Delbert Johnson, Harrisburg
John Kern, Brice
Paul Lamothe, Hilliard
Mrs. Ann Larrick, Grandview Heights
David Madison, Bexley
Mrs. Rosemary Martin, Franklin County
Richard Manifold, Reynoldsburg
Gerald Mayo, Franklin County
James McGuire, Franklin County
Cletus McPherson, Franklin County Engineer
Mrs. Lois Mills, Franklin County
John Moffitt, Dublin
Oscar Needles, New Albany
Joseph Polis, Franklin County
Raymond Riley, Whitehall
Cecil G. Shirkey, Lockbourne
Lorenzo D. Sheets, Township Trustee
Blaine T. Sickles, Upper Arlington
Thomas Singell, Westerville
John Stewart, Westerville
Robert Southwick, Franklin County Com.
Richard V. Warren, Franklin County
Willis C. Welch, Franklin County
Mrs. Jane Young, Columbus

Robert J. Holland, Legal Counsel

ACKNOWLEDGMENTS

So many individuals and organizations assisted in the preparation of this manual that we cannot begin to acknowledge all of them here. If we have overlooked some it was not intentional.

Special recognition should be given to the Development Committee for Greater Columbus (DCGC) and its Special Committees who were responsible for the Mid-Ohio Regional Planning Commission (MORPC) initiating development of the Stormwater Management Policy Study. Mr. Tom Bay, former Executive Director of the DCGC, warrants special thanks.

Through the efforts of DCGC and MORPC the study was financed by special funds contributed by Bexley, Columbus, Dublin, Franklin County, Gahanna, Grandview Heights, Grove City, Groveport, Hilliard, Lockbourne, Marble Cliff, Minerva Park, Obetz, Reynoldsburg, Urbancrest, Upper Arlington, Westerville, Whitehall, and Worthington. Mr. Lou Valente, Chairman of the MORPC in 1975, supported the study and Warren Cremean, who succeeded him, appointed the Stormwater Technical Advisory Committee (STAC) in June, 1975. Current Chairperson Lou Briggs, as a member of the League of Women Voters, MORPC's Citizens Advisory Committee (CAC) and Member of the Commission, lent her support throughout the study.

The greatest commendation must go to the members of STAC who spent many long hours of dedicated work advising consultants, staff, MORPC, and its various committees concerned with the study. Those members of STAC included: Jack Bachtel (Planned Communities, Inc.); Ed Bischoff, Jr. (Grove City Engineer); Lin Carver (Department of Development, City of Columbus, representing Jack Huddle); Ted Cochran (Reliance Universal, Inc.); Ernest G. Fritsche (Ernest G. Fritsche & Co.); John Gallucci (Concrete Construction Company, Chairman of STAC); Robert Goettemoeller (Division of Soil and Water Districts, Ohio Department of Natural Resources); James R. Hanson (attorney); James Hoffman (Building Industry Association); Jack Huddle (Director, Columbus Department of Development); Harold Jett (Evans, Mechwart, Hambleton & Tilton, Inc., Vice Chairman, STAC); Cletus McPherson (Franklin County Engineer); Ralph Metzger (Building Industry Association); Robert Parkinson (Service Director, City of Columbus); Richard Rush (Soil Conservation Service); Thomas Singell (City Engineer, City of Westerville); and Richard Zande (Zande & Associates).

Other persons deserving special recognition are: Dave Norman (Ohio Department of Natural Resources) and Joseph Harrington (State Conservation Engineer, Soil Conservation Service), who gave invaluable advice and provided input to the study; Willis Welch (Commission Member and former Chairman of the MORPC Water-Related Task Force), who attended many STAC meetings; Robert Holland (legal consultant); and Frank Baum, Milton Bosworth and Robert Wolfe (engineering consultants, Burgess & Niple Ltd.).

MEMORANDUM

TO: MORPC Stormwater Design Manual Owners

FROM: Steve A. Snyder, P.E. - MORPC Staff Engineer

DATE: November 20, 1986

RE: Clarifications to 1977 Stormwater Design Manual

- This office continues to receive inquiries concerning the availability of the Stormwater Design Manual (SDM) and requests for interpretations or explanations of its use.

- Questions seem to relate to several areas of concern which are not clearly presented in the SDM. The most frequent requests for information deal with storm sewer design frequency and the SCS TR-55 procedures.

- Inquiries as to how various agencies, municipalities or political subdivisions may be using or interpreting the SDM can only be addressed by the individual entities themselves.

- With this in mind, the following items are submitted for your use and consideration in utilizing the SDM:

Delete pages 23-28, 35-52, 80, 169 and 170 which attempt to cover the 1975 version of SCS's Technical Release Number 55 (TR-55). The TR-55 document itself is complex and this presentation changes all the table and chart numbers and fails to cover all aspects of the calculations. The State of Ohio SCS office subsequently published, in 1981, an Ohio supplement to the Federal SCS publication which addresses TR-55 more accurately and in more detail than the SDM. Also, the long awaited Federal revision to the the 1975 version has just been completed this year. As this topic is covered in more detail and more accurately in its recently released version (1986), it is no longer practical or desirable to continue distributing the incomplete and outdated 1977 version appearing in the SDM.

The following should be inserted into section 3.2.1. page 12 after the 1st sentence of the 1st paragraph and section 7.3.1. page 77 after the 2nd sentence of the 2nd paragraph: "Typically, this is accomplished by using Manning's Equation with a 2 year storm (flowing-full conditions) checking to ensure the 5 year Hydraulic Grade Line will also be contained within the system (pressurized flow)."

Members of the MORPC staff who should be recognized are: William C. Habig, Executive Director, Robert S. Winterhalter, Deputy Director, Robert K. Whittier, Public Information Administrator, Harmon T. Merwin, Program Manager, Water-Related Program, Candace Hughes, Assistant Public Information Administrator, Robert Wood, Research Analyst, Birnie Wetherell and Reva Spiert, secretaries.

Special notice is accorded Stuart "Sonny" Cohen who was employed specifically to coordinate and do a major portion of the work. His perseverance and innovation provided impetus toward the overall success of the study.

CONTENTS

FIGURES

TABLES

EXHIBITS

1. INTRODUCTION

1.1. Purpose

An inescapable law of drainage design dictates that water drained from one area must be drained to another. No magic can make water disappear. To get water off streets, away from doorsteps, and keep it out of basements, alternative places must be found.

Deciding how to move water is an engineering decision requiring substantial skills in hydrology and hydraulics. Deciding where to move water is a public decision reflecting public desires and an awareness of the cost of alternative solutions.

Considerable discussion has been held within the Mid-Ohio Region to address stormwater management. Meetings were held with public officials, community groups, developers, and engineers. In addition, a technical committee representing diverse public and private interests has studied this matter in depth.

From information obtained from these participants and other research, it is evident that storm drainage problems are most often associated with increases in the rate and volume of runoff resulting from changing land use. These problems include overloading and backing up of storm sewers, stream flooding, and channel erosion. The consensus reached is that the release of storm runoff into the drainage system must be controlled. Responsibility for providing control rests with those whose actions increase the runoff. The Stormwater Design Manual provides engineering tools for developing control policies.

The drainage design manual is not a text of hydrology or hydraulic design. It assumes the user has an understanding of hydrology and hydraulic engineering. It does not provide uniform solutions to all drainage problems. Stormwater system design presents an opportunity for the creative and innovative design engineer. The engineer should not be restricted to standardized designs or procedures. Nor should the responsible reviewing authority insist on rigid adherence to a standard set of design specifications. As reflected in this manual, the emphasis should be on performance.

This manual provides a uniform design procedure and worksheets for summarizing and submitting the design plans in an acceptable and understandable manner to the various reviewing and approving agencies. While the designer is not restricted to these recommended procedures or worksheets, sufficient documentation must be provided with any submission to insure that methods, procedures, and data are clear.

1.2. Scope

This design manual provides sufficient information to develop drainage systems in accord with local policy. For a design engineer, such systems begin with the first drop of rainfall and end when the water is safely discharged to receiving waters of adequate capacity.

Drainage is only one part of a complex urban system. Drainage considerations do not have to dominate site development decisions. Yet drainage does have its place on the site planner's checklist. Chapter 2 outlines how the extent to which drainage is considered, planned, and integrated into development plans and determines the successful functioning of the drainage sub-system as well as its costs.

Controlled release of stormwater runoff is the fundamental policy in the design manual. Chapter 3 offers a specific statement of this policy. Standards for achieving a controlled release rate are detailed here.

Rainfall is the first design element to be considered. This phenomenon is basic to the design of stormwater facilities. Chapter 4 outlines the proper use of rainfall information and appropriate data sources relevant to Franklin County.

The behavior of rainfall on the ground, when it becomes runoff, is responsive to a number of variables. Watershed area and shape, ground slope, soils, seasons, and impervious areas determine the characteristics of runoff. Chapter 5 discusses these critical elements and shows how each may be used in different mathematical models to provide appropriate design information.

Streets are important collecting units for drainage. Large concentrations of surface runoff first appear on streets. As a multi-use facility, streets must be designed carefully to preserve their transportation function yet satisfy surface drainage needs. Maximizing the drainage function of streets within clearly defined and publicly acceptable performance standards presents an excellent opportunity for cost-efficiency in new developments. Chapter 6 outlines these standards.

Subterranean flow of water through storm sewers and culverts is discussed in Chapters 7 and 8. These chapters provide a checklist of design factors which engineers should evaluate. Sample calculations are used to illustrate application of the recommended standards and procedures to hypothetical design situations.

Transmission of stormwater runoff through open channels may be desirable for economic and/or aesthetic reasons. Open channel design presents many opportunities but also has the potential for creating hazards and maintenance problems. Chapter 9 examines the use of open channels and their roles in the urban environment. Criteria are proposed to determine the appropriateness of open channel flow. Design standards are proposed to minimize hazards and assure the physical integrity of channels.

Often temporary storage of stormwater through controlled release is required to meet runoff control policies. Stormwater runoff storage may be accomplished in many ways. Chapter 10 identifies a range of methods available for these needs, outlining positive and negative aspects of each. While different developments require different strategies and unique design, standards have been established to assure that facilities will serve their drainage functions without creating nuisances.

A recommended set of designer's worksheets is provided in Chapter 11. These were prepared in response to many requests from plan review authorities to standardize type and format of data submitted with drainage designs.

Charts for determining the pressure changes of storm sewer junctions as well as design procedures for both open channel and pressurized conduit design are contained in Appendix A.

The glossary presented in Appendix B acknowledges that nomenclature and jargon in the field of stormwater management often is incomprehensible to lay citizens, and frequently confuses and prevents communication among engineers. Until there is greater standardization of many terms, this glossary will help to bridge gaps.

1.3. Background

The Stormwater Design Manual is a product of the Stormwater Management Policy Study conducted by the Mid-Ohio Regional Planning Commission from Spring, 1975 until Spring, 1977. The Study was sponsored by the Franklin County government and local governmental units in the County.

The program brought together developers, engineers, planners, attorneys, and public officials to examine drainage problems, develop alternatives and find acceptable solutions. Discussions were held with city councils, planning commissions, citizens and special interest groups to engage them in the policy-making process.

Legal counsel and engineering consultants were employed to prepare alternative administrative structures, funding mechanisms, and technical design standards to make the proposed policies function. Recommendations on finance and administration have been printed in companion reports to this design manual.

2. PLANNING FOR STORMWATER MANAGEMENT

2.1 The Natural System

Nature has carved an effective and functional drainage system from the unique topographic features of Franklin County. The Scioto River, its tributary rivers, and creeks and streams which flow into these are the more apparent parts of this drainage system. Less obvious are the shallow gullies and sloping, rolling land features which collect, concentrate, and direct storm runoff to the larger watercourses. The size and capacity of these watercourses and gullies is related to the amount of rainfall and snow melt which is typical of the area and to the characteristics of the soil.

2.1.1 Urbanization Impact on the Natural System

Change in the surface of the land is one feature associated with urbanization. To accommodate structures, rolling land is graded to eliminate the high areas and fill the low areas. Buildings, streets and parking lots replace meadows and forest land. Changes like these cause the storm runoff to behave differently. Where formerly water would soak into the earth or run off slowly, the impenetrable surface of the roof or parking lot causes more water to runoff and to flow at a faster rate.

The increased water and rate of flow places a stress on the existing natural, drainage system. Because the system does not have the necessary capacity for the demand placed on it, the system will flood its banks. Backups occur and water remains standing in the street or finds alternate paths into the basements of homes. Over a longer period of time, watercourses adjust to the change in flow. This can be observed as stream banks begin to erode, and sediment is carried downstream and deposited in the stream or in man-made reservoirs. Frequently this sediment will collect in one area, restricting channel or sewer capacity, and causing water to back up behind it.

Unfortunately, the failure of urban drainage systems to function properly often affects those who have very little to do with the cause of the problem. Buildings located near a small ditch may be destroyed as the ditch enlarges to accommodate increased flow. Water backing up in storm sewers may flood basements because the outlet of the storm sewer is blocked with sediment from a nearby construction project.

2.1.2 Impact of the Natural System on Urbanization

Nature has the capacity to bend and yield to stresses placed on it. But nature may also strike back with unexpected fury. Storm runoff will occur no matter how well or poorly the drainage system is planned. An infrequent, intense storm may dramatically illustrate how inadequate a drainage system is.

The only law of flowing water is that it will seek the lowest level. Water will not stay within its banks or flow along a prescribed course unless that course respects the law of flowing water and has adequate capacity to carry the volume of water. Homes or businesses constructed without regard to drainageways will flood as surely as if the structures were built in the path of the Scioto River.

Urban designers and developers responsible for site layout should think in terms of natural drainage easements and street drainage patterns. Site layout should be coordinated with drainage engineers. Planning of urban drainage facilities should incorporate natural waterways, artificial channels, storm sewers, runoff storage facilities and other drainage works into the development of an urban community. Attempting to superimpose drainage works on a development after it is laid out as is often done with water supply and sanitary sewer facilities cannot be nearly as effective. And the cost of corrective drainage and flood control measures more than offsets any savings which can be achieved through lack of drainage planning. Good planning results in lower cost drainage facilities and a better community.

2.2 Planning for Urban Drainage

The development of an urban drainage plan requires the consideration of four drainage elements. These are initial drainage, major drainage, stormwater storage, and erosion controls.

Planning and design must consider the regular, frequently occurring storm; that is, the initial storm, and the less frequent but more extensive major storm occurrence. Planning for storage is essential to insure water will go where it will not create a problem. Erosion controls must be considered before the earth is disturbed and significant losses and damage occur.

2.2.1 The Initial Drainage System

The initial drainage system is for collecting and transporting storm runoff and snow melt from frequently occurring storms. The initial system includes street curbs and gutters, underground storm sewer pipes, and manholes, open drainageways, culverts, and small open channels. Its purpose is to eliminate inconveniences associated with runoff and to prevent health hazards associated with low areas where water might ordinarily stand. This portion of the urban drainage system has received the most attention from engineers and is what most citizens consider, incorrectly, to be the total urban drainage system. It is what directly contributes to orderly community development by handling, without nuisance, the flow of frequent storm runoff.

Early planning can do more than provide a functional drainage system. The preliminary layout of the system has more effect on the cost of the storm sewers than the final hydraulic design, preparation of the specifications, and choice of materials. The ideal time to undertake the layout of the storm sewers is prior to finalization of street layout in a new development. Once the street layout is set, the options open to the drainage engineer are greatly reduced.

Streets serve an important and necessary drainage service, even though their primary function is for the movement of traffic. Traffic and drainage uses are compatible up to the point at which drainage must be subservient to traffic needs. Gutter flow in streets is necessary to transport runoff to storm inlets. Good planning of streets can help reduce the size and length of a storm sewer system. The longer street flow can be kept from concentrating in a street, the further the distance from a ridge line the storm sewer system can begin. This is significant because a larger part of storm sewer construction cost is represented by small diameter laterals. Various layout concepts should be developed, reviewed, and analyses made to arrive at the best layouts.

Design standards for the collection and conveyance of runoff water on public streets is based on an acceptable frequency of traffic interference. That is, depending on the character of the street, and the intensity of the rain, a certain amount of stormwater encroachment into the traffic lanes will be permitted. These standards are more specifically explained in Chapter 6 (Streets and Inlets). Additional standards for initial drainage are discussed in Chapter 7 (Storm Sewers), Chapter 8 (Culverts and Miscellaneous Structures), and Chapter 9 (Open Channels).

2.2.2 Major Drainage

It is not economically feasible to size a storm sewer system to collect and convey more than the frequent storm runoff. However, runoff which exceeds the capacity of the storm sewer system must have a route to follow. Essentially, the complete drainage system of an urban area contains two separate drainage elements. While the storm sewers belong to the initial system, surface drainageways must be provided for the major flow from more intense storms.

The intent of planning for the major drainage element is to insure stormwater runoff which exceeds the capacity of the initial drainage system has a route to follow which will not cause a major loss of property or any loss of life. It should be remembered the major drainage system exists even when it is not planned and whether or not development exists wisely in respect to it. To get in its way is imprudent and costly.

Street rights-of-way are a common choice for conveying major drainage flows. Again, such use must be anticipated when the street layout is established. Side and rear lot lines offer one alternative to the street. The problem with this alternate is the possibility individual property owners may usurp the major drainage easement. Rarely is the problem recognized until the infrequent rainstorm occurs and the major system fails to operate properly.

One planning rule of thumb which seems to be effective in maintaining the integrity of non-street drainage rights-of-way is to design them for multi-purpose functions. Pedestrian and bicycle paths lend themselves naturally to this application. Linear parks aligned along the major drainage corridor are also very effective, but usually require greater width than would normally be necessary for drainage purposes.

2.2.3 Storage

The emphasis of policy in this design manual is to control the increases of runoff resulting from development with various storage mechanisms. While considerable storage can be achieved within channels and storm sewers, on lawns and natural surface depressions, it is likely special storage facilities, either single or multi-purpose, will have to be established for new developments. Like the rest of the drainage system, both the location and type of storage facilities should be determined as part of the site plan.

Park land presents an excellent opportunity for the temporary detention of runoff from adjacent areas. In many cases, the use of park land for this purpose allows storm drainage, which is often considered both a nuisance and a hazard, to be used productively in permanent ponds. Such detention storage areas may be established as an integral part of the open space of a single development or, with the cooperation of the municipal government, may be included in a more centrally located area serving several developments. Advanced planning and contact with the recreation department or committee in the community provides the greatest chance for success. School districts, also, may be interested in acquiring open space as a natural preserve or for athletic facilities.

Alternative storage procedures should be explored and evaluated for their appropriateness within different developments. Parking lots, rooftop storage, permanent pools, infiltration trenches, and other procedures are discussed in greater detail in Chapter 10 (Runoff Control Methods). The greatest chance for success can be achieved if storage is considered at the earliest stages of site planning.

2.2.4 Erosion Control

Erosion is a natural process and zero erosion is an unrealistic goal. However, accelerated erosion which occurs at the time of development when land surfaces are cleared of vegetation can create costly problems. Silted ponds, lakes, and reservoirs have less room to store stormwater; water supplies may be damaged and flood hazards increased. In the Mid-Ohio Region, the reservoirs formed by Griggs Dam, Hoover Dam, O'Shaughnessy Dam, and Alum Creek Dam may all be affected by accelerated erosion.

Closer to home, accelerated erosion may undermine and weaken foundations of buildings. Once deposited in streams and storm sewers, sediment can block the flow of water causing upstream flooding and even forcing streams to cut new channels in unplanned locations.

Erosion control should be thought of not only as damage prevention but also preservation of an economically valuable commodity. Top soil which is properly removed and stored prior to excavations can provide savings in the cost of finishing and landscaping new developments.

Erosion control must be programmed into the development process. It must be considered as part of the land disturbance activity. Too often these activities are initiated as part of the finishing process. Such action is tantamount to closing the barn door after the cows have gone. This adds a cost to development without a commensurate increase in benefit.

3. STORMWATER RUNOFF POLICY

3.1 Control of Stormwater Runoff

This design manual is premised on the policy that land uses and developments which increase the runoff rate or volume shall be required to control the discharge rate of runoff prior to its release to off-site land. The purposes of this policy are to:

1. Permit development without increasing the flooding of other lands.

2. Reduce damage to receiving streams and impairment of their capacity which may be caused by increases in the quantity and rate of water discharged.

3. Establish a basis for design of a storm drainage system on lands below undeveloped areas which will preserve the rights and options of both dominant and servient property owners and assure the long-term adequacy of storm drainage systems.

This runoff control policy applies to all land developments not specifically exempted by 3.1.2 or granted a waiver as provided by 3.1.3. Other sections of this Chapter specify the performance requirements of on-site drainage system and runoff control standards.

3.1.2 Exemptions

Exemptions are appropriate for certain land use activities which clearly do not generate significant increases in stormwater runoff. Exemptions shall refer to the requirement for runoff controls only and does not in any way imply a relaxation of the requirement for adequate on-site drainage or the ability of the system to accept runoff from tributary land. The following land uses and developments are within this exemption category:

1. Land preparation for agricultural crops, orchards, woodlots, sod farms and nursery operations.

2. Land grading or leveling for erosion control under direction of the local soil conservation district.

3. Land subdivisions for residential purposes with minimum lot size of five acres or more.

3.1.3 Waivers

It is conceivable that development situations, not automatically subject to exemption by 3.1.2, may exist such that development will have none of the harmful effects associated with increases in runoff rates and volume. Such developments are eligible for a waiver. The waiver applies only to the requirement that runoff be controlled, and does not in any way imply a relaxation in the requirement for adequate on-site drainage or the ability to accept runoff from land tributary to the development.

The waiver application must request in writing that the requirements for stormwater runoff control be waived. The application shall include sufficient detail to determine that granting a waiver will not result in increased flooding and that the added volume of runoff will not damage the receiving stream.

A condition of the waiver shall be that any addition, extension or modification of a development for which a waiver has been granted shall be required to provide stormwater runoff control for the entire site if preceding limitations are exceeded by subsequent additions, extensions or modifications.

Development activities for which waivers might ordinarily be considered include the following:

1. Single family residential developments:

Minimum Lot Size	Maximum Subdivision Size
1 acre	10 acres
½ acre	5 acres
15,000 square feet	2 acres

2. Multi-family residential developments which total two acres or less.

3. Buildings, their related parking lots, and structures where less than two acres are to be altered by grading, draining, removing existing ground cover or paving; and of which one acre or less will be impervious areas, such as roofs, walks, and parking areas.

4. Situations where existing and adequate off-site stormwater runoff control facilities provide the required control. However, this shall not be construed to imply the first development requesting use of existing storage capacity shall have full use of available capacity. Rather, such waiver may grant a proportional use of available storage capacity to insure that later developments have a similar opportunity to utilize a portion of the storage capacity.

3.1.4 Stormwater Runoff Control Criteria

Stormwater runoff control addresses both peak rate and total volume of runoff.

1. The peak rate of runoff from an area after development shall not exceed the peak rate of runoff from the same area before development for all storms up to a 100-year frequency, 24-hour storm.

In addition, if it is found a proposed development will increase the volume of runoff from an area, the peak rate of runoff from certain more frequent storms must be controlled further.

There are two reasons why increases in volume of runoff require a control standard more restrictive than controlling to the predevelopment condition. First, increases in volume mean runoff will be flowing for a longer period of time. When routed through a watershed, these longer flows may join at some point or points downstream thereby creating new peak flows and the problems associated with peak flow (flooding). This is known as the "Routing Problem." Second, longerflowperiods of large runoff quantities place a highly erosive stress on natural channels. This stress can be minimized by reducing the rate of discharge. The permissible peak rate shall be determined as follows:

2. Determine the total volume of runoff from a 1-year frequency 24-hour storm, occurring over the area before and after development.

3. Determine the percent of increase in volume due to development and using this percentage, pick the critical storm from the following table:

If the percentage of increase in volume of runoff is equal to or greater than	and less than	The critical storm for discharge limitation will be
-	10	1 year
10	20	2 years
20	50	5 years
50	100	10 years
100	250	25 years
250	500	50 years
500	-	100 years

4. The peak rate of runoff from the critical storm occurring over the development shall not exceed the peak rate of runoff from a 1-year frequency storm occurring over the same area under predevelopment conditions. Storms of less frequent occurrence (longer return period) than the critical storm, shall have peak rate of runoff not greater than for the same storm under predevelopment conditions. As an example if the total volume is shown to be increased by 35%, the critical storm is a 5-year storm. The peak rate of runoff for all storms up to this intensity shall be controlled so as not to exceed the peak rate of runoff from a 1-year frequency storm under predevelopment conditions in the area. The runoff from a more intense storm need only be controlled so as not to exceed the predevelopment peak rate from the same frequency of storm.

5. Storage volume does not have to be provided for off-site upstream streams. Flow from such areas will be routed through the drainage system in the development under consideration at a rate determined in the same manner as the on-site system. Off-site land uses prevailing at the time of development shall be considered as the predevelopment condition for the purpose of calculating changes in runoff.

3.2 Stormwater System Design Criteria

3.2.1 Design Storms

The initial drainage system is that part of the storm drainage system which is used regularly for collecting, transporting, and disposing of storm runoff, snow melt, and miscellaneous minor flows. The capacity of the initial drainage system should be equal to the maximum rate of runoff expected from a design storm of established frequency. For purposes of design in the Mid-Ohio Region, it has been determined the initial drainage portion of the drainage system shall be designed to carry the runoff from a storm with a return period of not less than five years.

The major drainage system is that part of the storm drainage system which carries the runoff which exceeds the capacity of the initial drainage system. The major drainage system shall have the capacity to carry runoff from a storm with a return period of not less than 100 years without posing significant threat to property or public safety.

3.2.2 Initial Storm: Physical Design Criteria for On-site Improvements

Depth of flow in natural channels shall not exceed bank full stage, backwater effects considered.

Depth of flow in artificial channels shall not exceed 0.8 bank full stage. Velocity of flow shall be determined in accordance with the design criteria for open channels (Chapter 9) and shall not exceed 7 feet per second. Where flows do exceed this rate, special channel lining and erosion protection shall be provided.

Depth of flow in road side ditch swales shall not exceed one foot or be of such depth that flow would extend out of the right-of-way if the side ditch is less than one foot in depth. Velocity at this depth shall not exceed six feet per second with grass swales or ten feet per second with paved ditches.

Depth of flow in streets with curb and gutter shall not exceed the curb height. Velocity of flow in the gutter at design depth shall not exceed ten feet per second. In addition to the above, the following are maximum encroachments of the initial design storm (the five-year storm) onto the pavement:

1. For local streets carrying traffic from the individual residence to sub-collectors or collectors and to minor cross streets, the flow may spread to the crown of the street.

2. For sub-collector or collector streets, one lane shall be free from water.

3. For arterial streets, one lane of traffic in each direction shall be free from water.

4. For freeways, no encroachment is allowed on traffic lanes.

In design of conduit, the conduit shall be designed on the basis of flowing full with surcharge to gutter line. Backwater effects shall be considered.

3.2.3 Major Storm: Physical Design Criteria for On-Site Improvements

The major storm floodway and floodway fringe for natural streams shall be as defined by the U.S. Army Corps of Engineers, the U.S. Department of Housing and Urban Development, or the Ohio Department of Natural Resources, where such determinations have been made.

Many of the drainageways associated with the major storm system are in areas beyond those designated as floodway or floodway fringe. For these areas, the major storm flood limits shall be determined by the U.S. Corps of Engineers HEC-2 method or other accepted methods of determining water profiles using the major design storm runoff. One-half foot elevation shall be added to the flood profile as freeboard for protection in the event of future encroachments into the floodway fringe or the drainageway.

Where the street is designated as the major drainageway, the depth of flow shall not exceed 18" at gutter line for local and collector streets and shall not exceed 6" depth at crown for arterial streets and freeways. The same maximum depth criteria will apply where a major drainageway crosses the street. Where a major drainageway is located outside a street right-of-way easements,shall be provided.

In determining the required capacity of surface channels and other drainageways provided for the major storm runoff, the street storm inlets and conduit provided for the initial design storm shall be assumed to be carrying not more than one-half their design capacity. This is a safety factor to allow for surcharged outlets, obstructed inlets or other malfunctions.

4. RAINFALL

4.1. Introduction

Rainfall information is basic to design of stormwater facilities. Rainfall is a natural event and precise projections of its frequency and intensity cannot be made. However, useful information can be obtained by analysis of past storms. Reasonable predictions of the frequency of occurrence (recurrence interval), the duration, the total amount, the distribution of the amount with respect to area, the distribution of the intensity with respect to time and the seasonal probability of an occurrence can be made. The United States Department of Commerce, through the National Oceanic and Atmospheric Administration and its predecessor, the Weather Bureau, have gathered and compiled hourly precipitation data in Columbus for many years and from time to time have published it in various forms.

4.2. Rainfall Intensity-Duration-Frequency

The most familiar presentation of rainfall data is a set of curves representing different frequencies of occurrence of rainfall events with the intensity of the rainfall plotted against its duration. The City of Columbus has prepared a set of intensity-duration-frequency curves for occurrences up to 53-year return periods. The United States Department of Commerce has prepared such curves for Columbus and 202 other U.S. cities and have published them in Technical Paper No. 25 (See Bibliography, Section 4.5). These curves are based on an analysis of the annual series, using the maximum value of storm for each year. They also have published in Technical Paper No. 40 a series of maps showing equal intensities of rainfall for various durations and frequencies of occurrences. These isopluvial maps are based on the partial-duration series, utilizing all high values of rainfall rather than the high value per year. The partial-duration series results in curves showing slightly higher intensities of rainfall for the more frequent recurrence interval. A series of intensity-duration-frequency curves for rainfall in Franklin County have been derived from the data in Technical Paper No. 40 and is shown in Exhibit IV-1.

4.3. Rainfall Distribution by Time

Rainfall intensity-duration-frequency curves give a value of the intensity of rainfall during a particular time interval but do not necessarily present the sequence of events during a storm. The rainfall event causing stormwater runoff from watershed for which this manual is intended will most probably be of the nature of a thunderstorm or frontal storm. These storms are characterized by a period of rainfall of gradually increasing intensity with a comparatively short intense mid-storm followed by additional rainfall of gradually decreasing intensity. Each storm is a separate distinct event. However, with enough of a common pattern that an intensity distribution with time can be developed for use where such information is required to estimate stormwater runoff, such as in development of hydrographs. The Soil Conservation Service of the U.S. Department of Agriculture has developed several typical storm distributions. Their Type II Storm is most suitable for rainfall of the summer thunderstorm type as occurs in Ohio and similar climatic regions of the United States. This distribution is based on the 24-hour rainfall for a given recurrence interval with the accumulation to a given hour shown as a ratio to the total 24-hour rainfall amount. This is shown in Table 4-1.

The 24-hour total rainfall for Franklin County for use with this table is as follows:

Recurrence Interval	24-Hour Rainfall
Years	Total Inches
1	2.35
2	2.55
5	3.30
10	3.80
25	4.30
50	4.75
100	5.00

Although the distribution is based on 24-hour duration of storm, almost 64% of the rainfall occurs in the 4-hour period between the 10th and 14th hour and 27.6% in the 15-minute interval preceding the 12th hour.

4.4 Rainfall Distribution by Area

The data given in the tables and exhibits is for point rainfalls but should be valid for areas up to 10 square miles. For larger watersheds, a correction reflecting a lesser intensity over the greater area should be applied. This correction factor is given in Technical Paper No. 40 and numerous other publications.

The data also is generalized for Franklin County and does not reflect any variation in intensity. The topography of Franklin County is level enough to avoid irregular distribution due to orographic effects and at present there is insufficient information available to assign an urban effect adjustment.

TABLE 4-1

TIME DISTRIBUTION OF RAINFALL
ACCUMULATION OF RAINFALL TO 24 HOURS
AS RATIO TO TOTAL FOR SOIL CONSERVATION SERVICE TYPE II STORM

TIME (HOURS)	RATIO
0	0
2.0	0.022
4.0	0.048
6.0	0.080
8.0	0.120
9.0	0.147
9.5	0.163
10.0	0.181
10.5	0.204
11.0	0.235
11.5	0.283
11.75	0.387
12.0	0.663
12.5	0.735
13.0	0.772
13.5	0.799
14.0	0.820
16.0	0.880
20.0	0.952
24.0	1.000

4.5. Bibliography

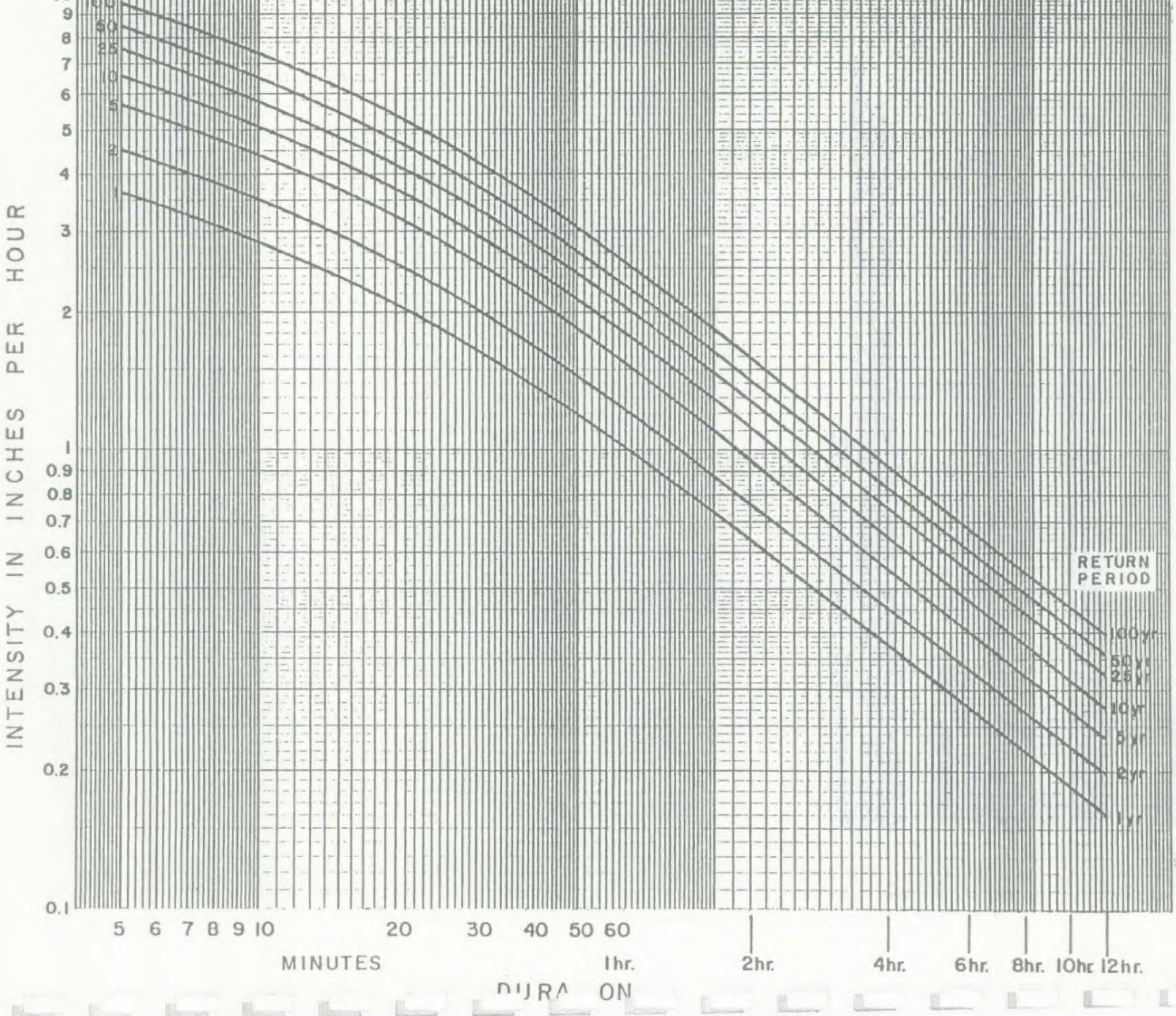
1. Rainfall Frequency Atlas of the United States, Technical Paper No. 40, Weather Bureau, U.S. Department of Commerce, Washington, D.C., May, 1961.

2. Rainfall Intensity-Duration-Frequency Curves, Technical Paper No. 25, Weather Bureau, U.S. Department of Commerce, Washington, D.C., December, 1955.

3. A Method for Estimating Volume and Rate of Runoff in Small Watersheds, SCS - TP - 149, Soil Conservation Service, U.S. Department of Agriculture, Washington, D.C., Revised April, 1973.

EXHIBIT IV-1

RAINFALL INTENSITY - DURATION FREQUENCY CURVES



Source: U.S. Weather Service, T.P. Nº 40

5. STORMWATER RUNOFF

5.1 Introduction

Calculating the amount of stormwater runoff is fundamental to the design of the storm drainage system. The amount of stormwater runoff depends on a great number of factors. Some of these factors are reasonably fixed and subject to accurate determination, such as watershed area and shape, ground slope, and natural ponding. Others are seasonally variable, such as frozen soil, soil moisture condition, evaporation or transpiration. Still others vary by land use, such as type of ground cover and impervious area, or method of cultivation. Finally, rainfall is extremely variable and subject only to laws which govern natural occurrence. Despite the indeterminate nature of these factors, methods for obtaining useful information about stormwater runoff have been developed. The methods vary from the empirical Talbot Formula to deterministic models of the watershed, such as the Environmental Protection Agency's (EPA) Storm Water Management Model (SWMM) which requires a great amount of data about the physical condition of the watershed. This Chapter presents several acceptable runoff calculation methods. The design engineer will want to select the appropriate method depending on the information needed and the size of the area under study. This selection is outlined in Exhibit V-1.

5.2 Rational Method

The Rational Method may be used to determine peak rate of runoff from areas generally not larger than 200 acres. The other methods discussed later in this chapter may also be used to obtain peak rate of runoff.

The basic formula for the Rational Method is $Q=CiA$ in which:

Q = Peak rate of runoff in cubic feet per second.

C = Runoff coefficient, an empirical coefficient representing a relationship between rainfall and runoff.

i = Average intensity of rainfall in inches per hour for the time of concentration (T_C) for a selected frequency of occurrence or return period.

T_C = Time of concentration, the estimated time required for runoff to flow from the most remote part of the area under consideration to the point under consideration. It consists of the total of time for overland sheet flow, open channel flow and pipe flow.

A = Area drained in acres.

5.2.1 Runoff Coefficient

The table of runoff coefficients, Exhibit V-2, presents average values for use with the Rational Method. This tabulation relates the coefficient to land use and to the hydrologic group of the soil. Because runoff from undeveloped land is important for establishing the pre-development runoff conditions, more values for such land are presented than in the usual compilation of such data. Further information on use of the land use descriptors and hydrologic soil groupings may be found in Chapters 8 and 9 of Section 4, "Hydrology" of the Soil Conservation Service National Engineering Handbook.

Soils have been divided by infiltration rate characteristics into four hydrologic soil groups. These groups are:

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

A list of Franklin County soils and their hydrologic classification is shown in Exhibit V-3. Soil maps showing soil series for different areas may be obtained from Franklin County Soil Conservation Service Office.

Because the mixing and compacting of soil during development generally tends to decrease its infiltration rate, an appreciation of soil mechanics is essential. Note also that the coefficients are applicable to storms of not greater than 10-year return frequency. For storms of a greater return period, the runoff from the additional rainfall would be equivalent to runoff from a paved area. As an example, given 10 residential acres, developed into one-quarter acre lots with a hydrological soil group C and a time of concentration of 30 minutes, the runoff coefficient would be 0.54. The rainfall intensity for a 10-year return period would be 2.9 inches per hour, for a 25-year return period would be a 3.4 inches per hour or 0.5 inches per hour additional. The runoff would be $0.54 \times 2.9 \times 10 + 0.96 \times 0.5 \times 10 = 20.46$ cubic feet per second.

The intensity of rainfall in inches per hour for various return periods and duration of rain (Exhibit IV-1) is derived from data given in U.S. Weather Bureau, Technical Paper No. 40. The data is for a point storm but will be applicable to all areas for which the Rational Method may be used.

5.2.2 Time of Concentration

The time of concentration is a combination of time of overland or sheet flow from the point of rainfall to an established drainage channel such as gutter, open ditch or inlet and time of travel in the drainage course. The first part, sometimes referred to as inlet time, may be estimated from the overland flow time chart (Exhibit V-4). The minimum inlet time used shall be ten minutes. The time of flow in the open channel portion may be estimated by making a preliminary estimate of the quantities of flow and applying the Manning Formula for flow in open channel. If in a gutter, the velocity may be derived from charts for gutter capacity, if in a ditch or natural channel, the shape must be known or measured. The value of "n", the coefficient of friction, will depend on the judgment of the designer but should be justified by a description of the channel. Values of "n" for various channels are listed in Exhibit IX-2. Velocity in a sewer may be estimated based on a full-flowing pipe carrying the estimated quantity. The time of flow should be adjusted if the calculated quantity of runoff obtained on the first computation differs greatly from the estimated quantity.

In applying the Rational Method, the drainage area should be delineated on a topographic map, then divided into sub-areas for analysis. Each sub-area would represent a point of contribution to a definite drainage course. If the runoff coefficient varies over a sub-area, a composite coefficient can be calculated as an average, weighted by area of the various runoff coefficients. An example, of the Rational Method technique is included in Chapter 7 (Storm Sewers).

5.3 Hydrograph Methods

For areas larger than 200 acres or where it is necessary to know the volume of water discharged in addition to the peak rate of discharge, the Rational Method is not adequate. Appropriate calculation techniques include one of the various hydrograph methods or a deterministicmodel such as the Environmental Protection Agency (EPA) Storm Water Management Model (SWMM) or the STORM Program developed by Water Resource Engineers.

The SWMM and STORM models are quite sophisticated tools for use in urban stormwater problems, both in analysis and control. They also can provide information on water quality as well as quantity. The models require a great amount of accurate input data for accurate results. They are valuable as tools for operation of an existing stormwater system where real time data is available and real time decisions are required for day to day management.

In designing stormwater systems for developing areas, there is usually insufficient information about future conditions for a meaningful use of a complex model. There are numerous adaptions of the hydrograph method available which present a reasonable compromise considering data required, complexity of computations and information obtained.

A hydrograph is a plot or tabulation of rate of stormwater runoff against time for a given watershed. A unit hydrograph is the runoff hydrograph from one inch of rainfall over the drainage area in a specified time. Standard unit hydrographs are derived from analysis of historical records correlated with watershed characteristics. Hydrographs from a given storm can then be obtained by scaling the height to the design rainfall and the base length to the duration of resulting runoff. Of various unit hydrograph methods, the one especially appropriate in addressing problems relating to changes in stormwater runoff due to changes in land use is outlined in "Urban Hydrology for Small Watersheds", Technical Release (TR) No. 55, Engineering Division, Soil Conservation Service (SCS), U.S. Department of Agriculture. This method is presented for use with this design manual.

The methodology of the S.C.S. explained in Technical Release No. 55 can be used to provide peak rates of runoff, tabular values of rate of runoff versus time from which a hydrograph can be plotted, estimated total volume of runoff or estimated volume of storage required to reduce rate of runoff to a desired value. The basic information required is:

1. Area of the watershed
2. Soil type, as for the Rational Method
3. Land use, as for the Rational Method
4. Time of concentration, as for the Rational Method
5. Travel time - Time of flow from the point of collection of the water in the design watershed to a point of evaluation or design. This is required in combining hydrographs from more than one watershed or where channel routing is considered.
6. Watershed factors:

    Slope
    Flow length
    Natural ponding

7. Design Storm based on 24-hour duration rainfall for design recurrence interval.

5.3.1 Peak Discharge Method

Where peak rate of flow and total volume of runoff are needed, the Peak Discharge Method of Chapter 4 of S.C.S., TR55 may be used.

5.4 Bibliography

1. Design and Construction of Sanitary and Storm Sewers, American Society of Civil Engineers - Manuals and Reports on Engineering Practice - No. 37 (Water Pollution Control Federation Manual of Practice No. 9), New York, 1970.

2. Hydrology Handbook, Manual of Engineering Practice No. 28, American Society of Civil Engineers, New York, 1949.

3. Section 4, Hydrology, Soil Conservation Service National Engineering Handbook, U.S. Department of Agriculture, U.S. Government Printing Office, Washington, D.C., January, 1971.

4. Franklin County, Ohio, Soil Survey Legend, Approved Symbols, Watershed Division, Franklin County Soil Conservation Service District Office, January, 1975.

5. Airport Drainage, Department of Transportation, Federal Aviation Administration, Washington, D.C., 1965.

6. Urban Hydrology for Small Watersheds, Technical Release No. 55, Soil Conservation Service Engineering Division, U.S. Department of Agriculture, January, 1975.

7. Tabular Method of Flood Routing - 24 Hour Type II Storm Distribution, TSC Technical Note - Engineering - UD-20, Soil Conservation Service Regional Technical Service Center, U.S. Department of Agriculture, Upper Darby, Pennsylvania, January, 1972.

8. Handbook of Applied Hydrology, Ven Te Chow, Editor-in-Chief, McGraw-Hill Book Company, New York, New York, 1964.

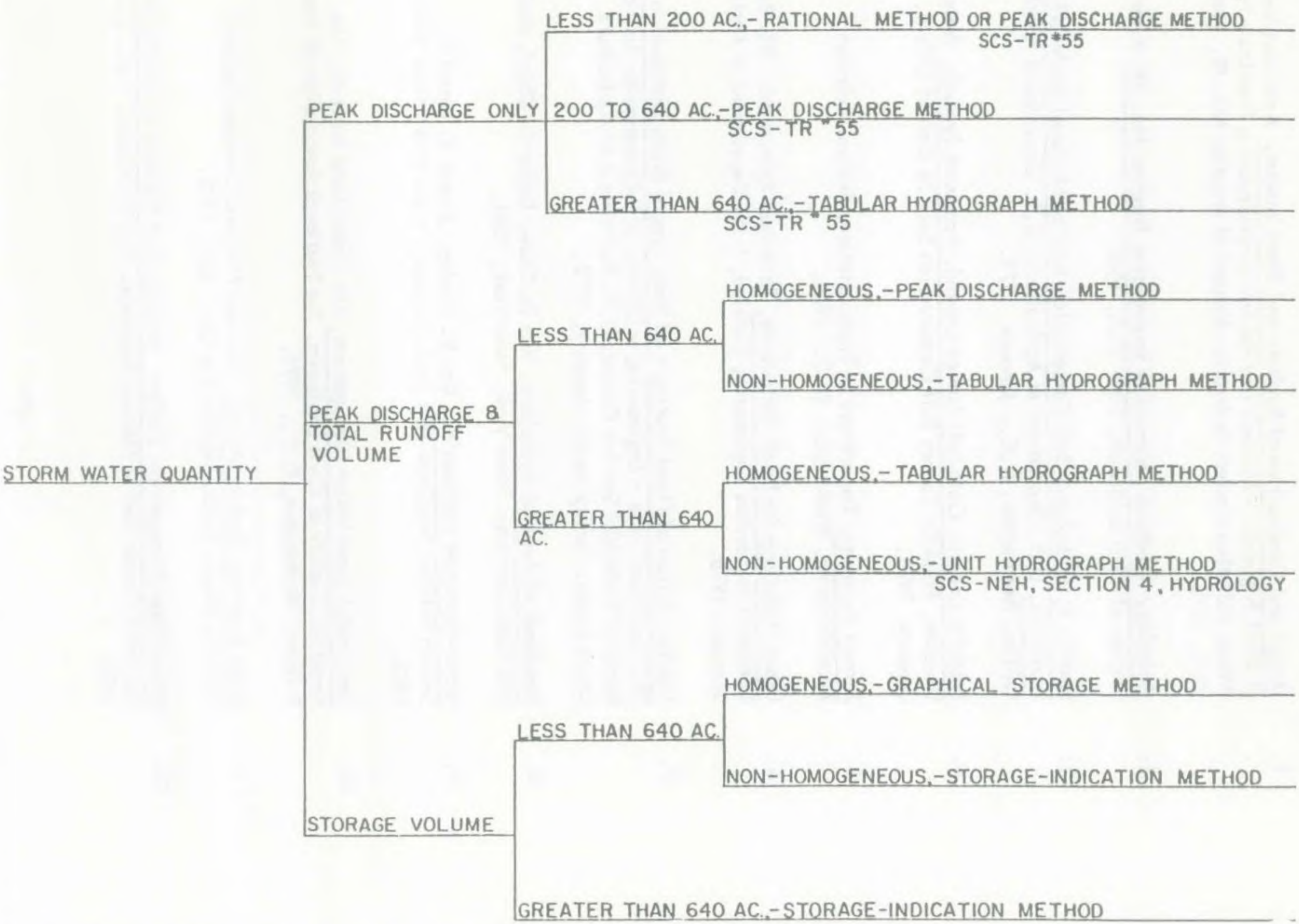
9. Water-Resources Engineering, Ray K. Linsley, Joseph B. Franzini, Second Edition, McGraw-Hill Book Company, New York, New York, 1972.

10. Residential Storm Water Management, The Urban Land Institute, The American Society of Civil Engineers, The National Association of Home Builders, Washington, D.C., 1975.

11. Storm Drainage Design Manual, Erie and Niagara Counties Regional Planning Board, Harza Engineering Co., July, 1972.

12. Storm Water Management Design: A Manual of Procedures and Guidelines, Maryland Department of Natural Resources, Roy F. Weston, Inc., April, 1976.

DESIGN FLOW CHART



RUNOFF COEFFICIENTS

LAND USE DESCRIPTION	HYDROLOGIC SOIL GROUP			
	A	B	C	D
Cultivated land : without conservation treatment	.32	.50	.66	.74
: with conservation treatment	.17	.30	.43	.50
Pasture or range land: poor condition	.26	.45	.61	.69
good condition	.05	.16	.36	.47
Meadow: good condition	.05	.13	.30	.43
Wood or Forest land: thin stand, poor cover, no mulch	.05	.23	.41	.54
good cover	.05	.10	.29	.41
Open Spaces, lawns, parks, golf courses, cemeteries, etc.				
good condition: grass cover on 75% or more of the area	.05	.16	.36	.47
fair condition: grass cover on 50% to 75% of the area	.05	.28	.45	.57
Commercial and business areas (85% impervious)	.69	.77	.83	.86
Industrial districts (72% impervious)	.50	.66	.74	.80
Residential:				
Average lot size — Average % Impervious				
1/8 acre or less — 65	.41	.59	.72	.77
1/4 acre — 38	.16	.37	.54	.64
1/3 acre — 30	.12	.32	.50	.61
1/2 acre — 25	.09	.29	.47	.59
1 acre — 20	.06	.26	.45	.57
2 acres	.05	.23	.41	.50
Paved parking lots, roofs, driveways, etc.	.96	.96	.96	.96

The coefficients are applicable for storms of five to ten year return frequencies.

For recurrence intervals longer than ten years, the indicated runoff coefficients should be increased assuming that nearly all of the rainfall in excess of that expected from the ten year recurrence interval rainfall will become runoff and should be accommodated by an increased runoff coefficient.

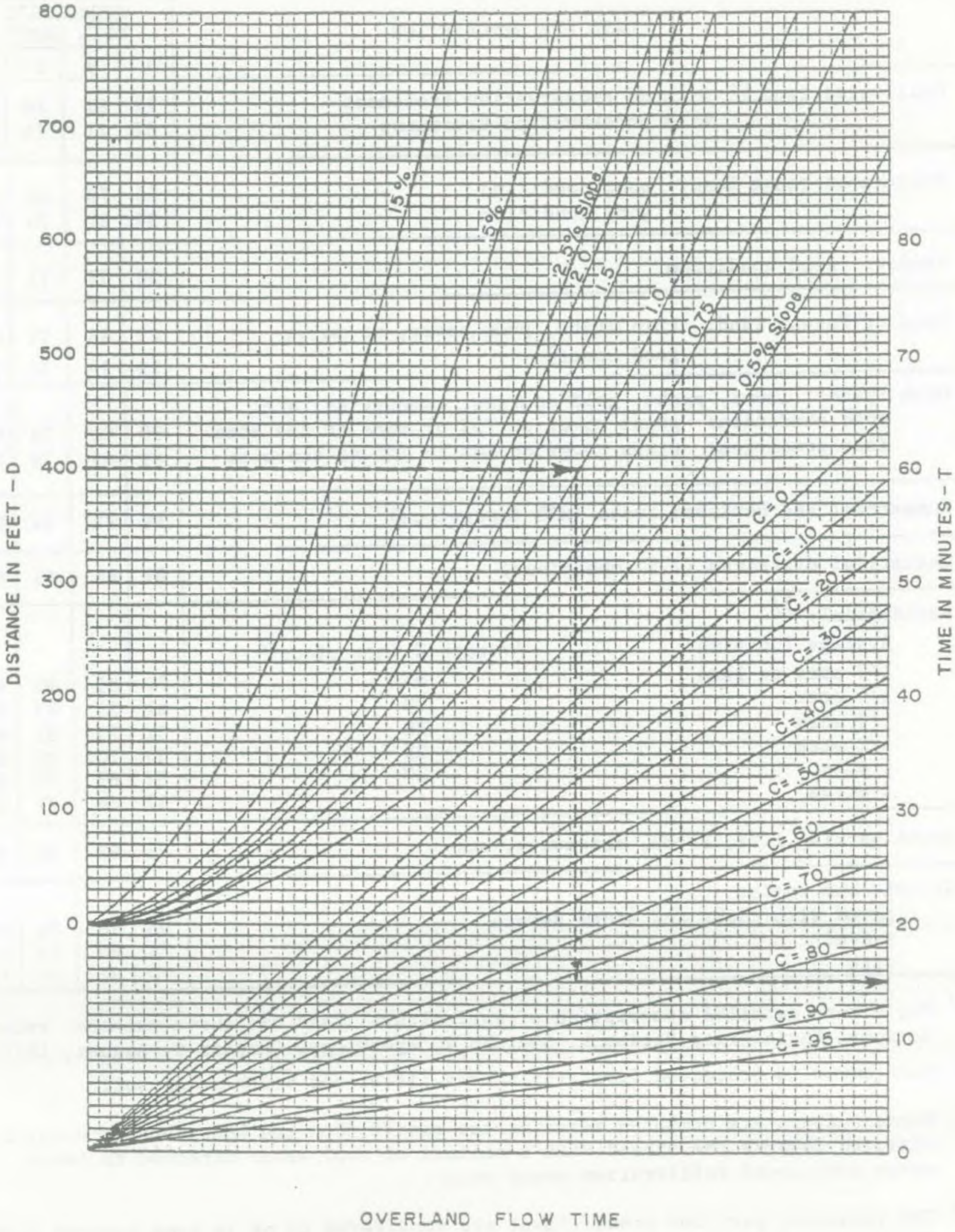
EXHIBIT V-3

HYDROLOGIC SOIL GROUPS
FOR
FRANKLIN COUNTY, OHIO

Soil Names	Identification Symbol	Hydrologic Classification
Alexandria silt loam	Ad	C
Algiers silt loam	Ag	C/D
Bennington silt loam	Be	C
Bennington - Urban land complexes	Bf	C
Blount silt loam	Bo	C
Brookston silty clay loam	Bs	B/D
Brookston - Urban land complex	Bt	B/D
Cardington silt loam	Ca	C
Cardington - Urban land complexes	Cb	C
Carlisle muck	Cc	A/D
Celina silt loam	Ce	C
Celina - Urban land complexes	Cf	C
Condit silt loam	Cn	D
Crane silt loam	Cp	B
Crosby silt loam	Cr	C
Crosby - Urban land complexes	Cs	C
Eel silt loam	Ee	C
Eldean silt loam	El	B
Genesee silt loam	Gn	B
Glynwood silt loam	Gw	C
Hennepin and Miamian silt loams	He	B and C
Kendallville silt loam	Ke	C
Lewisburg-Crosby complex	Le	C
Medway silt loam	Mh	B
Miamian silt loam	Ml	C
Miamian clay loam	Mm	C
Miamian - Urban land complex	Mn	C
Milton silt loam	Mo	C
Mittawanga	Mt	C
Montgomery silty clay loam	Mp	D
Ockley silt loam	Oc	B
Pewamo silty clay loam	Pm	B/D
Pewamo - Urban land complex	Pn	B/D
Ritchey silt loam	Rh	D
Ross silt loam	Rs	B
Shoals silt loam	Sh	C
Sleeth silt loam	Sl	C
Sloan silt loam	So	B/D
Stonelick sandy loam	St	B
Thackery silt loam	Th	B
Warsaw silt loam	Wd	B
Wea silt loam	We	B
Westland silty clay loam	Wt	B/D

NOTES: A blank hydrologic soil group indicates the soil group has not been determined. Two soil groups such as B/C indicate the drained/undrained situation.

SOURCES: Soil Conservation Service, NEH Notice 4-102, August, 1972. Franklin County Soil Conservation Service Office.



OVERLAND FLOW TIME

Source: "Airport Drainage" FAA 1965

RUNOFF CURVE NUMBERS
(Antecedent moisture condition II)

LAND USE DESCRIPTION	HYDROLOGIC SOIL GROUP			
	A	B	C	D
Cultivated land 1/: 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 2/	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: 3/				
Average lot size — Average % Impervious 4/				
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
2 acres	47	66	77	81
Paved parking lots, roofs, driveways, etc.	98	98	98	98
Streets and roads:				
paved with curbs and storm sewers	98	98	98	98
gravel	76	85	89	91
dirt	72	82	87	89

1/ For a more detailed description of agricultural land use curve numbers, refer to S.C.S. National Engineering Handbook, Section 4, Hydrology, Chapter 9, August, 1972.

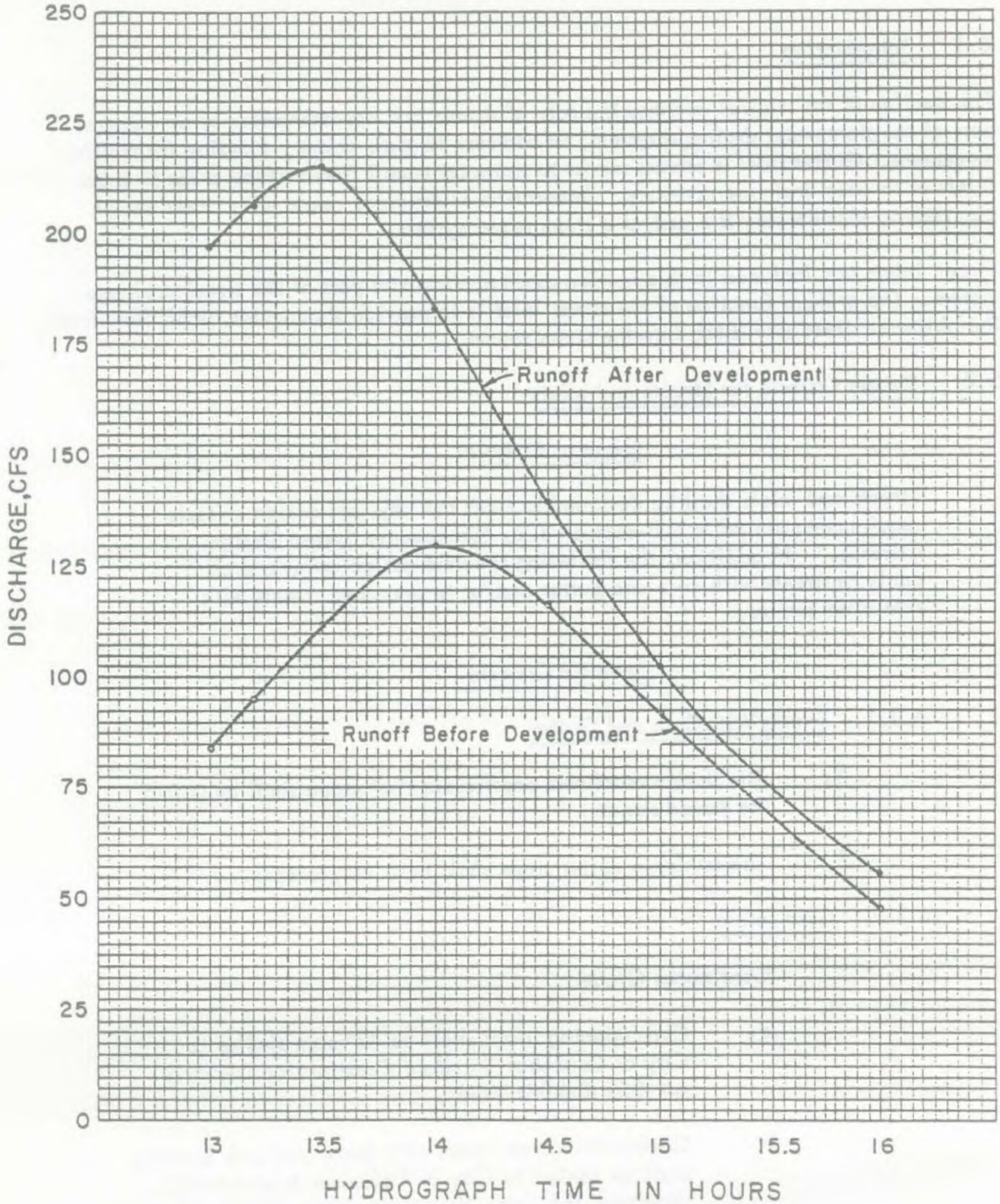
2/ Good cover is protected from grazing and litter and brush cover soil.

3/ 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.

4/ The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

SOURCE: S.C.S. Urban Hydrology For Small Watersheds, T.R. No. 55

CALCULATED COMPOSITE RUNOFF HYDROGRAPHS



6. STREETS AND INLETS

6.1 Introduction

This section of the manual outlines the role of streets and appurtenances as an integral part of the stormwater drainage system, in addition to their primary function for traffic movement. Gutter flow in streets serves to transport runoff from surface areas to storm inlets or to open drainage channels. Integration of drainage needs with street design can reduce considerably the cost of a storm sewer system.

Street inlets are included in this section as an appurtenant part of the street drainage system. The primary function of the street inlet is to provide stormwater runoff flow from the gutter or street side ditch into the storm sewer.

6.2 Design Criteria for Streets and Inlets

General Criteria

Streets and inlets shall be designed to meet the physical design criteria requirements stated in Sub-section 3.2, "Stormwater System Design Criteria." In addition, the following specific design criteria shall apply to streets with curb and gutter and/or to uncurbed streets with side ditch swales.

Specific Criteria

6.2.1 Streets With Curb and Gutter

1. The design roughness coefficient "n" value shall be 0.015 for paved streets.

2. The minimum gutter slope shall be 0.40 per cent.

6.2.2 Curb Inlets

1. Continuous Grade

   a. Curb opening inlets shall be 42" size similar to City of Columbus Standard Construction Drawing AA-S50 (Exhibit VI-6).

   b. Combination type inlets with grate and curb opening shall be similar to City of Columbus Inlet AA-S63. However, the grate flow shall not be included in the inlet capacity determination (Exhibit VI-7).

c. A local depression of 4" below the normal gutter flow line shall be provided at inlets.

d. Curb inlets shall be located at all points where the maximum pavement encroachment is reached and at the low points of street intersections. No cross street flow is permitted for the initial design storm.

2. Sag or Sump at Adverse Slopes

a. Combination type inlets are recommended at all sag or sump locations except low points of street intersections.

b. When combination type inlets are used, the grate capacity alone shall be considered the capacity of the inlet. The curb opening serves as a relief in the event the grate is clogged.

c. Recommended inlet locations are at the points of vertical curvature on each side of the sag and at least one inlet at the low point.

6.2.3 Streets with Side Ditch Swales

1. Side ditch swales shall be designed in accord with the general procedures stated in Section 9, "Open Channels" of this manual.

2. The minimum bottom slope shall be 0.40 per cent.

6.3 General Design Procedures

The following general design procedures are included to outline a uniform approach to the determination of gutter carrying capacity, and capacity of curb inlets. Examples are included in Sub-section 6.4 to illustrate the design procedures.

6.3.1 Gutter Capacity

1. Draw the street cross section and determine the permissible pavement encroachment (3.2.2).

2. Determine the gutter slope in ft/ft, and the reciprocal of the the cross slope (z).

3. Calculate the capacity for each gutter using Exhibit VI-1. The recommended procedure for computing the theoretical gutter carrying capacity is the modified Manning's formula, $Q = 0.56 (z/_n) S^{1/2} d^{8/3}$ for flow in triangular channels.

A nomograph for the solution of this formula is shown on Exhibit VI-1. The nomograph may be used for all gutter configurations. Instructions for the capacity determination of composite sections are included on the nomograph. Spread-Depth-Discharge charts for particular street shapes can be found in the material referenced. "Drainage of Highway Pavements", U.S. Department of Transportation, March 1969, is one such publication.

6.3.2 Capacity of Curb Opening Inlet on Continuous Grade

Once the gutter capacity of the street section is determined, it is necessary to determine the capacity of a curb inlet to intercept part or all of the gutter flow. Economy of design dictates that the first inlet should be located at that point where the maximum permissible gutter capacity is reached (Sub-section 3.2.2).

1. Determine the length (L) of the inlet opening and the depth of local flow line depression (a) at the inlet.

2. Calculate the design gutter discharge (Q_a) for the initial design storm as stated in the preceding Sub-section 6.3.1, including carryover from previous inlets.

3. Determine the gutter flow depth (d) at design Q_a for the particular street section using Exhibit VI-1.

4. Enter Chart A of Exhibit VI-2 with depth of flow in gutter (d) and local depression (a) and determine the interception per foot of inlet opening (Q_a/L_a) if the inlet were intercepting 100 per cent of the gutter flow.

5. Calculate the length, $L_a = Q_a \div (Q_a/L_a)$. If length L_a is less than actual inlet length L, 100 per cent of the flow is being intercepted. If $L_a > L$, determine percentage intercepted.

6. Calculate the ratio of actual inlet length (L) in feet to length of inlet required to intercept 100 per cent of gutter flow (L_a). The ratio is expressed as (L/L_a). Also, calculate the ratio a/d.

7. Enter Chart B of Exhibit VI-2 with the ratios calculated in Step 6 and determine Q/Q_a (the ratio of total flow intercepted by the inlet to gutter flow).

8. Calculate the total intercepted flow, $Q = Q/Q_a \times Q_a$.

9. The carryover flow to the next inlet is $Q_a - Q$.

10. Calculate the per cent of intercepted flow (per cent pick-up) = $Q/Q_a \times 100$.

6.3.3 Capacity of Curb Opening Inlet at Street Intersections

Inlets are usually placed immediately upstream from pedestrian crosswalks and street intersections. Curb opening inlets so placed should intercept 100 per cent of the gutter flow at adverse cross slopes for the initial design storm. The ponded water depth at such low points of street intersections is determined in terms of curb opening height (Exhibit VI-3).

1. Determine the vertical height of curb opening (h) at the curb face, including local depression (a).

2. Calculate the required capacity of the inlet per foot of length of opening, Q/L in cfs/foot.

3. Determine the ratio of ponded water depth (H) to vertical curb opening height (h), H/h, using Exhibit VI-3.

4. Calculate the ponded water depth, $H = H/h \times h$ (in feet).

5. The ponded water depth (H) is compared to the maximum allowable depth of flow in the gutter including local depression (a).

   a. If H is less than $(d + a)$ using the same units of depth, the curb opening inlet is intercepting 100 per cent of the initial design storm discharge.

   b. If H is greater than $(d + a)$, the physical design criteria are exceeded and adjustments in the design are necessary.

6.3.4 Capacity of Grate Inlet in Sump (Water Ponded on Grate)

The capacity of a grate inlet located at a low point or sump depends on either the exposed perimeter or the area of the grate openings and the depth of water over the grate. Hydraulic experiments have determined a grate will act as a weir for heads on the grate up to 0.4 foot, and as an orifice for heads of 1.4 feet and greater.

The type of grate flow for heads between 0.4 and 1.4 feet is indefinite due to turbulent hydraulic characteristics (vortices and eddies).

Both the effective perimeter and the effective area of a grate are considered in determining the appropriate type of hydraulic flow. Exhibit VI-4 includes discharge curves for weir and orifice flow. The following general procedures are stated for a combination type grate and curb opening inlet. The same procedures would apply to a grate only inlet; however, in consideration of possible clogging of the grate it is recommended the design perimeter and the design area of the grate be one-half ($\frac{1}{2}$) of the effective perimeter (P) and effective area (A) determined below.

1. Determine the effective perimeter of the grate opening (P) in feet ignoring the bars and omitting any side of the grate over which water does not enter; e.g., side against face of curb.

2. Calculate the discharge per foot of perimeter ($Q/_p$). Q is the total gutter discharge from each side or the grate.

3. Determine the total clear opening area (A), excluding the area of the bars.

4. Calculate the discharge per square foot of effective area (Q/A).

5. Enter Exhibit VI-4 with the values of $Q/_p$ and $Q/_A$ and read the required head (H) in feet using the appropriate weir or orifice curve.

6. Compare the two head values from Curve A and Curve B to determine the type of flow; i.e., weir flow or orifice flow.

7. If the required head (H) is between 0.4 and 1.4 feet, the actual head may be anywhere in this head range. Use the value that gives the more conservative result (highest H).

8. Compare the value of H determined in the preceding steps to the maximum allowable gutter depth (d) including local depression (a).

   a. $H > (d+a)$ indicates that the allowable ponding limits are exceeded and that additional inlets are required.

   b. $H < (d+a)$ indicates the inlet has ample grate capacity and the maximum allowable ponding limits will not be exceeded.

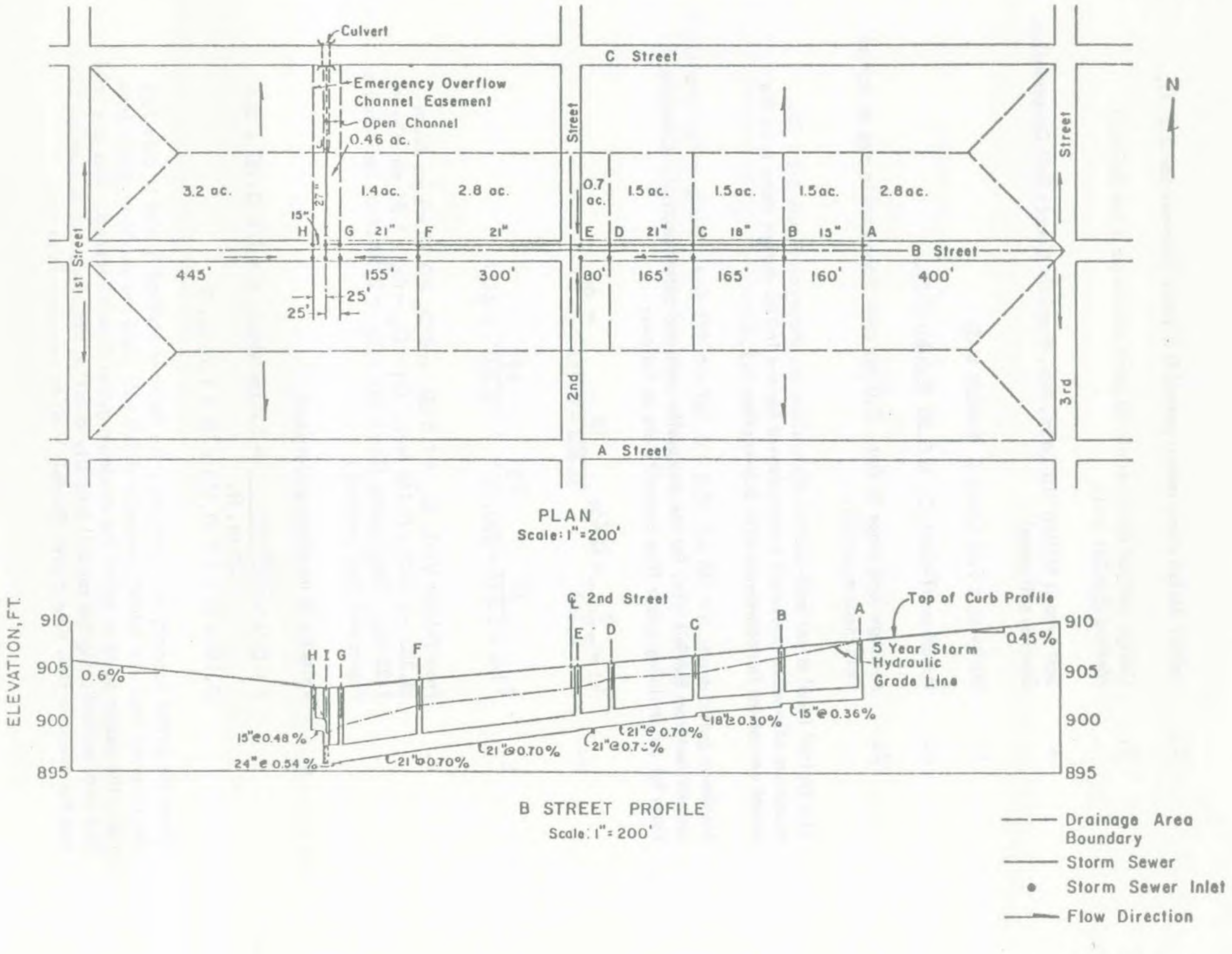
As stated earlier, the curb opening of a combination inlet serves as an emergency overflow in the event the grate becomes clogged; and, therefore, the curb opening capacity is not included in the capacity determination of combination type inlets in a sump.

Example calculations are included in the following Sub-section to illustrate the general design procedures stated herein.

6.4 Example Calculations

A typical residential area as shown in Figure 6-1 is proposed for development into one-third acre lots. The following design data are either given or are determined from other sections of this manual:

1. Tributary drainage area along street B = 15.9 acres (Figure 6-1).



TYPICAL RESIDENTIAL AREA

2. Initial design storm return period is 5 years (Sub-section 3.2.1).

3. Design method for determining peak discharge is the Rational Method (Exhibit V-1).

4. Soil type is Milton Silt Loam, Mo. (Franklin County Soil Conservation Service soil maps).

5. Hydrologic Soil Group C (Exhibit V-3)

6. Runoff coefficient, C, is 0.50 (Exhibit V-2).

7. Average land slope is flat, 2.0 per cent (topographic map or actual field measurements).

The typical local street half-section dimensions are shown on Figure 6-2. The maximum allowable pavement encroachment for the initial design storm is to the street centerline in accordance with Sub-section 3.2.2.

Maximum gutter depth, d = (10 x 0.056) + 0.167 = 0.323 feet. Design gutter capacity is determined from Exhibit VI-1 for the composite curb and gutter section. Computations for determining gutter flow capacity are as follows:

a. $Z_1 = \frac{2}{0.167} = 12; Z_2 = \frac{10}{0.323 - 0.167} = 64$

b. $Z_{1/n} = \frac{12}{0.015} = 800; \frac{Z_2}{n} = \frac{64}{0.015} = 4267$

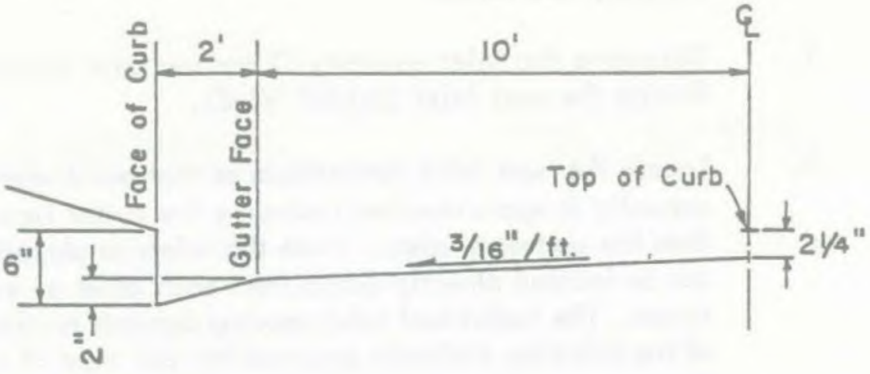
c. From Exhibit VI-1, $Q_a = 1.5$ cfs, and $Q_b = 0.25$ cfs for depth (0.323 - 0.167) = 0.156 feet. $Q_1 = Q_a - Q_b = 1.25$ cfs, $Q_2 = 1.25$ cfs. Total gutter flow = $Q_1 + Q_2 = 2.50$ cfs as indicated on Figure 6-2 (half section).

d. Velocity @ maximum gutter depth, d.

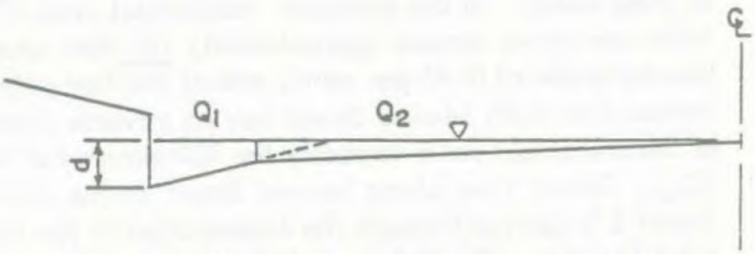
$V = Q/A = \frac{2.50 \text{ cfs}}{1.26 \text{ sq. ft.}} = 2.0$ fps where, $A = 1/2\ (0.167 \times 2) + (0.156 \times 2) + 1/2\ (0.156 \times 10) = 1.26$ sq. ft.

Once the gutter capacity is determined, the Rational Method is used to calculate the tributary area and rainfall intensity which will yield the maximum gutter flow i.e., the design point at which the maximum gutter flow is reached. This is a trial and error method using the rainfall intensity-duration frequency curves (Exhibit IV-1) and the overland flow time curves (Exhibit V-4) to calculate inlet time.

Figure 6-2



TYPICAL LOCAL STREET HALF SECTION



d = Maximum Gutter Depth = 0.323 feet

Q_1 = 1.50 - 0.25 = 1.25 cfs

Q_2 = 1.25 cfs

Total Q = 2.50 cfs

EXAMPLE GUTTER FLOW CAPACITY

A generalized design approach to inlet spacing is as follows:

1. Locate the first inlet at the point where the maximum gutter capacity is reached.

2. Determine the inlet capacity Q and per cent carryover (gutter flow) to the next inlet (Exhibit VI-2).

3. Locate the next inlet downstream at that point where the gutter capacity is again reached including the gutter flow (carryover) from the upstream inlet. Note the inlets so placed may or may not be located directly across from each other on each side of the street. The individual inlet spacing depends on the configuration of the tributary drainage area and the per cent of carryover from the upstream inlet.

4. Continue locating inlets at maximum gutter capacity points on continuous grades until a street intersection or a low point (sag) in the street profile is reached.

5. At street intersections, inlet locations vary, depending upon the respective street grades and pedestrian convenience. In general, inlets should be located at the upstream curb turnouts adjacent to cross walks. In the example residential area (Figure 6-1), inlets are shown spaced approximately <u>165</u> feet apart on the continuous grade of 0.45 per cent, and at the low points of the intersection with Second Street having adverse slopes. The inlets at Second Street have capacity for 100 per cent of the gutter flow (Q_a). Gutter flow along Second Street on the downstream side of Street B is carried through the intersection to the next (downstream) inlet location. The inlets at design points "F" and "G" are located at maximum gutter capacity points and the procedure is repeated to the sag vertical curve.

6. Inlets are located at the sag vertical curve. The inlets are designed for a capacity adequate to intercept 100 per cent gutter flow.

Example pavement drainage computations are shown on Exhibit VI-5. The curb inlet selected for streets on continuous grade and at street intersections is the 42 inch City of Columbus Curb Inlet (Exhibit VI-6). The curb inlet selected for the sag vertical curve is the combination type grate and curb opening inlet shown in Exhibit VI-7.

Again, the design procedures discussed herein for inlet locations are a generalized approach to design and are not intended as a "cook book" procedure. Engineering experience and judgment is an important factor in any design and in many instances will justify deviation from usual design procedures to achieve the desired system performance with respect to the unique site features.

6.5 Bibliography

1. Urban Storm Drainage Criteria Manual, Volume 1, Denver Regional Council of Governments, Wright-McLaughlin Engineers, Denver, Colorado, March, 1969.

2. Design Charts for Open-Channel Flow, U.S. Department of Commerce, Bureau of Public Roads, Washington, D.C., August, 1961.

3. Design of Roadside Drainage Channels, U.S. Department of Commerce, Bureau of Public Roads, Washington, D.C., May, 1965.

4. Residential Streets, The Urban Land Institute, The American Society of Civil Engineers, The National Association of Home Builders, Washington, D.C., 1974.

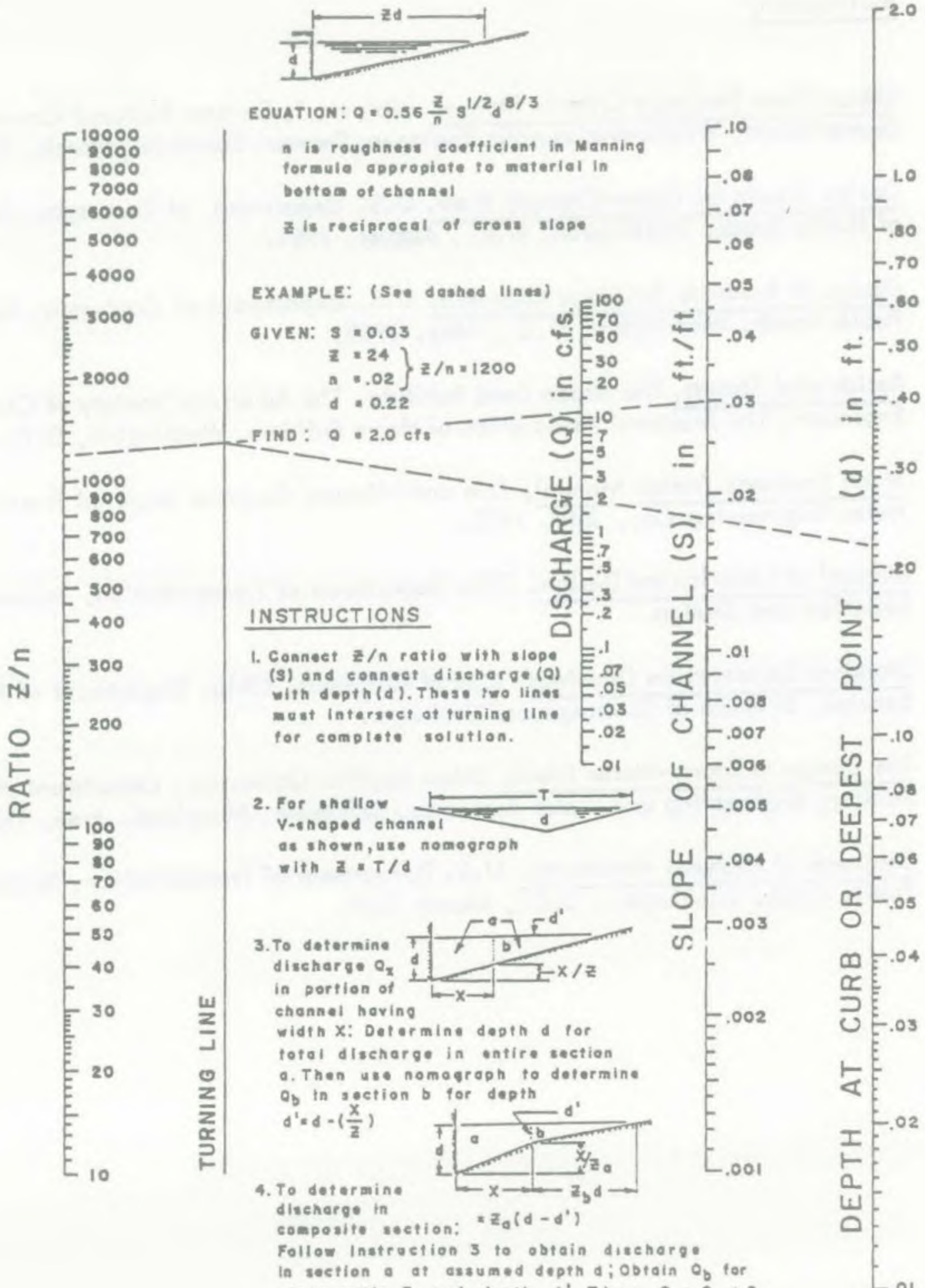
5. Storm Drainage Design Manual, Erie and Niagara Counties Regional Planning Board, Harza Engineering Co., July, 1972.

6. Manual of Location and Design, Ohio Department of Transportation, Bureau of Location and Design.

7. Standard Construction Drawings, City of Columbus, Ohio, Department of Public Service, Division of Sewerage and Drainage.

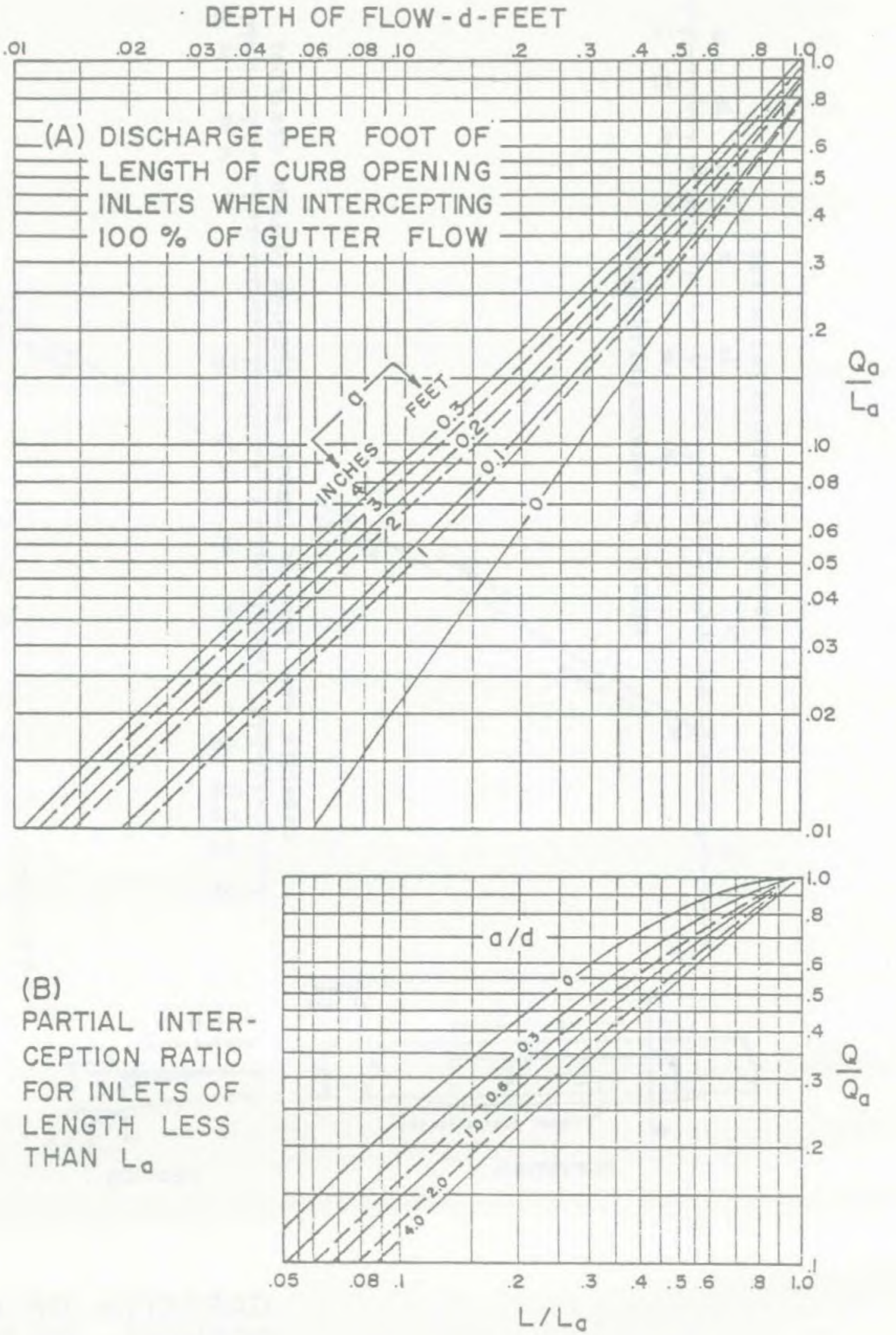
8. The Design of Storm-Water Inlets, Johns Hopkins University, Department of Sanitary Engineering and Water Resources, Baltimore, Maryland, June, 1956.

9. Drainage of Highway Pavements, U.S. Department of Transportation, Bureau of Public Roads, Washington, D.C., March 1969.



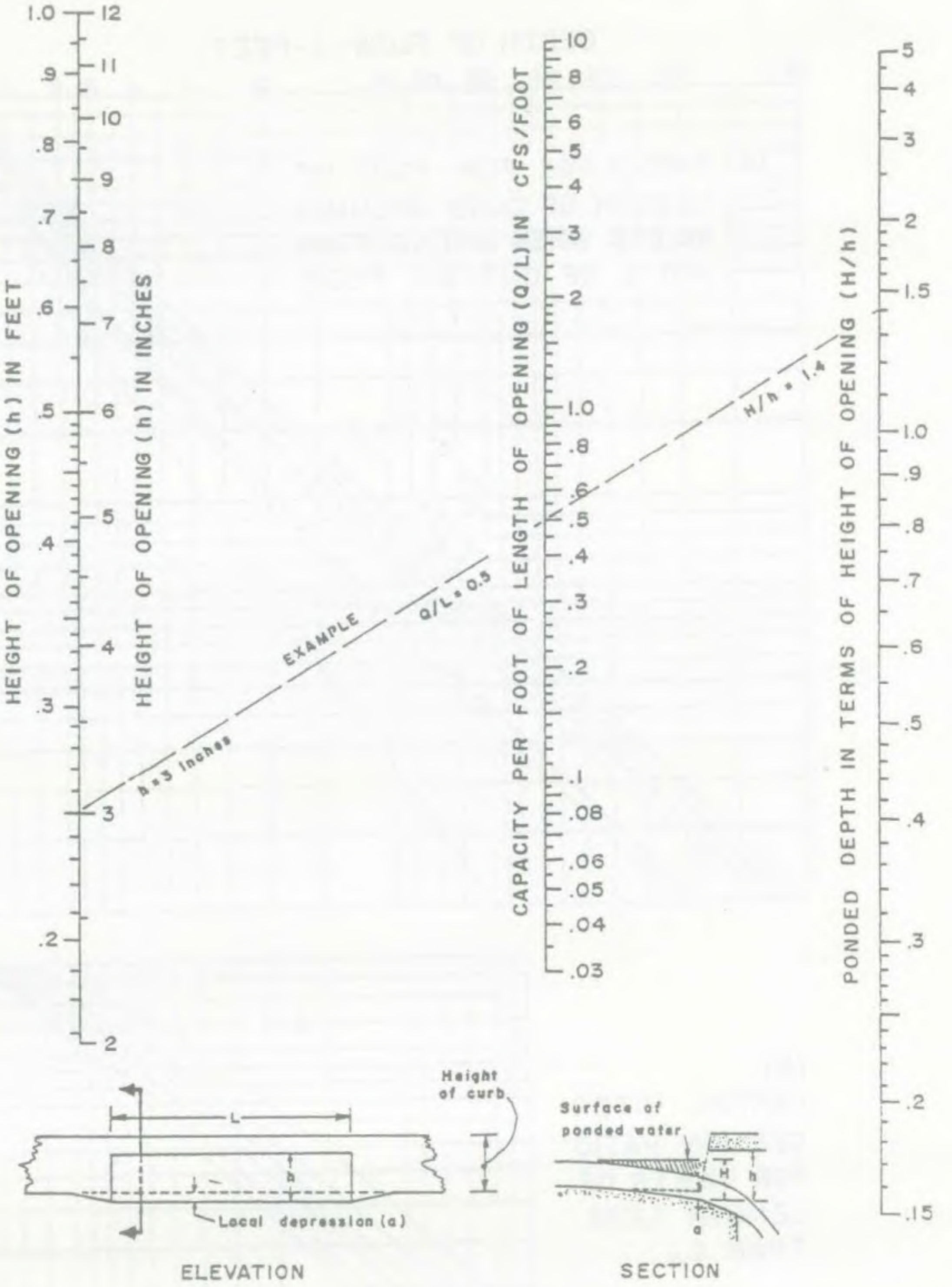
NOMOGRAPH FOR FLOW
IN TRIANGULAR CHANNELS

Source: Bureau of Public Roads, "Design Charts for Open-Channel Flow"



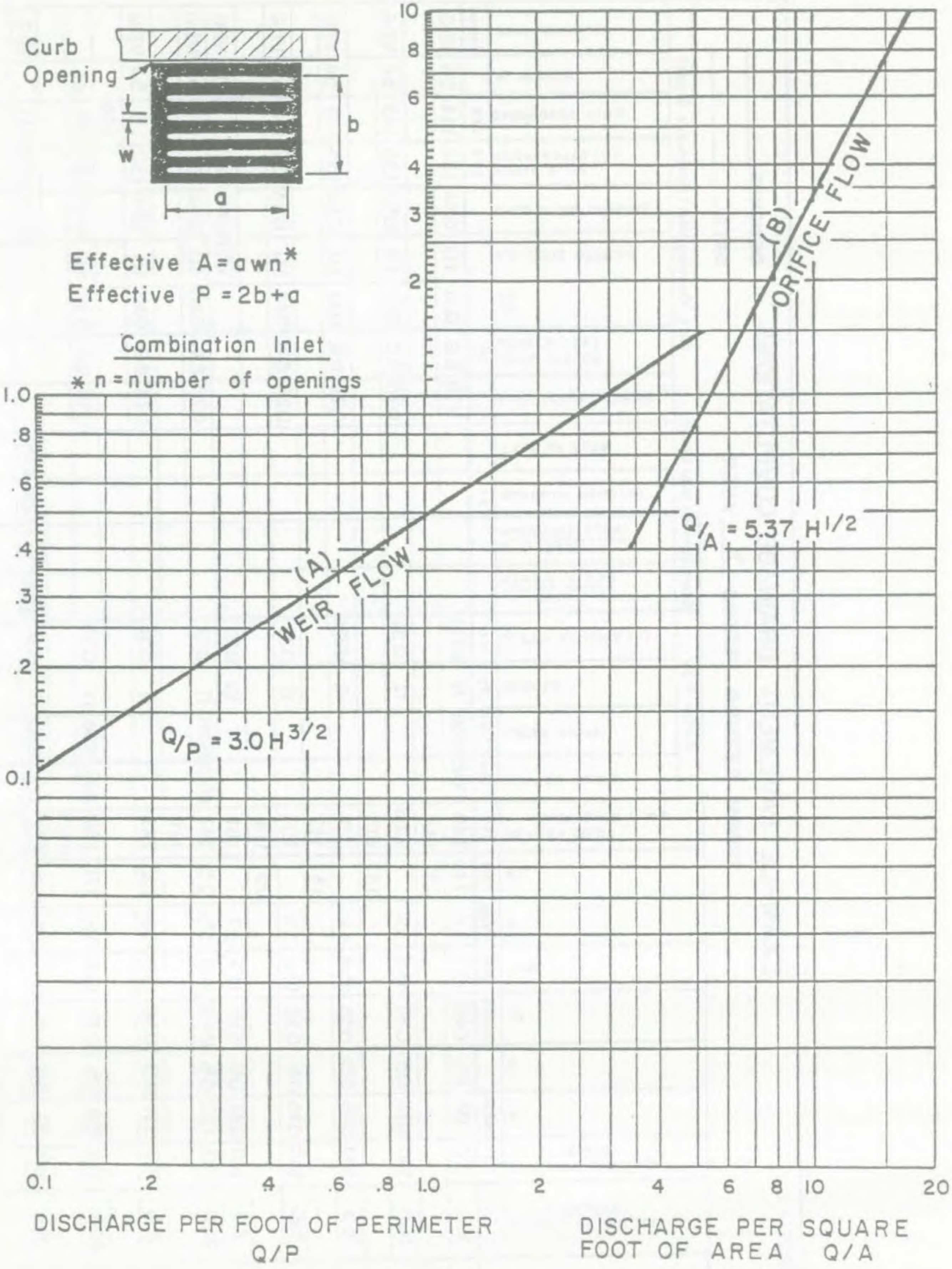
CAPACITY OF CURB OPENING INLETS ON CONTINUOUS GRADE

Source: Bureau of Public Roads



CAPACITY OF CURB OPENING INLET AT LOW POINT IN GRADE (STREET INTERSECTION)

Source: Manual of Location & Design
Ohio Department of Transportation



HEADS UP TO 0.4, CURVE (A) APPLIES
HEADS ABOVE 1.4, CURVE (B) APPLIES
HEADS BETWEEN 0.4 & 1.4, TRANSITION
SECTOR & OPERATION IS INDEFINITE

CAPACITY OF GRATE INLET
IN SUMP
(WATER PONDED ON GRATE)

Source: Bureau of Public Roads

PROJECT:

EXAMPLE PAVEMENT DRAINAGE COMPUTATIONS

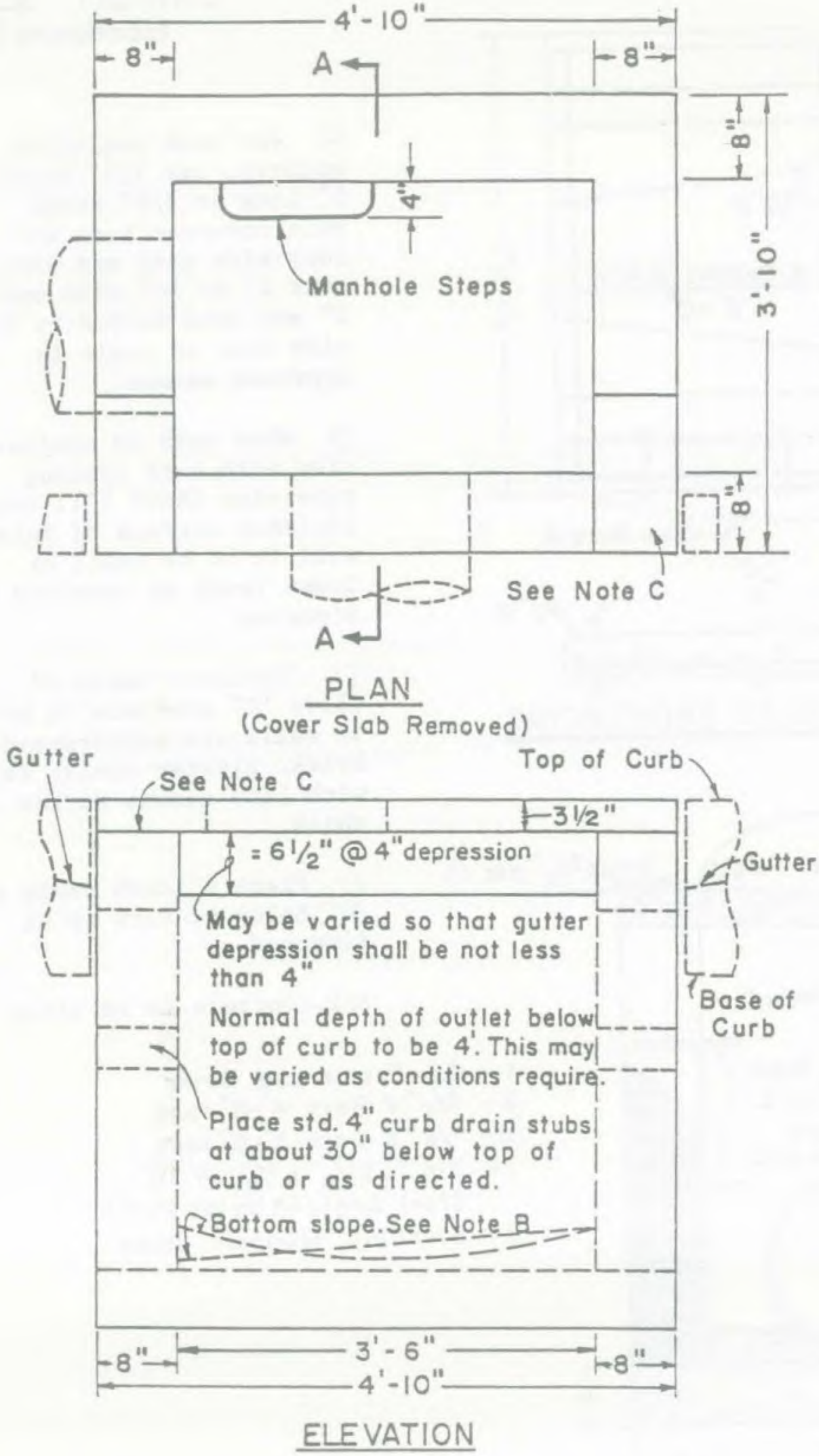
STORM RETURN PERIOD 5 YEAR

DESIGNER ____________

DATE ____________

										Gutter Flow				Grate Opening Inlet (Sag)				Curb Opening Inlet (Continuous Grade)								
INLET LOCATION (DESIGN POINT)	LENGTH OF GUTTER FLOW, FEET	DRAINAGE SUBAREA WIDTH	A	C	CA	TIME	I	ΔQ	Qa = ΔQ + INLET CARRYOVER FLOW	GUTTER SLOPE	CROSS SLOPE	SPREAD	DEPTH AT CURB (a)	Qa/p / H (WEIR FLOW)	Qa/A / H (ORIFICE FLOW)	MAXIMUM HEAD (H)	TYPE OF FLOW	Qa/La CURB OPENING	100% PICKUP LENGTH (La)	L/La	a/d CURB OPENING	Q/Qa CURB OPENING	TOTAL FLOW INTERCEPTED (Q)	CARRYOVER FLOW	% PICKUP	TYPE OF INLET
		FT.	ACRES		ACRES	MIN.	IN/HR	C.F.S.	C.F.S.	FT/FT	FT/FT	FT.	FT.			FT.			FT.				C.F.S	CFS		EXHIBIT
A	400	200	1.38	0.5	0.69	16.2	3.6	2.48	2.48	.0045	.0156	12	0.323					0.29	8.6	0.41	1.0	0.54	1.34	1.14	54	VI-6
								1.32	1.14																	
B	160	200	0.73	0.5	0.37	15.3	3.6		2.46			12	0.32					0.28	8.8	0.40	1.0	0.54	1.33	1.13	54	VI-6
								1.37	1.13																	
C	165	200	0.76	0.5	0.38	15.4	3.6		2.5			12	0.323					0.29	8.6	0.41	1.0	0.54	1.35	1.15	54	VI-6
								1.37	1.15																	
D	165	200	0.76	0.5	0.38	15.4	3.6		2.52			12	0.33					0.29	8.6	0.41	1.0	0.54	1.36	1.14	54	VI-6
								0.67	1.14																	
E	80	200	0.37	0.5	0.18	14.7	3.7		1.81			10	0.27								(Exhibit VI-3)				100	VI-6
F	300	200	1.39	0.5	0.69	16.5	3.5	2.5	2.42	.0045	.0156	12	0.32					0.28	8.6	0.40	1.0	0.54	1.31	1.11	54	VI-6
									1.11																	
G	155	200	0.71	0.5	0.36	15.3	3.6	1.30	2.41			12	0.32					0.28	8.6	0.40	1.0	0.54	1.30	1.11	54	VI-6
																								2.36		
H	445	200	1.58	0.5	0.79	16.3	3.6	2.0	2.84	.006	.0156	12	0.32					0.29	9.8	0.36	1.0	0.47	1.33	1.51	47	
									1.51(H)																	
I(sag)	50	200	.23	0.5	0.12	14.2	3.8	0.46	1.11(G) / 3.08	.001	.0156	12	0.36	0.63 / 0.36	1.78 / -	0.36	Weir					(Exhibit VI-4)				VI-7

EXHIBIT VI-5



42" CURB INLET

NOTES

A: Where inlet is to be located within limit of circle curb, if ordered, the slab edge at curb line shall be built to conform to the required radius.

B: The inlet bottom shall be shaped to provide slope of 3" to 4" to outlet pipe. The cross sectional form of the bottom and longitudinal slope to be adapted to location of outlet pipe as directed.

C: In brick wall construction, suitable concrete or stone blocks not less than 8" x 8" x 16" in size shall be used at corners facing curbs.

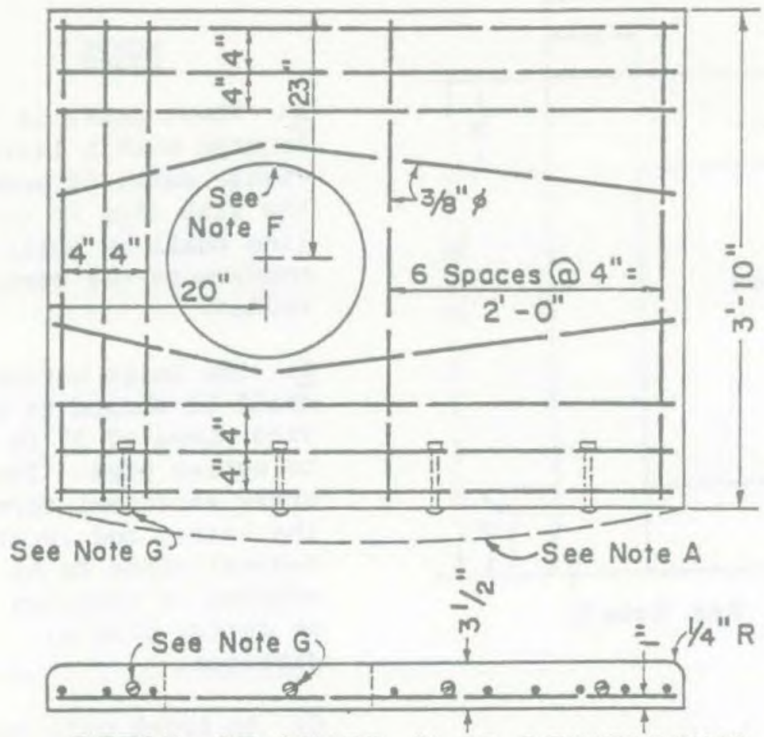
D: Outlet pipe may be located in end corners or sidewall. In either case, the outlet pipe shall be directed towards the center of inlet.

E: In existing pavements, an area approx. 4 ft. outside inlet opening or as otherwise ordered shall be cut out of the gutter so that repaving may be shaped to meet the depressed lip of the opening as directed.

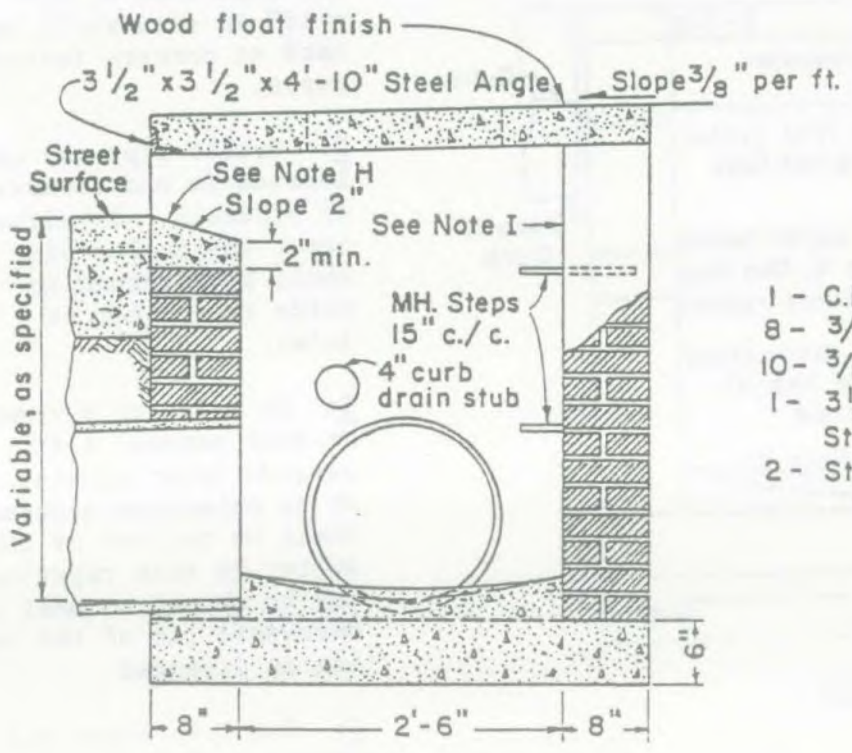
F: Use C.I. Frame and cover equal to Clow F 3310 (17 7/8" opening - 68 lb.) as listed in Clow-National catalogue No. 4, 1943.

Source: City of Columbus
Division of Sewerage and Drainage

EXHIBIT VI-6
(continued)



DETAIL OF COVER SLAB REINFORCING



SECTION A-A

G: For curb angleiron anchors, use 3/8" bolts 6" long or 3/8" steel reinforcement bars 10" long with each end bent over 2" at 90° with one 2" end bend welded to inside face of angle in approved manner.

H: When used in conjunction with curb opening extension (AA-S 10), top finished surface of inlet wall is to be built at lower level as required or directed.

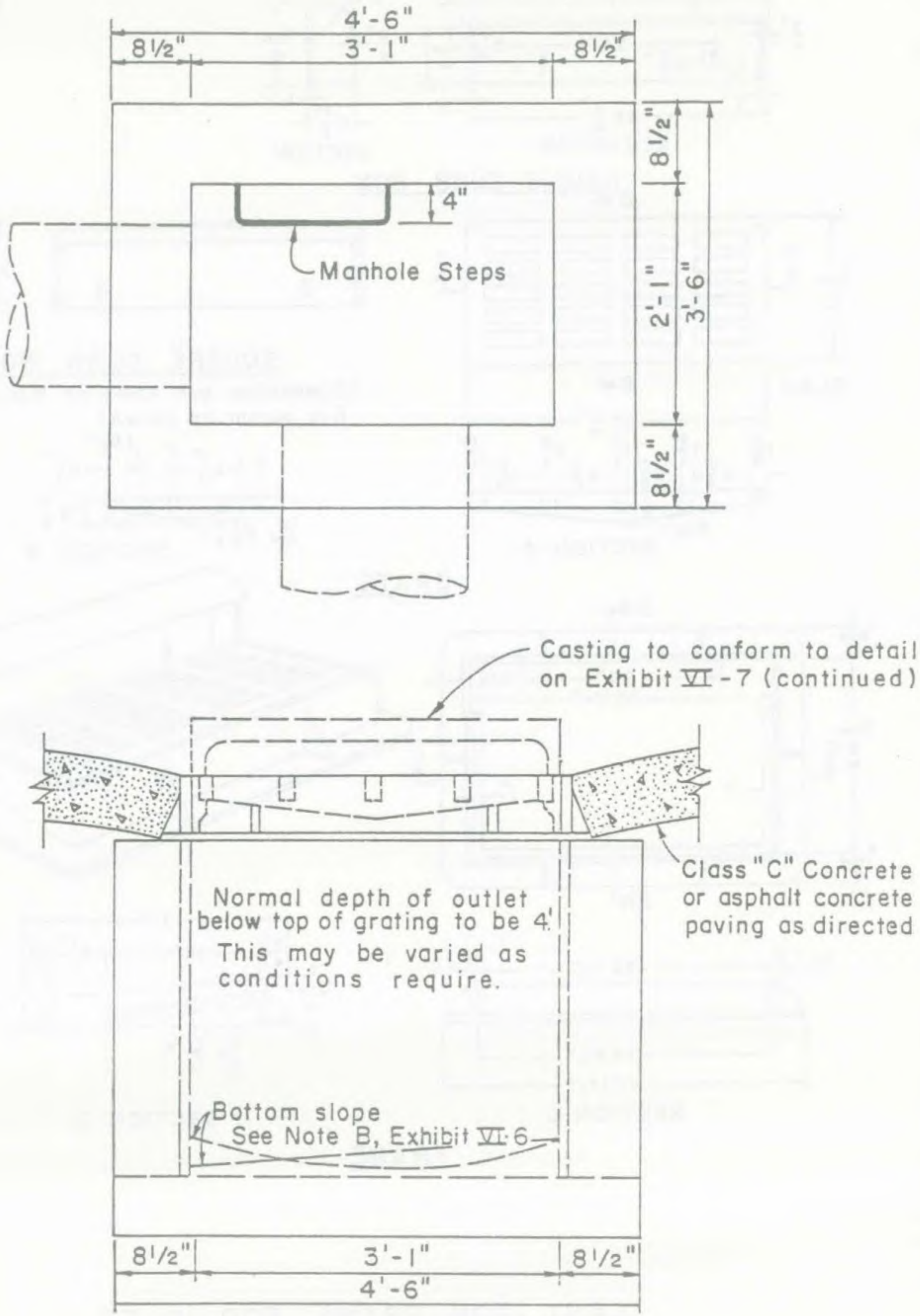
I: Construct walls of Class "C" concrete or brick. If walls are constructed of brick, plaster inside face with lime cement mortar 1/2" thick.

J: Place 4" curb drain stubs 30" below of curb or as directed.

All concrete to be Class "C".

1 - C.I. Frame and Cover
8 - 3/8" ø Bars 4'-6" long
10 - 3/8" ø Bars 3'-6" long
1 - 3 1/2" x 3 1/2" x 3/8" x 4'-10" Steel Angle (8 1/2 lbs. PLF)
2 - Standard Manhole Steps

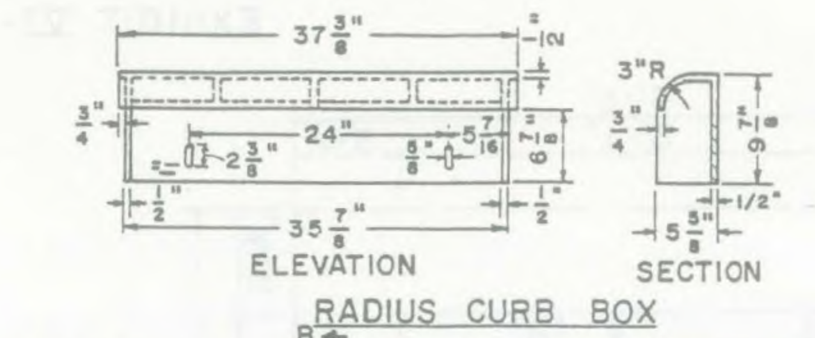
42" CURB INLET

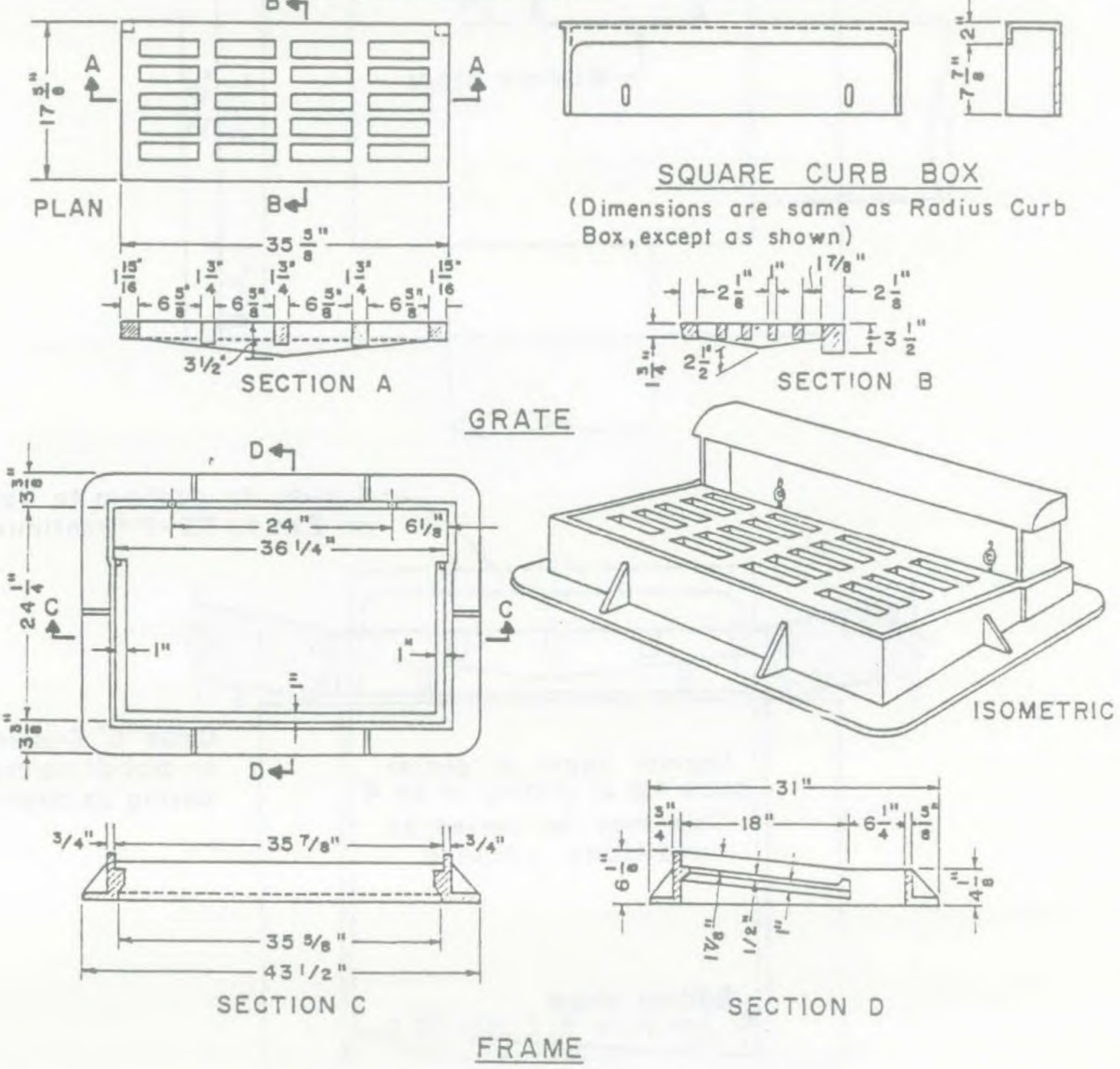


COMBINATION CURB INLET

Source: City of Columbus
Division of Sewerage and Drainage

EXHIBIT VI-7
(CONTINUED)





CAST IRON FRAME FOR INLET

Source: City of Columbus, Standard Construction Drawings

7. STORM SEWERS

7.1. Introduction

Storm sewer systems are designed to collect and convey stormwater runoff from street inlets, from runoff control structures, and from other locations where the accumulation of stormwater is undesirable. Storm sewers are closed conduits which convey stormwater runoff from a drainage area to an outlet. Although the storm sewer is a basic part of the initial storm drainage system, it also serves to carry a significant portion of the runoff during the infrequent major storms.

The objective of a storm sewer system is to remove runoff from an area fast enough to avoid unacceptable amounts of ponding damage and inconvenience. Therefore, design criteria and general design procedures are presented herein to meet these objectives. Example calculations are included for illustration of the general design procedures.

7.2. Design Criteria for Storm Sewers

General Criteria

Storm sewers shall be designed to meet the physical design criteria requirements stated in Sub-section 3.2, "Stormwater System Design Criteria." In addition, the following specific design criteria are presented to guide the engineering design of storm sewers.

Specific Criteria

1. Depth - The depth of the storm sewer shall be sufficient to receive water from street inlets and other drainage structures. The minimum cover for storm sewers crossing streets with curb and gutter shall be one foot (clearance) from the bottom of the curb or underdrain to the top of conduit. A minimum cover of two feet from finished ground surface is recommended at all other locations.

2. Velocity - A minimum velocity of three feet per second (fps) is recommended to insure self-cleaning. The maximum allowable velocity shall be 12 fps unless special materials are included for protection against scouring.

3. Design Discharge Method - The Rational Method or Peak Discharge Method from Soil Conservation Service Manual, TR #55 may be used for drainage areas less than 200 acres to determine peak discharge. The design method for areas greater than 200 acres can be determined from Exhibit V-1.

4. Hydraulic Design - The hydraulic design of storm sewers shall be based on the Manning Equation; $V = \frac{1.49}{n} r^{2/3} S^{1/2}$

   Special slide rules are available for the solution of this equation and nomographs are included herein. The hydraulic grade line for both the initial and major design storms shall be considered.

5. Roughness Coefficients - Table 7-1 lists the range of Manning roughness coefficients (n) to be used for different conduit materials.

6. Manhole Spacing - Manholes should be located at junctions of conduits, at changes in conduit direction, and at changes in slope. Maximum spacings are usually in the range of 300 feet to 400 feet to facilitate maintenance activities. Large conduit sizes which permit a person to enter may have manholes spaced 500 feet to 600 feet apart.

7. Conduit Size - The minimum conduit size shall be 12 inches in diameter or equivalent. Although open channel design is an alternative which should always be considered for its appropriateness, drainage systems requiring 72 inch and larger conduits may be especially suited for open channel flow.

8. Hydraulics at Structures - The inverts of curb inlets, manholes, and other structures shall be formed to minimize turbulence and collection of debris. Special consideration shall be given to the hydraulic head losses through structures connecting different size conduits, and conduits having a change in either horizontal or vertical alignment. (See Appendix A).

7.3. General Procedures for Storm Sewer Design

The following general design procedures provide a uniform approach to storm sewer design. The procedure as outlined is for a storm sewer system serving an urban area with curbed streets. With minor modifications it can apply as well to streets with side ditch swales.

1. The general procedures for street and inlet design, and a generalized approach to inlet spacing are discussed in the preceeding section of the manual (Chapter 6). Street and inlet design is a basic part of the storm sewer drainage system. Maximum use should be made of the street gutter capacity to transport stormwater runoff to inlets, and thereby reduce the size of the storm sewers.

TABLE 7-I

MANNING ROUGHNESS COEFFICIENTS

Closed Conduit Material	Manning "n" (ft. 1/6)
Asbestos - cement pipe	0.011 - 0.015
Brick	0.013 - 0.017
Cast iron pipe, cement lined and seal coated	0.011 - 0.015
Concrete (monolithic)	
Smooth forms	0.012 - 0.014
Rough forms	0.015 - 0.017
Concrete Pipe	0.011 - 0.015
Corrugated metal pipe (1/2 inch x 2 3/4 inch corrugations)	
Plain	0.022 - 0.026
Paved invert	0.018 - 0.022
Spun asphalt lined	0.011 - 0.015
Plastic pipe (smooth)	0.011 - 0.015
Vitrified clay	
Pipes	0.011 - 0.015
Linear Plates	0.013 - 0.017

Source: American Society of Civil Engineers, Manuals and Reports on Engineering Practice No. 37, 1970.

2. The following basic data is required:

   a. Map of drainage area for which the storm sewer system is to serve. (Subdivision plan supplemented by United States Geological Survey (USGS) maps for off-site area, if required).

   b. Typical street cross sections and profiles (use local community design standards).

   c. Pavement Drainage Computations (formatted work sheet provided).

   d. Soil maps and data (Exhibit V-3).

   e. Outfall Elevation (from field measurement).

   f. Rainfall intensity-duration-frequency curves (Exhibit IV-1).

3. Once the locations of the proposed curb inlets, as determined by gutter capacity (Section 6.3) are determined, the designer can begin sizing storm sewers. Calculations for both the preliminary and final design can be indicated on the storm sewer design computations sheet. Since most storm sewers in this area will probably be designed by the Rational Method, the example design calculations included, Exhibit VII-1, are based on that method.

4. One approach to storm sewer design is to calculate the conduit sizes working downstream along the system. At each design point, $\Sigma Q = CA\,I$. Flowing full (without surcharge) conditions are assumed without regard to the hydraulic head losses through structures. Then, the hydraulic grade line elevations are computed by working back upstream from some control point in the system. The hydraulic head losses at each structure are calculated to determine the type of flow in the conduit (open channel or pressurized flow), and to assure that the hydraulic grade line elevation remains below the gutter elevation.

   Detailed design procedures from <u>Urban Storm Drainage Criteria Manual, Volume 1,</u> Denver Regional Council of Governments, March, 1969, are included in Appendix A. This presents design procedures for both open channel and pressurized conduits.

5. The final design is drawn on prepared plan and profile sheets.

6. The hydraulic effects of the major design storm on the drainage system are determined for compliance with the physical design criteria presented in Section 3.2.3.

7.3.1 Example Calculations (Initial Design Storm)

Example calculations are presented for the typical residential area discussed in Section 6.4 and shown in Figure 6-1, where drainage sub-areas are delineated and curb inlet locations noted. Other design data is given in Section 6, Streets and Inlets.

Storm sewer design computations are shown on Exhibit VII-1. In this Exhibit the category, "Other Controlled Runoff", applies to the controlled runoff from storage structures such as rooftops, parking lots, and detention basins. The design features of these structural control facilities is presented in Chapter 10.

In this example, the proposed storm sewer material is reinforced concrete, Manning's n = 0.011 (Table 7-I). The initial system design storm frequency is five years. The hydraulic grade line (HGL) elevations for a five-year storm are determined. These HGL elevations are compared to the gutter line elevations at each design point for compliance with the physcial design criteria of Sub-section 3.2.2. HGL elevations above the gutter line indicate a modification in conduit design. The backwater effects from the downstream channel at design point "J" are considered to determine the beginning HGL elevation. The conduit friction loss, hf, is added to the pressure elevation at the downstream end of a conduit, then the pressure rise at the junction for each upstream conduit is added to determine a new starting elevation for the next line of conduit upstream. When the hydraulic grade line (HGL) is below the crown of the conduit, open channel flow methods must be used and the HGL is the water surface (flow depth, dn) in the conduit. This condition exists in conduit I-J of the example (Exhibit VII-1).

The final design conduit sizes and slopes together with the five-year HGL are shown in the profile of Figure 6-1. After the physical design criteria for the initial storm are fulfilled, consideration is given to the hydraulic effects of the major storm on the drainage system.

7.4 Major Storm Considerations

The major storm runoff is routed through the drainage system to determine if the combined capacity of the street and storm sewer system is sufficient to maintain surface flows within permissible limits from Sub-section 3.2.3). The capacity of the conduit at any given point is assumed to be the same for the major storm as for the initial design storm for preliminary design purposes. If the major storm runoff exceeds the combined capacity of the street and storm sewer drainage system, revision in the major drainage design is required.

7.4.1 Example Calculations (Major Design Storm)

Runoff computations for a typical residential area (Figure 6-1) are presented to illustrate the major storm considerations. It is assumed that runoff from other drainage areas is served by other streets and storm sewers, and that "B" Street is designated as a major drainageway.

A. Allowable Street Carrying Capacity

The depth of flow shall not exceed 18 inches at the gutter line for "B" Street (Sub-section 3.2.3). The following design data is stated for the allowable major floodway area shown on Figure 7-1:

Area

$A_1 = 1.26$ square feet

$A_2 = 14.25$ square feet

$A_3 = 10.36$ square feet

$A_1 + A_2 = 15.51$ square feet (half section)

Total $= (A_1 + A_2)$ Area $= 31.02$ square feet

Manning's "n"

Pavement, $n = 0.015$

Lawn, $n = 0.03$

Hydraulic Radius

$$r_{(1+2)} = \frac{A}{WP} = \frac{31.02}{25} = 1.24$$

$$r_3 = \frac{10.36}{13} = 0.80$$

Using Manning's Formula, $V = \frac{1.49}{n} r^{2/3} s^{1/2}$

for slope = 0.0045 ft/ft, $V_{(1+2)} = 7.69$ fps, $Q_{(1+2)} = 238.6$ cfs

$V_3 = 2.87$ fps, $Q_3 = 29.8$ cfs

Therefore, the maximum street capacity is 298.2 cfs.

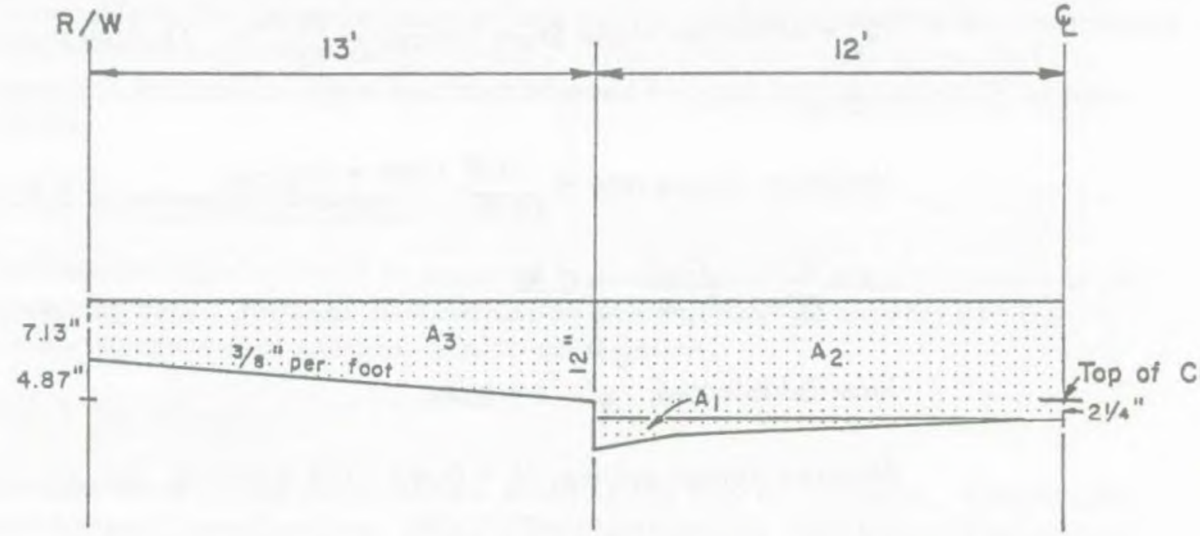
B. Peak Discharge and Total Runoff Volume

Example calculations for determining the major storm runoff of the total drainage area (B Street) are shown in Figure 7-2.

Total runoff volume - 174,240 cf

Peak discharge - 57.6 cfs

The maximum street drainageway capacity is much greater than the major storm peak discharge. Therefore, the actual depth of flow in the street on a continuous slope is within allowable limits.



Half-Section

EXAMPLE MAJOR FLOODWAY AREA

C. Ponding at Sump (Design Point "I")

The street low point actually functions as a structural runoff control facility with the combination inlets and proposed 27 inch storm sewer being the flow control device. The graphical flow routing method discussed in Sub-section 10.8.1 can be used to determine the approximate storage volume.

Calculations are as follows:

Q_i = Major Storm Discharge = 57.6 cfs (Step B)

27" Initial Storm Capacity = 30.0 cfs (Exhibit VII-1)

Q_o = Allowable Major Storm Capacity= $\frac{30.0}{2}$ = 15.0 cfs (Sub-section 3.2.3)

Maximum release rate = $\frac{15.0}{16.0}$ (640) = 600 csm

Ratio $\frac{Q_o}{Q_i} = \frac{15.0}{57.6} = 0.26$

From Exhibit X-6; $V_s / V_r = 0.44$

Required storage volume, $V_s = 0.44 \quad 174{,}240 = 76{,}666$ cf

The maximum storage volume provided at the street sump is approximately 15,000 cf which is less than the required storage volume. Therefore, an overflow channel is provided to carry the excess runoff volume (Figure 6-1).

D. Major Storm Overflow Channel Design

The overflow channel is designed in accordance with Chapter 9 - Open Channels. The design channel discharge is 57.6 - 15.0 = 42.6 cfs.

A triangular channel is selected having a well-maintained Kentucky Bluegrass lining. The maximum permissible velocity is 5 fps (Exhibit IX-4).

Size calculations are as follows:

Required Area= $\frac{Q}{V} = \frac{42.6}{5}$ = 8.5 square feet

Manning's n = 0.05 (Exhibit IX-4)

Maximum d_f = 1.0 feet

Slope = 1.0%

From Exhibit VI-1; $^{z}/_{n} = 900$

$z = 900 \times 0.05 = 45 = {}^{T}/_{d_f}$ (Instruction 2. (Exhibit VI-1)

Required top channel width = 45'

A fifty foot channel easement is provided for the major storm flow.

7.5. Construction Standards

7.5.1 Construction Materials

Storm Sewers (conduits) and appurtenances may be constructed of any suitable material acceptable to the governing body as long as it is capable of meeting the requirements set forth in this drainage design manual. When alternate types of material are proposed, hydraulic designs must be performed for each material to verify acceptability.

7.5.2 Construction Drawings

Construction drawings shall be prepared in accordance with the requirements of the governing body. Prints of all construction drawings shall be submitted to the reviewing agency for approval prior to construction.

7.5.3 Specifications

Standard construction specifications shall be the City of Columbus, "Construction and Material Specifications, 1976." The specifications may be modified to apply to specific projects subject to approval.

Supplementary construction specifications shall be in sufficient detail to guarantee first class material and installation workmanship.

7.6 Bibliography

1. Design and Construction of Sanitary and Storm Sewers, American Society of Civil Engineers - Manuals and Reports on Engineering Practice - No. 37 (Water Pollution Control Federation Manual of Practice No. 9), New York, New York, 1970.

2. Urban Storm Drainage Criteria Manual, Volume 1, Denver Regional Council of Governments, Wright-McLaughlin Engineers, Denver, Colorado, March, 1969.

3. Construction and Material Specifications, City of Columbus, Ohio, 1976.

4. Concrete Pipe Design Manual, American Concrete Pipe Association, Arlington, Virginia, February, 1970.

5. Clay Pipe Engineering Manual, National Clay Pipe Institute, Washington, D.C., 1974.

6. Storm Drainage Design Manual, Erie and Niagara Counties Regional Planning Board, Harza Engineering Co., July, 1972.

7. Policies and Guidelines for the Preparation of Subdivision Plans and Site Development Plans, County of Fairfax, Virginia, January, 1964.

PROJECT:

EXAMPLE STORM SEWER DESIGN COMPUTATIONS

DESIGNER ______

DATE ______

LOCATION:

DESIGN FREQUENCY 5 YEAR n = .0011

INLET LOCATION (DESIGN POINT)	RUNOFF COEFFICIENT C	DRAINAGE AREA A	CA	ΣCA	INLET OR CONDUIT TIME	TIME OF CONCENTRATION T_c	RAINFALL INTENSITY I	RUNOFF = ΣCAI	OTHER CONTROLLED RUNOFF	TOTAL RUNOFF, Q	PIPE LINE DESIGNATION	PIPE DIAMETER	LENGTH, L	SLOPE	VELOCITY HEAD $V^2/2g$	FRICTION SLOPE, S_f OR FLOW DEPTH [d_n]	ELEVATIONS: GUTTER AT INLET	ELEVATIONS: INLET or MANHOLE BOTTOM	PRESSURE ELEVATION CALCULATIONS: ELEVATION DOWNSTREAM SIDE OF INLET or MANHOLE	PRESSURE RISE: CHART NUMBER	PRESSURE RISE: VELOCITY HEAD COEFFICIENT, K	PRESSURE RISE: PRESSURE RISE	ELEVATION UPSTREAM SIDE OF INLET or MANHOLE	PIPE LOSS, h_f = L × S_f	ELEVATION AT NEXT UPSTREAM INLET	VELOCITY, V	INLET NUMBER
		ACRES			MIN.	MIN.	IN/HR	CFS	CFS	CFS		IN.	FT.	FT/FT	FT.							FT		FT.		F.P.S.	
A	0.5	2.8	1.4	1.4	16.2	16.2	3.5	4.9	-	4.9							907.7	902.5	906^{61}	8-6	4.30	1.08	907^{69}				A
					0.7						AB	15	160	.0036	0.25	.0042								0.67	906^{61}	4.0	
B	0.5	1.5	0.75	2.15		16.9	3.5	7.5	-	7.5							906.8	901.6	905^{60}	8-8	1.05	.34	905^{94}				B
					0.7						BC	18	165	.003	0.27	.0036								0.59	905^{60}	4.2	
C	0.5	1.5	0.75	2.90		17.6	3.4	9.9	-	9.9							906.1	900.9	904^{84}	8-8	0.65	.17	905^{01}				C
					0.7						CD	21	165	.007	0.26	.0028								0.46	904^{84}	4.1	
D	0.5	1.5	0.75	3.65		18.3	3.35	12.2	-	12.2							905.3	899.7	904^{03}	8-8	0.9	.35	904^{38}				D
					0.3						DE	21	80	.007	0.39	.0042								0.34	904^{03}	5.0	
E	0.5	0.7	0.35	4.0		18.6	3.35	13.4	-	13.4							905.0	899.2	903^{43}	8-8	0.54	.26	903^{69}				E
					0.9						EF	21	300	.007	0.49	.005								1.50	903^{43}	5.6	
F	0.5	2.8	1.4	5.4		19.5	3.25	17.6	-	17.6							903.6	897.1	901^{03}	8-8	1.10	0.81	901^{93}				F
					0.4						FG	21	155	.007	0.83	.009								1.40	901^{02}	7.3	
G	0.5	1.4	0.7	6.1		19.9	3.2	19.5	-	19.5							902.9	895.7	898^{86}	8-8	0.48	0.76	899^{62}				G
					0.1						GI	24	25	.0054	0.62	[1.6]								0.14	898^{86}	8.2	
I	0.5	0.46	0.23	7.93		20.0	3.2	25.4	-	25.4							902.8	895.2	896^{76}	8-10	K_H = 1.8 K_L = 3.6	1.96 3.92	898.72 900.68				I
											HI	15	25	.0048	0.33	.0054								0.14	900^{82}	4.6	
H	0.5	3.2	1.6	1.6	16.3	16.3	3.5	5.6	-	5.6							902.9	899.4	900^{82}	8-6	4.6	1.52	902^{34}				H
											IJ	27	200	.0068	0.99	[1.4]								1.36	896^{76}	8.0	
J	Outfall Point (Concrete Endwall)										d_n = 1.4'						-	893.8					895.4				J

8. CULTVERTS AND MISCELLANEOUS STRUCTURES

8.1 Introduction

Culverts are used to convey stormwater in an open channel, natural or artificial, through an embankment such as a roadway or railroad embankment. A culvert is actually a closed conduit restriction in the channel, and the designer's concern is to provide a size and shape to carry the required discharge without causing excessive ponding. Other factors include structural adequacy, flood peaks, and overall construction and maintenance costs. This section presents the general hydraulics of conventional culverts only. References are included for other types of culverts and appurtenances.

8.2 Culvert Hydraulics

There are two major types of culvert flow; they are Inlet Control and Outlet Control. For either type of control, different hydraulic factors and formulae are used. Factors influencing the discharge of a culvert are shown in Figure 8-1, and are used to determine the type of control. A culvert flows either part full as an open channel or flowing full as a pressure conduit water main.

Inlet control exists as long as the ability of the culvert to carry the flow exceeds the ability of water to enter through the inlet. Under this condition of flow, the factors influencing discharge are cross sectional area, inlet geometry and the amount of ponding (Headwater) at the entrance. Whenever the flow in a culvert is less than critical depth, the capacity is controlled by the entrance.

Outlet control occurs when the capacity of the inlet exceeds the ability of the conduit to carry the flow. In this situation, all design factors listed in Figure 8-1 must be considered to determine the culvert capacity. The usual case in outlet control is for the conduit to flow full for a major portion of its length. Whenever flow in a culvert is greater than critical depth, it is under outlet control. Outlet control will also exist when the downstream depth (tailwater) submerges the outlet and forces the culvert to flow full.

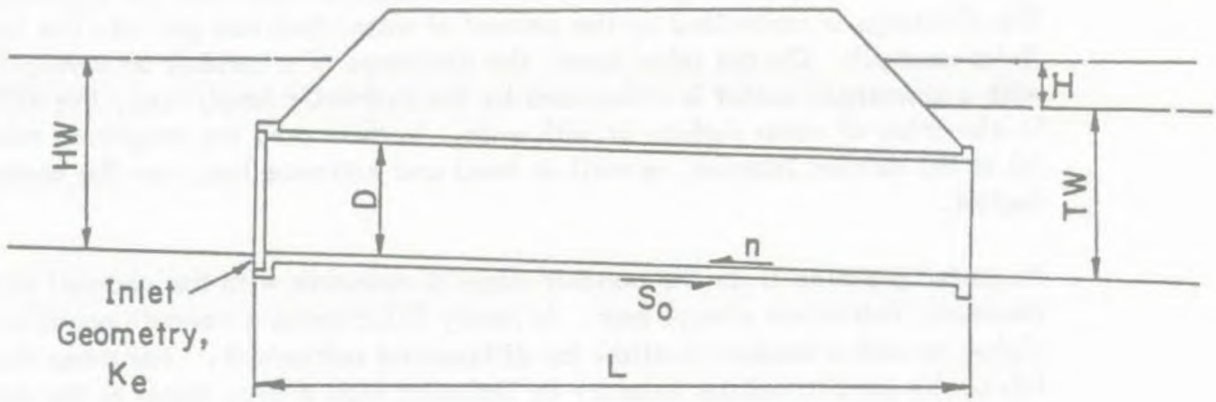
8.2.1 Inlet Geometry

The hydraulic operation of a culvert entrance, which causes the initial flow restriction is the same as for an orifice. Any one of the various inlet control charts illustrates the more efficient the entrance, the greater the discharge of the culvert. By using different types of entrances with different types of conduit material, alternate design configurations can be achieved. By comparing the costs of alternate designs, a final decision can be reached. Over the years many tests involving culvert capacity have demonstrated the importance of inlet geometry on the hydraulic operation of culverts.

The entrance loss coefficient (Ke) is an indication of inlet efficiency. Values of Ke range from 0.2 to 0.9 depending on inlet geometry and the type of conduit material.

FIGURE 8-1

HYDRAULIC FACTORS AFFECTING CULVERT DISCHARGE



D = Height of culvert, in feet
HW = Headwater depth at culvert entrance
L = Length of culvert
n = Surface roughness coefficient
S_o = Slope of the culvert conduit
TW = Tailwater depth at culvert outlet
K_e = Entrance loss coefficient
H = Head loss through culvert

8.2.2 Slope

The ideal slope for a culvert is one that does not produce either silting or excessive velocities and scour. The discharge of a culvert with a free outlet (not submerged) is not increased by placing the conduit on a slope steeper than its critical slope. The discharge is controlled by the amount of water that can get into the inlet (inlet control). On the other hand, the discharge of a conduit on a very flat slope with a submerged outlet is influenced by the hydraulic head; i.e., the difference in elevation of water surface at both ends. In this case, the roughness coefficient (n) of the culvert interior, as well as head and entrance loss, are the controlling factors.

Accepted practice is for the conduit slope to coincide with the channel slope. However, this is not always best. In newly filled areas a conduit could be set higher or with a camber to allow for differential settlement. For steep slopes, drop inlets canbe used to reduce velocity by inducing high energy losses at the entrance.

8.2.3 Velocity

A culvert, because of its hydraulic characteristics, increases the flow velocity beyond that in an open channel. High velocities are most damaging just downstream from the culvert outlet and the erosion potential at this point must be evaluated. Culvert outlet velocity should be compared to the maximum permissible velocity of the downstream channel in determining the need for channel protection.

Outlet velocities for culverts with inlet control may be determined by computing the mean velocity for the culvert cross section using Manning's equation:

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

Since depth of flow is not known, the velocity (V) can be determined by using a table of hyraulic elements for circular sewers and the ratio of flows:

$$\frac{Q\ \text{design}}{Q\ \text{full}}$$

From this ratio ,the corresponding parts full and mean velocity can be found from curves of hyraulic flow conditions for circular conduits. For rectangular or square conduit, the velocity or depth of flow can be determined directly from Manning's Formula by trail and error.

For culverts having outlet control, if the tailwater is greater than the height of the culvert, then V=Q/A, "A" is the full cross sectional area of the culvert.

If tailwater is less than the height of the culvert, then $V = V_c$ or $V = V_{tw}$, whichever is less. V_c is the critical velocity of the culvert. V_{tw} is the velocity of the downstream channel.

8.3 General Design Procedures

There are many design procedures available for determining culvert size. Empirical formulae, while being easy to use, do not consider all the factors influencing flow through a culvert. The mathematical approach, while giving precise results, is time-consuming.

Simplified methods have been developed by the United States Department of Commerce, Bureau of Public Roads. The design procedures are stated in Hydraulic Engineering Circular Number 5(1) and Number 10(2). In Circular Number 5, the design procedure is to determine headwater depths from the charts for both inlet and outlet control. The higher headwater is the control. Circular Number 10 contains a series of hydraulic capacity charts for direct selections of culvert size.

The charts cover nearly all culverts commonly used for highway construction. However, these charts have certain limitations; e.g., headwater depth must be less than twice the height of the culvert and tailwater depth must not be greater than critical depth at the outlet. When these limiting conditions exist, use the design charts in Circular Number 5.

Following is a recommended process for calculating culvert size:

1. List design data (figure 8-1)
   a. Design discharge (Q) in cfs (Q initial and Q major design storm runoff)
   b. Approximate length (L) of culvert, in feet
   c. Culvert slope, in ft./ft. (S_o)
   d. Allowable headwater depth (AHW), in feet
   e. Downstream channel velocity and depth (TW)
   f. Type of culvert, first trial (entrance type, material, and shape)
2. Determine the first trial size by one of the following methods
   a. Arbitrarily select a size (based on engineering experience)
   b. Assume $^{HW}/_{D} = 1.5$, determine size using Exhibit VIII-1.
   c. $^{Q}/_{V} = A$, where V = upstream channel velocity.

If a single conduit culvert size is too large because of limited overhead clearance or availability of size, try alternate design methods such as lowering the invert, drop inlet or multiple conduits. Assume the flow is equally divided among each of the conduits for multi-conduit design of the same size.

3. Assume Inlet Control

a. Using nomographs, find the headwater (HW) depth, for trial size. $HW = \frac{HW}{D} \times D$ (Exhibit VIII-1).

b. If HW is greater than the allowable HW, try a different culvert size. Tailwater conditions are neglected in this part of the procedure.

4. Assume Outlet Control

a. Find H (=HW − H_o) from outlet control nomograph (Exhibit VIII-2).

b. Find Critical Depth of Culvert (Exhibit VIII − 3)

c. Add Culvert Height, D, to Critical Depth, dc, and divide by two.

d. Determine h_o = greater value of $\frac{D + dc}{2}$ or tailwater depth (TW)

e. Compute LS_o (Culvert Length x slope)

f. Compute $HW = H + h_o - LS_o$

g. Compare the headwater values determined for inlet control and outlet control, the higher HW value governs and indicates types of control.

h. If Outlet control governs and HW is greater than the allowable headwater, try a larger conduit. Since Outlet control is the constraint and a smaller size was acceptable for Inlet control, the larger conduit does not have to be checked for Inlet control.

5. Check accuracy of HW value for outlet control.

a. If $HW \geq D + \frac{(1 + Ke)\ V^2}{2g}$, HW is accurate. ($V = Q/A$.

b. If $HW <$ (a) and $> 0.75D$, HW is sufficiently accurate.

c. $HW < 0.75D$, redesign is required.

6. Compute outlet velocity (V_o)

a. If $V_o \leq$ permissible downstream velocity, no channel protection is needed.

b. If $V_o \geq$ permissible downstream velocity, channel protection or energy dissipation is required.

7. Record final design data.

8.4 Example Calculations for Culvert Design

The following example calculations are included to illustrate the general design procedures

1. Given:

Drainage Area = 47 acres = 0.073 square miles

CN = 77

T_c = 0.3 hours

Initial design storm = 5 years

Major design storm = 100 years

2. Discharge:

Peak c.s.m. per inch of runoff = 658	(Exhibit V-13)
5-year storm = 3.30" rainfall	(Chapter 4)
100-year storm = 5.00" rainfall	(Chapter 4)
5-year runoff = 1.28"	(Exhibit V-7)
100-year runoff = 2.62"	(Exhibit V-7)

5-year Q = 1.28 x 0.073 x 658 = 61.48 cfs

100-year Q = 2.62 x 0.073 x 658 = 125.85 cfs

3. Upstream Channel Data: (Chapter 9)

2 foot bottom, 3:1 sideslope, 0.75% slope, n = 0.03, d_f = 0.8 bankfull. (Subsection 3.2.2)

Using Manning's Equation, $Ar^{2/3} = \frac{nQ}{1.49\ S^{1/2}}$

$$Ar^{2/3} = \frac{0.03 \times 61.5}{1.49\ (0.0075)^{1/2}} = 14.30$$

From Table 9-1:

d_f	$A_r^{2/3}$
1.80	13.28
1.86	14.30 (By interpolation), and Area (A) = 14.10 sq. ft.
1.90	15.05

Mean $V = \frac{61.50}{14.10} = 4.36$ fps, which is less than the maximum permissible velocity (8 fps).

Having determined the design data, several trial sizes are tested for inlet control until the inlet control headwater is acceptable before computing the headwater for outlet control.

In this example, a 53"x 34" elliptical pipe is selected as the final design. The culvert size calculations are shown on Exhibit VIII-4.

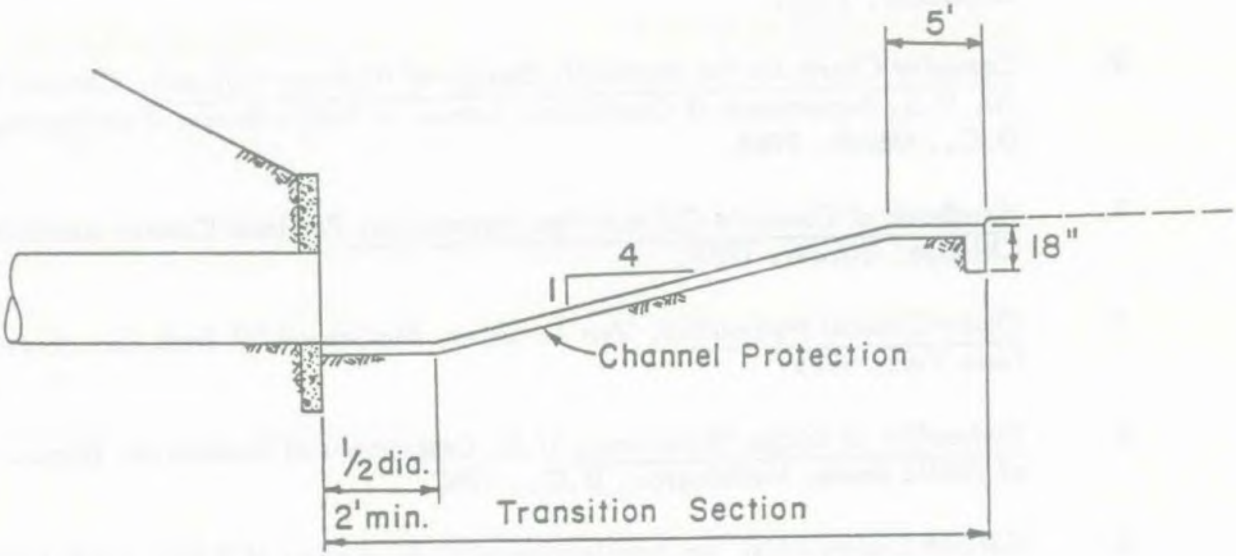
8.5 Alternate Culvert Designs

Sometimes it is necessary to depart from the usual design procedures. For example, such a departure may be necessary when insufficient headwater elevation or low clearance under an embankment condition exists as stated earlier. Alternative design configurations are usually applied when elliptical, pipe, box, or multi-conduit culverts are not acceptable. When the culvert invert is lowered to obtain necessary headwater depth, special consideration must be given to the potential for scouring in the transition area. The use of gabions, riprap, concrete drop structures and special headwall designs can be investigated and compared to obtain an acceptable hydraulic and economic design.

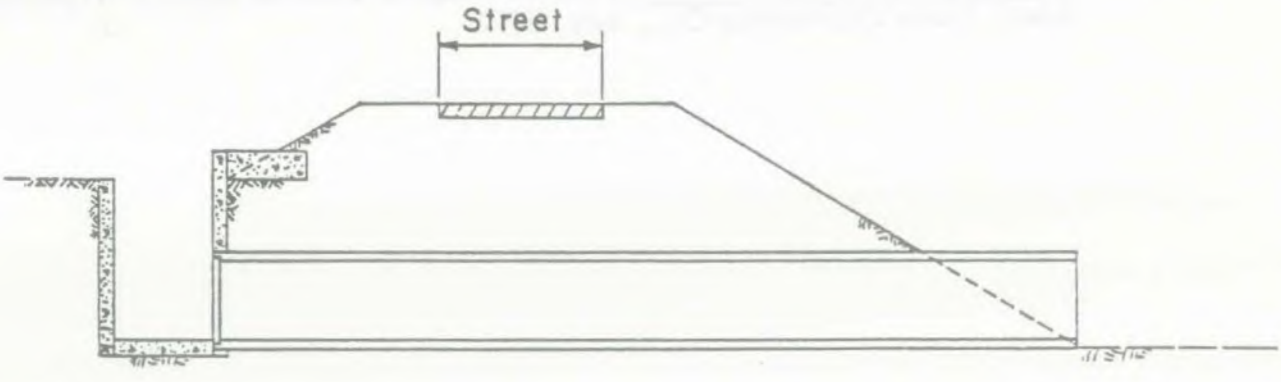
A dropped channel is shown in Figure 8-2A which lowers the culvert invert, and provides more headwater depth at the entrance. Special design consideration is given to the upper end of the transition section to prevent undercutting. Consideration should also be given to the effects of the lower culvert invert on the depth of the downstream channel. Sometimes the extra depth can be compensated by decreasing the slope on culvert conduit and/or part of the channel downstream.

In Figure 8-2B, a drop inlet is shown. Drop inlets are useful when head room at the entrance is limited or when it is desirable to reduce velocity by inducing high energy losses. The design procedures for sag culverts and drop inlets may be found in "Handbook of Concrete Culvert Pipe Hydraulics." (3)

FIGURE 8-2



A - DROPPED CHANNEL



B - DROP INLET

TYPICAL ALTERNATE CULVERT DESIGNS

NO SCALE

8.6. Bibliography

1. Hydraulic Charts for the Selection of Highway Culverts, Circular No. 5, U.S. Department of Commerce, Bureau of Public Roads, Washington, D.C., December, 1965.

2. Capacity Charts for the Hydraulic Design of Highway Culverts, Circular No. 10, U.S. Department of Commerce, Bureau of Public Roads, Washington, D.C., March, 1965.

3. Handbook of Concrete Culvert Pipe Hydraulics, Portland Cement Association, Chicago, Illinois, 1964.

4. Open-Channel Hydraulics, Ven Te Chow, McGraw-Hill Book Co., New York, New York, 1959.

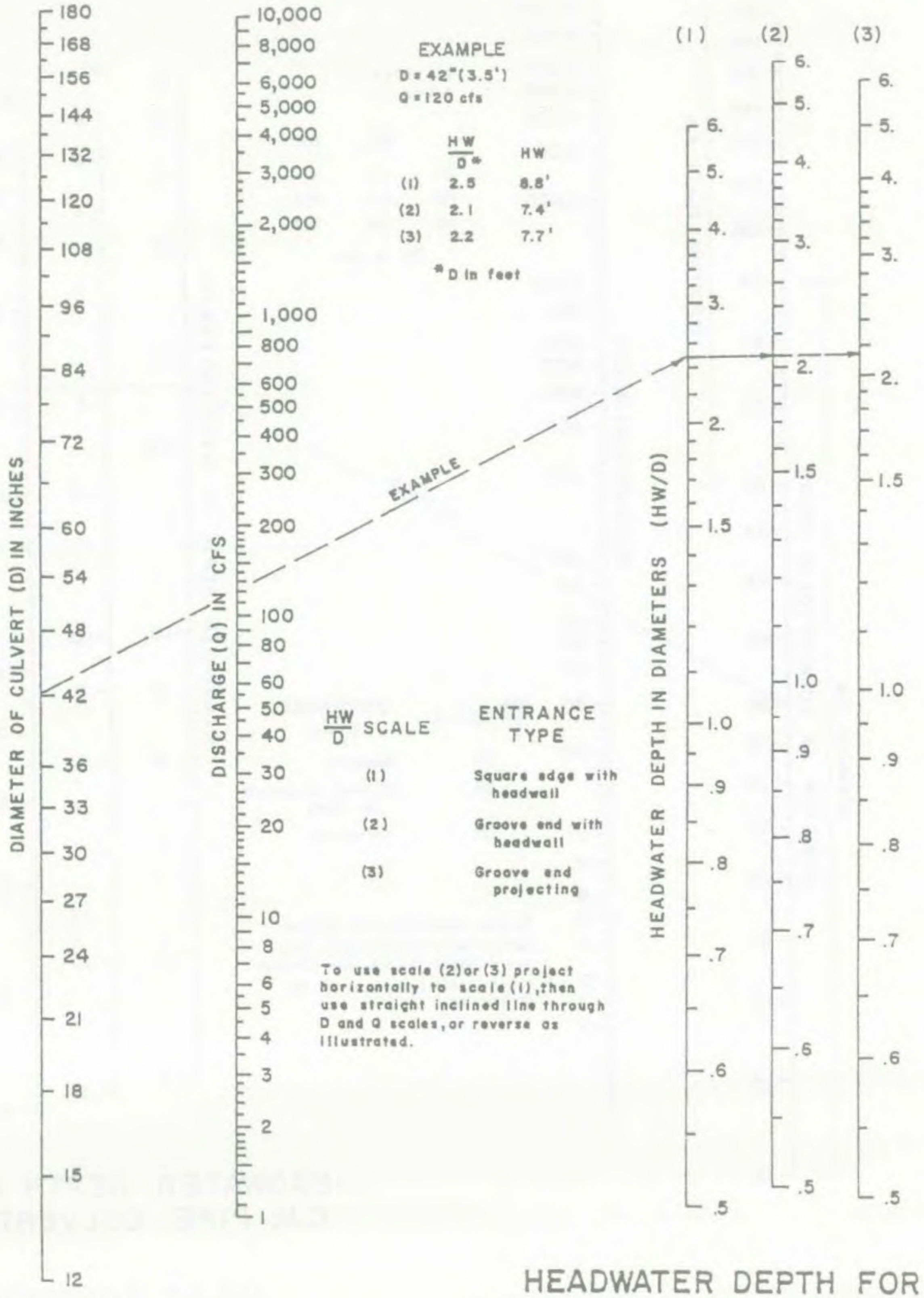
5. Hydraulics of Bridge Waterways, U.S. Department of Commerce, Bureau of Public Roads, Washington, D.C., 1960.

6. Culvert Design Aids: An Application of U.S. Bureau of Public Roads Culvert Capacity Charts, Portland Cement Association, Chicago, Illinois, 1962.

7. Urban Storm Drainage Criteria Manual, Volume 2, Denver Regional Council of Governments, Wright-McLaughlin Engineers, Denver, Colorado, March, 1969.

8. Concrete Pipe Design Manual, American Concrete Pipe Association, Arlington, Virginia, February, 1970.

9. Storm Drainage Design Manual, Erie and Niagara Counties Regional Planning Board, Harza Engineering Co., July, 1972.

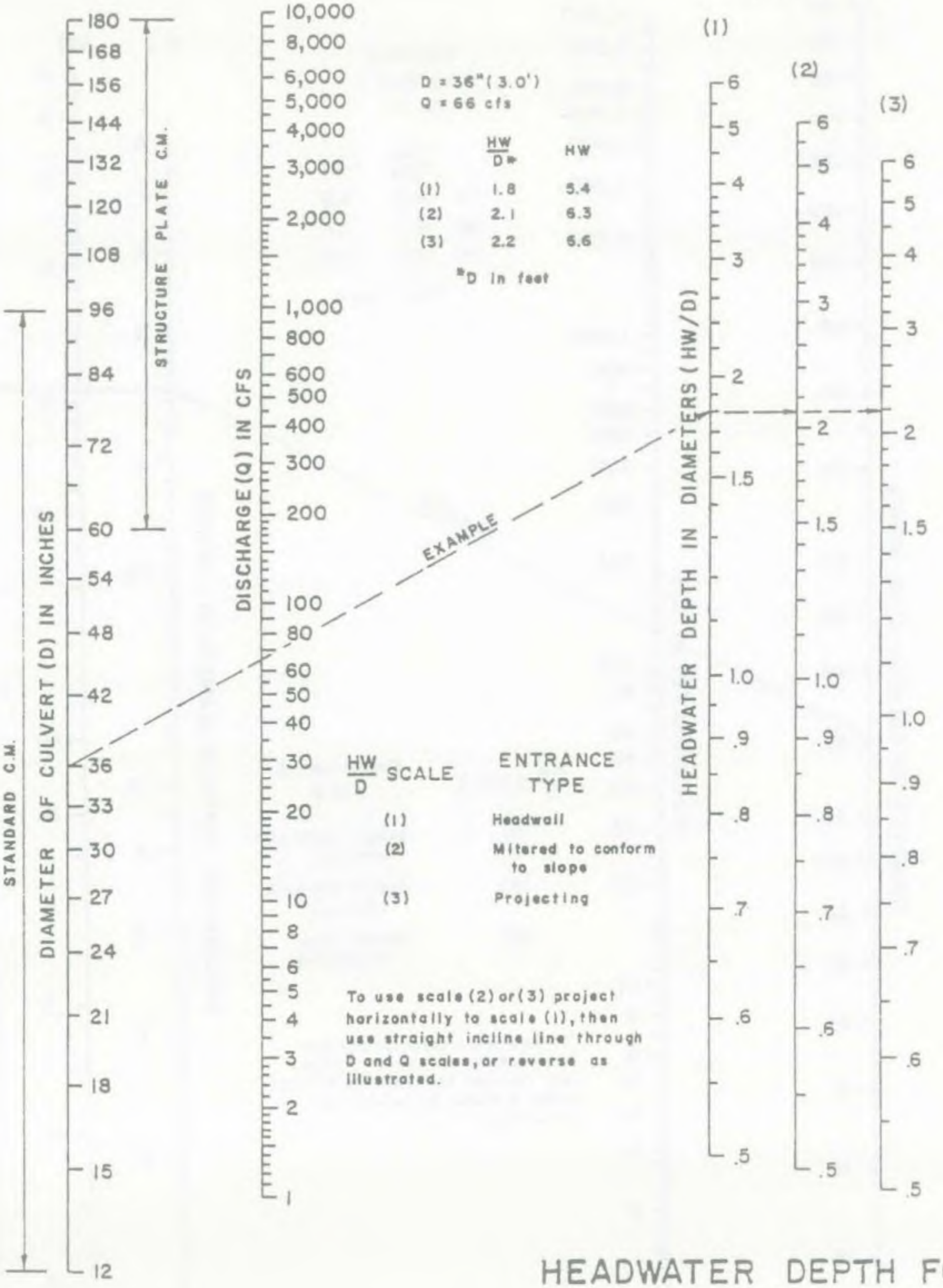


HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS

INLET CONTROL

Source: Bureau of Public Roads, Washington, D.C., December, 1965

EXHIBIT VIII-1
(continued)

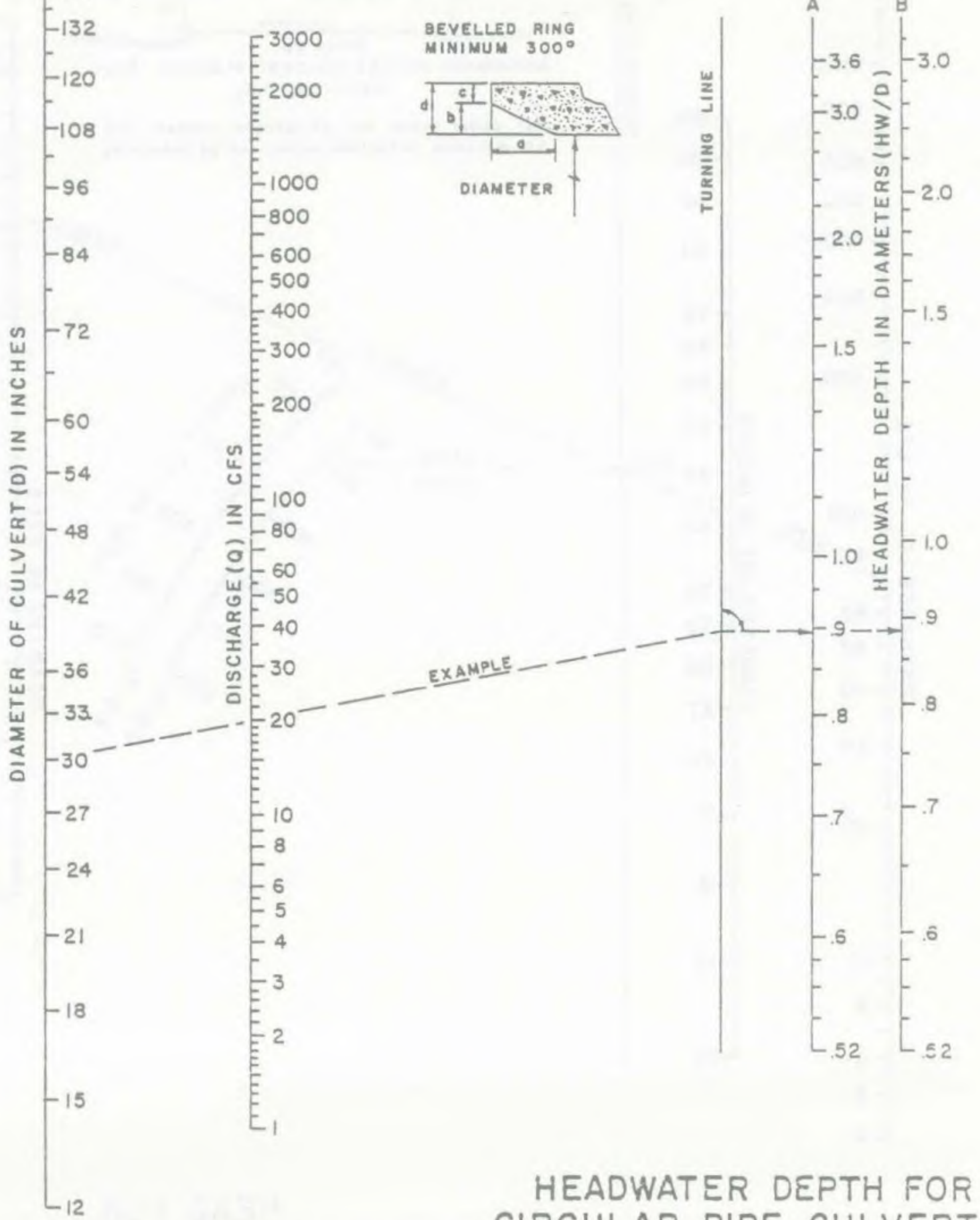


HEADWATER DEPTH FOR C.M. PIPE CULVERTS

INLET CONTROL

Source: Bureau of Public Roads, Washington, D.C., December, 1965

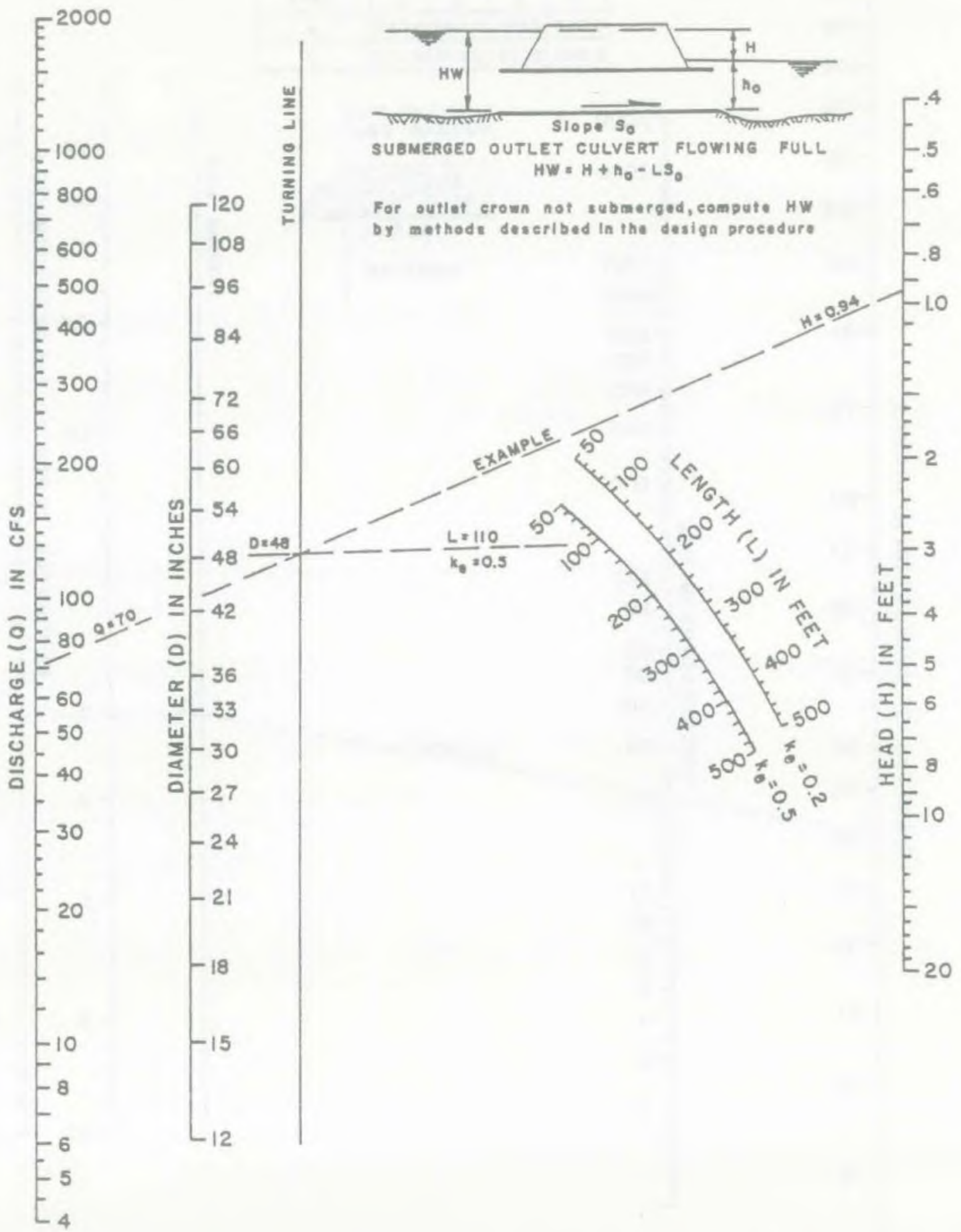
$\frac{b}{D}$	$\frac{a}{D}$	$\frac{c}{D}$	$\frac{d}{D}$	ENTRANCE TYPE
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B



HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELLED RING

INLET CONTROL

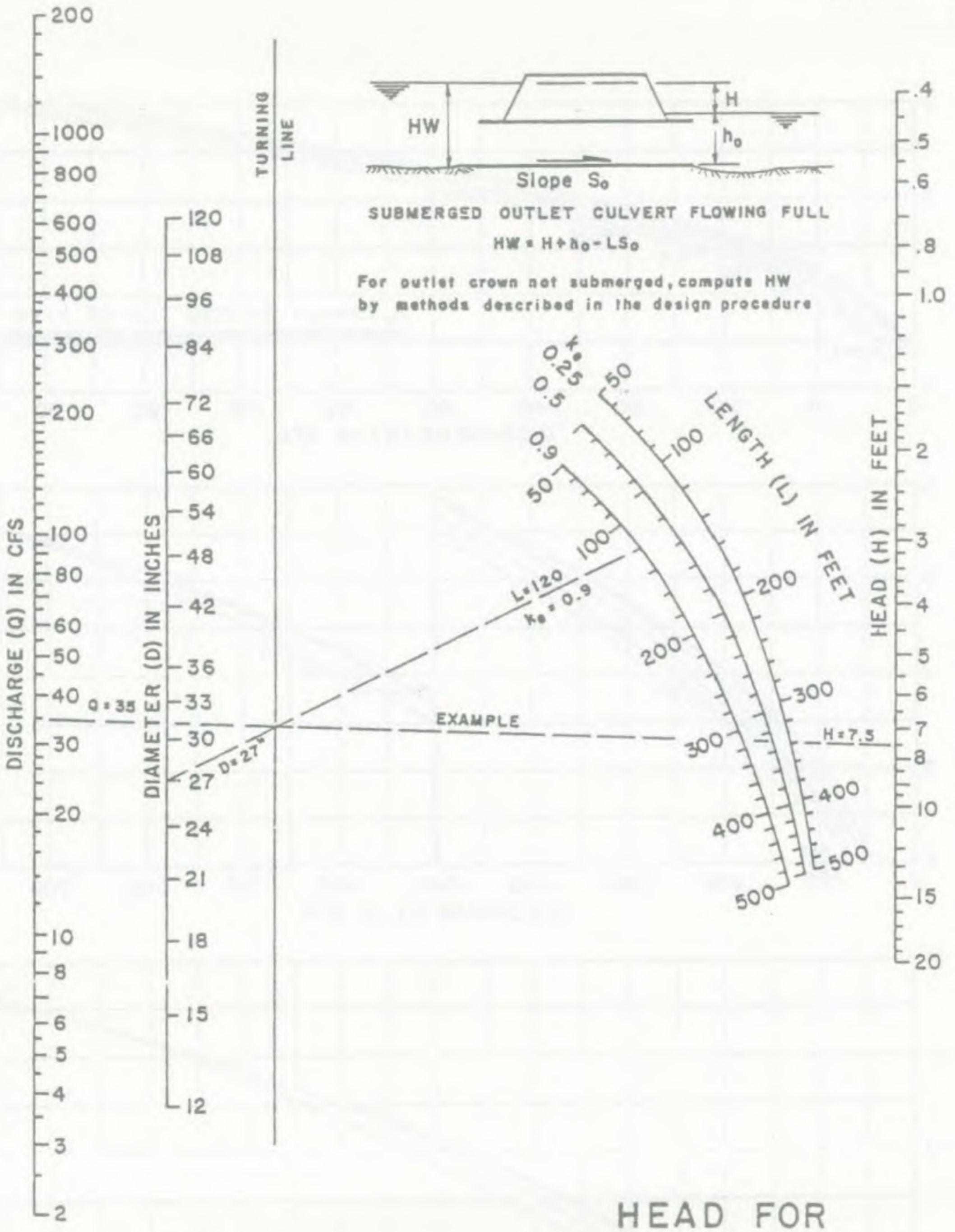
Source: Bureau of Public Roads, Washington, D.C., December, 1965



HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL
n=0.012

OUTLET CONTROL

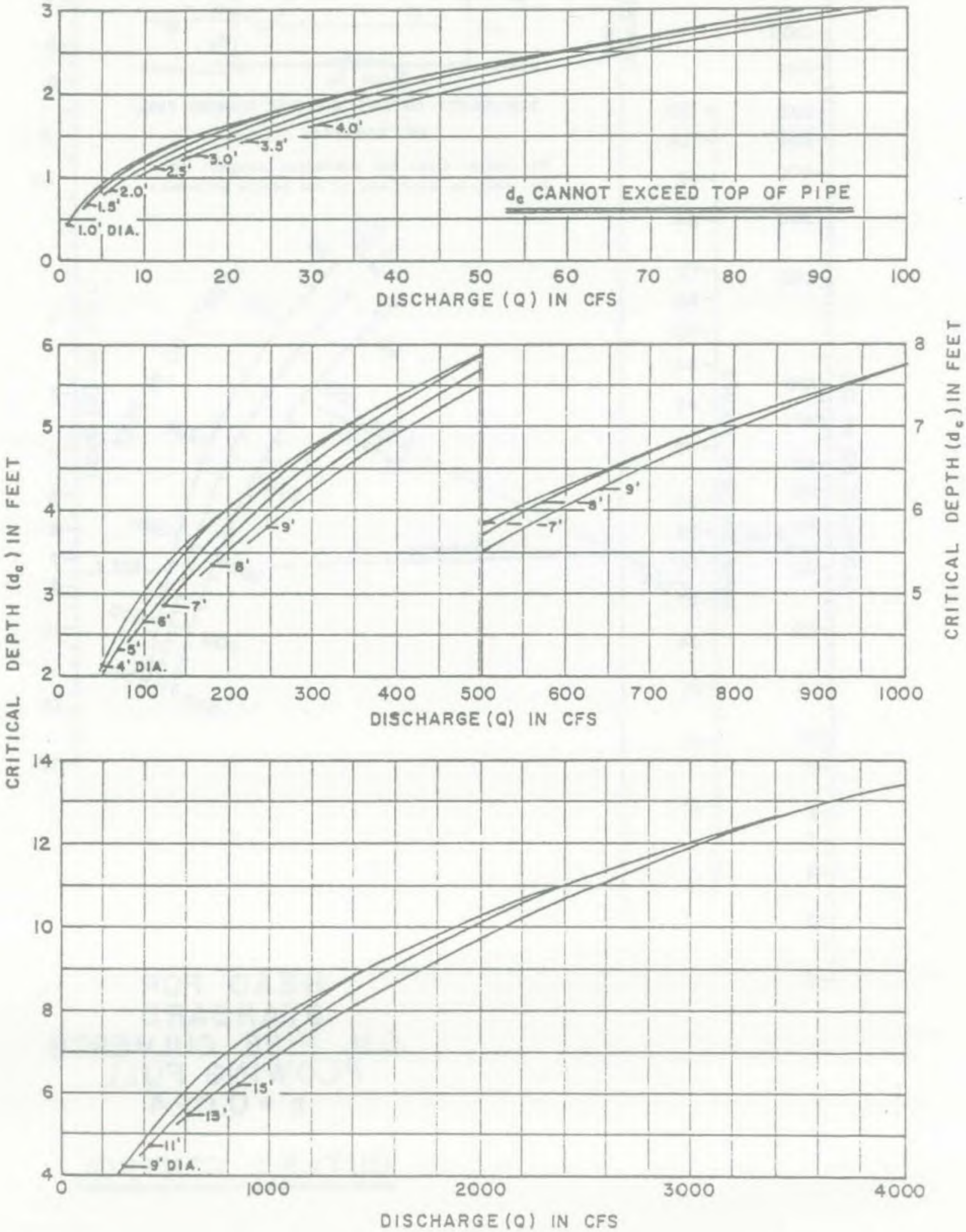
Source: Bureau of Public Roads, Washington, D.C., December, 1965



HEAD FOR STANDARD C.M. PIPE CULVERTS FLOWING FULL
n = 0.024

OUTLET CONTROL

Source: Bureau of Public Roads, Washington, D.C., December, 1965



CRITICAL DEPTH
CIRCULAR PIPE

Source: Bureau of Public Roads, Washington, D.C., December, 1965

CULVERT SIZE DESIGN

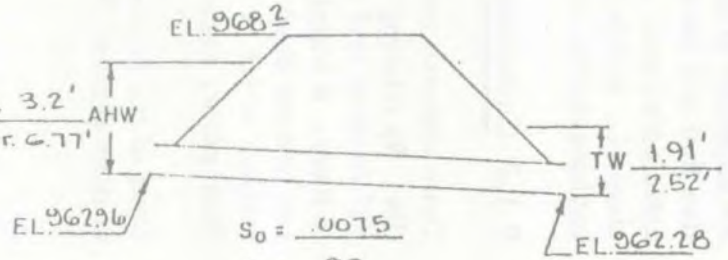
PROJECT:

DESIGNER ______

DATE ______

CULVERT DESCRIPTION (ENTRANCE TYPE)	SIZE	Q	HEADWATER COMPUTATIONS: INLET CONT. $\frac{HW}{D}$	INLET CONT. HW	OUTLET CONTROL $HW = H + h_o - LS_o$: Ke	H	d_c	$\frac{d_c + D}{2}$	TW	h_o	LS_o	HW	Controlling HW	OUTLET CONT. Size OK if HW > $D + \frac{(1+Ke)V^2}{2g}$	.75 D	OUTLET VELOCITY	COMMENTS
Concrete pipe, grooved	36"	61.5	1.5	4.5													AHW exceeded
	42"		1.1	3.9													AHW exceeded
	48"		0.85	3.4													< 48" Limited cover
	53×34	61.5	1.10	3.12	0.2	0.90	2.05	2.44	1.91	2.44	0.68	2.66	3.12	3.56	2.13	6.4	✓ o.k.
	53×34	126	2.20	6.23	0.2	4.0	2.83	2.83	2.52	2.83	0.68	6.15	6.23	9.07	2.13	13.1	✓ o.k.
	TWIN 30	30.8	1.2	3.00	0.2	1.3	1.9	2.20	1.91	2.20	0.68	2.82	3.00	3.23	1.88	8.3	Needs downstream protection
	TWIN 30	63	2.8	7.0	0.2	4.8	2.5	2.5	2.52	2.5	0.68	6.62	7.00	7.67	1.88	12.8	AHW exceeded

SKETCH



S_o = .0075

L = 90

$\frac{L}{100 S_o}$ = 120

DATA

Q_1 = 61.5 TAILWATER ELEV. 964.19 Q_1 = 5 YR. STM.

Q_2 = 126 TAILWATER ELEV. 964.80 Q_2 = 100 YR. STM.

FINAL DESIGN

SIZE 53×34 INLET ELEV 962.96 SKEW 0° HEADWALL ______

COVER 1.99' OUTLET ELEV 962.28 LENGTH 90' SLOPE 0.75%

VELOCITY 6.4 PROTECTION None

EXHIBIT VIII-4

9. OPEN CHANNELS

9.1 Introduction

9.1.1 Advantages and Disadvantages of Open Channel Flow

Open channel design has many advantages in the management and control of stormwater runoff. It provides an opportunity for natural infiltration of stormwater into groundwater supply and probably most important, extends the time of concentration of runoff, helping to maintain the runoff rate nearer to that which exists prior to development. Other advantages of open channels include lower construction cost, opportunities for recreational activities, an aesthetically pleasing rural look, and the capability of being designed with an emergency overflow for major storms.

The opportunities presented by open channel flow must be measured against the problems, real or potential. Stream beds and banks require the use of valuable land. Channels must be maintained to assure their proper functioning and retain their aesthetic quality. And design factors must assure this natural water resource does not present health and safety hazards.

Properly designed and maintained, open channels become an efficient and cost-effective system for controlling and disposing of stormwater runoff. This section of the manual addresses natural streams and man-made channels in small drainage areas. Studies of large streams, being more complex, should be undertaken only with the assistance of a qualified staff of experienced hydrologists and experts in related fields.

An objective of open channel flow design is to determine a channel shape and size that will have sufficient capacity to prevent undue flooding damage during the anticipated peak runoff period and a velocity that does not cause erosion of the channel. In this Chapter, methods of design, erosion control measures and implementation will be discussed.

9.1.2 Guidelines for Evaluating Open Channel Flow

Open channels and swales should harmonize with the natural features of the site. By relating closely to individual lots, the lot owner will help with the maintenance and not be tempted to dispose of grass clippings and other debris in them. If installed in neighborhoods without a strong pride of ownership, they can become depositories for debris. Integrating the open channel into a linear corridor which is appropriately landscaped can help make a channel an aesthetic focal point, encourage care and maintenance and discourage abuse.

Some of the features which should be considered when evaluating the appropriateness of open channel flow are listed as follows:

1) Maximum anticipated discharge

2) Maximum allowable velocity

3) Slope of channel bottom

4) Available area in corridor

5) Ability to drain adjacent lots

6) Type of soil

7) Maintenance

8) Availability of material

9) Area for waste disposal

10) Neighborhood character

11) Green belt and open space requirements

12) Traffic patterns

13) Neighborhood children population

14) Pedestrian traffic

15) Recreational needs

9.2 General Open Channel Design

9.2.1 Types of Flow

Flow in an open channel is steady if the depth of flow is not varying with time, and is unsteady if the depth of flow is varying with time. These two main types of flow, steady and unsteady, can further be defined as being uniform, non-uniform, gradually varied and rapidly varied. Steady uniform flow is the basic type of flow treated in open channel hydraulics. The depth of flow does not change during the time interval under consideration.

To calculate flow in an open channel, the elements of the cross section are needed. They are:

A = the cross-sectional area of flow, in square feet

wp = the wetted perimeter of channel, in feet

r = hydraulic radius of channel, in feet

General formulas for determining elements for various channel shapes are given in Exhibit IX-1.

With a given depth of flow (d_f) in an open channel having a uniform cross section, a mean velocity (V) can be calculated by the Manning equation:

$$V = \frac{1.49}{n} r^{2/3} \quad S^{1/2} \quad \ldots\ldots (1)$$

where,

n = Manning's roughness coefficient

S = slope of channel, feet/foot

V = velocity in feet per second

The discharge (Q) is then stated:

$$Q = AV \quad \ldots\ldots (2)$$

where,

Q = discharge in cubic feet per second

A = cross-sectional area of flow in square feet

The Manning equation is simple to use, and gives a reliable estimate of velocity as long as discharge, channel cross section and slope are constant.

9.2.2 Coefficient of Roughness (n)

The computed discharge for any given channel will only be as reliable as the estimated value of n used in making the computation. The value of n is not a fixed value and varies with the season, and from year to year. Each year, the value of n increases in the spring and summer months as vegetation grows, and diminishes in the fall as vegetation becomes dormant. The annual growth of vegetation, uneven accumulation of sediment in the channel, lodgment of debris, erosion and sloughing of banks, and other factors tend to increase the value of n from year to year until the hydraulic efficiency of the channel is improved by clearing or cleaning out.

All of these factors should be studied and evaluated with respect to type of channel, degree of maintenance, seasonal requirements, season of year design storm occurs, and other considerations before selecting the value of n.

In Exhibit IX-2, values for n have been tabulated to help the designer choose an appropriate value. To help make allowance for the variations in channel roughness, the higher value for n should be used in calculating the discharge capacity.

Because of the erosion effects velocity has on the channel, Exhibits IX-3 and IX-4 have been included to determine the maximum permissible velocity for a channel. In determining velocity, the lower value of n from Exhibit IX-2 should be used.

To illustrate this discussion of n, examine the calculations below using the two values of n in the same channel. The slope is 0.01 feet per foot, the hydraulic radius is 1.50 and the area is 12 square feet. From Exhibit IX-2 for an excavated channel, with a fairly uniform section natural lining of grass and some weeds, (III-B-2) n = 0.025-0.030.

1) $V = \frac{1.49}{0.025} 1.50^{2/3} \; 0.01^{1/2} = 7.81$ fps $\quad Q = 12 \times 7.81 = 93.7$ cfs

2) $V = \frac{1.49}{0.03} 1.50^{2/3} \; 0.01^{1/2} = 6.51$ fps $\quad Q = 12 \times 6.51 = 78.1$ cfs

Thus, the value of n is an important factor to consider in open channel design, the lower value giving the greater and possibly the controlling velocity, the higher giving the lesser and possibly the controlling capacity.

9.3 Critical Flow

Critical flow is the term used in open channel design to define the dividing point between sub-critical (tranquil) and super-critical (turbulent) flow. At this point, there exist certain relationships between specific energy and discharge and specific energy and depth of flow. Specific energy in a channel section is defined as the energy per pound of water at any section of a channel measured with respect to the channel bottom. For a given channel and discharge (Q), the specific energy in a channel section is a function of the depth of flow (d_f) only. The two conditions which describe critical flow are:

1) The discharge is maximum for a given specific energy head.

2) The specific energy head is minimum for a given discharge.

Changes in channel shape, slope, roughness, or alignment can be reflected by a drastic change in depth of flow. The designer should be aware the depth in a given channel section may be influenced by conditions either upstream or downstream, depending on whether the slope is steep (super-critical) or mild (sub-critical).

Uniform flow at or near critical depth is unstable. This results from the fact that the unique relationship between energy head and depth of flow which must exist in critical flow is substantially disturbed by minor changes in energy. Therefore, if any change in slope, roughness or sedimentation occurs, the flow would become wavy caused by appreciable changes in depth.

Because of this unstable flow, channels carrying uniform flow at or near critical depth should not be used unless the situation provides no alternative. In this case, allowance must be made in design for the height of wave generated. Often, when topography restricts the channel slope, the flow can be forced into sub-critical stable or super-critical stable flow by varying the width of the channel.

Super-critical flow is difficult to control because its inertial forces become dominant; so the flow has a high velocity and is usually described as rapid, shooting, and turbulent. Around the Mid-Ohio area, slopes that will produce this type of super-critical flow are uncommon. Changes in the channel slope (roughness, shape, or structures) could entail complex computations. For the design of open channels where super-critical flow could cause problems, "Open-Channel Hydraulics" by Ven Te Chow is a practical and simplified text.

Sub-critical flow is least affected by channel changes. It will require a much larger cross-section, but will be much easier to control than the same quantity of water flowing at a super-critical velocity.

If a choice is possible between the two, consideration will have to be given to amount of land available for channel, alignment, structures, and other channels or drains entering the system. When determining the type of flow, it is advisable to check using both the lower and higher values of n to determine the range.

9.3.1 Determination of Critical Depth

Critical depth depends only on discharge and shape of the channel, and is independent of the slope or channel roughness. Therefore, in any given channel size, there is only one critical depth for a particular discharge. By using Exhibit IX-5, the critical depth can be found for a trapezoidal or rectangular channel.

To compute critical depth by Exhibit IX-5:

1) Determine section factor (Z)

   $Z = Q/\sqrt{g}$, where Q is discharge (cfs) and g is the acceleration of gravity (32.2 fps)

2) Compute value of $Z/b^{2.5}$, where b is the bottom width of the channel in feet.

3) Find value on $Z/b^{2.5}$ scale and intersect it with the curve corresponding to the appropriate side slope (c:1)

4) Intersect this point with the scale for d_c/b (Exhibit IX-5)

5) Multiply this value by the bottom width (b)

An example is included here to illustrate critical depth computations.

9.3.2 Finding Critical Depth: Example 1

Find the critical depth (d_c) for a trapezoidal channel with a bottom width of two feet (b) and side slopes of 2:1, discharging 25 cfs.

1) $Z = Q/\sqrt{g} = 25/5.67 = 4.41$

2) $Z/b^{2.5} = 4.41/5.66 = 0.78$

3) find 0.78 on $Z/b^{2.5}$ scale and intersect with curve C=2

4) corresponding point on d_c/b scale is 0.60

5) $d_c = d_c/b \times b = 0.60 \times 2 = \underline{1.20}$ feet.

Knowing critical depth, critical velocity can be calculated

$$V_c = \frac{Q}{A_c}$$

$$A_c = (b + cd_c)\, d_c = (2 + 2 \times 1.2)\, 1.2 = 5.28 \text{ square feet}$$

$$V_c = \frac{25}{5.28} = 4.73 \text{ fps}$$

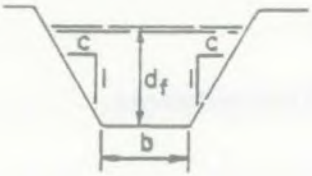
To design open channels using Manning's equation, the channel cross-section must be known as well as the depth of flow (d_f), to determine velocity. The known factors are usually discharge and slope. To use Manning's formula, a method must be used to arrive at the proper channel configuration.

One method is to use channel charts similar to the ones developed by the Bureau of Public Roads [(1)], "Design Charts for Open-Channel Flow," Hydraulic Design Series No. 3.

Knowing the discharge, the slope, and the coefficient of roughness, the depth of flow and velocity can be read directly for a given channel. It is a fairly simple method to use with reasonable accuracy. These charts are based on two to one (2:1) side slopes, and other charts must be developed for side slopes different from 2:1. The process to compute the curves and develop new charts is given in the manual referenced.

[(1)] Can be obtained from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 70402

Table 9-I
Example Trapezoidal Channels Hydraulic Characteristics



$$Ar^{2/3} = \frac{n\,Q}{1.49\sqrt{s}}$$

$$r = A/wp$$

d_f	b = 2, c = 2		b = 2, c = 3		b = 2, c = 4		b = 3, c = 2	
	$Ar^{2/3}$	A	$Ar^{2/3}$	A	$Ar^{2/3}$	A	$Ar^{2/3}$	A
0.5	0.75	1.50	0.85	1.75	0.95	2.00	1.05	2.00
1.0	2.90	4.00	3.56	5.00	4.21	6.00	3.83	5.00
1.1	3.53	4.62	4.38	5.83	5.21	7.04	4.60	5.72
1.2	4.23	5.28	5.30	6.72	6.35	8.16	5.47	6.48
1.3	5.00	5.98	6.33	7.67	7.63	9.36	6.41	7.28
1.4	5.86	6.72	7.48	8.68	9.06	10.64	7.44	8.12
1.5	6.79	7.50	8.74	9.75	10.64	12.00	8.56	9.00
1.6	7.81	8.32	10.13	10.88	12.38	13.44	9.77	9.92
1.7	8.91	9.18	11.64	12.07	14.29	14.96	11.07	10.88
1.8	10.10	10.08	13.28	13.32	16.37	16.56	12.47	11.88
1.9	11.38	11.02	15.05	14.63	18.63	18.24	13.97	12.92
2.0	12.76	12.00	16.97	16.00	21.07	20.00	15.56	14.00
2.1	14.23	13.02	19.03	17.43	23.70	21.84	17.27	15.12
2.2	15.81	14.08	21.23	18.92	26.53	23.76	19.07	16.28
2.3	17.48	15.18	23.59	20.47	29.55	25.76	20.99	17.48
2.4	19.26	16.32	26.10	22.08	32.78	27.84	23.01	18.72
2.5	21.14	17.50	28.77	23.75	36.22	30.00	25.15	20.00

Another method which gives comparable results is by tabulating the product of the area times the two-thirds power of the hydraulic radius ($Ar^{2/3}$) for various depths and areas of the section (see Table 9-I). The usefulness is apparent when this formula is applied:

$$Ar^{2/3} = \frac{nQ}{1.49\sqrt{s}} \quad \text{.......................................(3).}$$

Knowing Q, n and s, we can easily compute $Ar^{2/3}$. Then, by looking in Table 9-I, a channel can be chosen corresponding to a depth of flow. It can be seen by the table that any channel can meet the design flow, but at different depths of flow. The table is a quick method to determine if a particular trapezoidal channel will flow at a desirable depth.

Also, where certain maximum or minimum depth requirements must be met, a channel can be selected that will give the required depth of flow. As one can see, a number of channel sections can readily be tried with reasonable results in a short time (Example 2).

9.3.3 Finding Flow Depth: Example 2

What is the depth of flow in a trapezoidal channel with a two-foot bottom (b) and 3:1 side slope (c = 3) a roughness coefficient of 0.03 and a slope of 0.01/ft./ft. and a discharge of 30 cfs.

1) $Ar^{2/3} = \frac{0.030 \times 30}{1.49\sqrt{.01}} = 6.04$ (Exhibit IX -6)

2) From Table 9-I

$d_f = 1.20 \quad Ar^{2/3} = 5.30$

$d_f = 1.30 \quad Ar^{2/3} = 6.33$

3) By interpolation,

$$d_f = \frac{6.04 - 5.30}{6.33 - 5.30} \times .10 = 0.07$$

4) $d_f = 1.20 + 0.07 = \underline{1.27}$ feet

9.3.4 Finding Flow Depth: Example 3

The following example is included to further illustrate the open channel design procedures stated herein.

Given: A trapezoid channel with a three-foot bottom (b = 3) and 2:1 side slope (c = 2), with slope of 0.01 ft./ft., discharging 41 cfs, with n being based on excavated open channel, straight alignment, with short grass, some weeds.

Calculate: Depth, velocity, and type of flow

Depth Calculations

In Exhibit IX-2, find n = 0.027 (higher value of n)

$$Ar^{2/3} = \frac{0.027 \times 41}{1.49 \times \sqrt{.01}} = 7.43$$

Table 9-I, find depth = 1.4 feet

Velocity Calculations (using lower value n to compute veolocity)

$$Ar^{2/3} = \frac{0.022 \times 41}{1.49 \sqrt{.01}} = 6.05$$

Table 9-I: $Ar^{2/3} = 5.47$, A = 6.48 sq. ft.

$Ar^{2/3} = 6.41$, A = 7.28 sq. ft.

By interpolation:

$$A = \frac{6.05 - 5.47}{6.41 - 5.47} \times (7.28 - 6.48) + 6.48 = 6.97 \text{ sq. ft.}$$

$$V = \frac{41}{6.97} = 5.88 \text{ fps.}$$

This velocity should be checked against maximum permissible velocities in Exhibits IX-3 and IX-4.

Type of Flow Calculations

If $d_c < d_f$ flow is sub-critical, and if $d_c > d_f$ flow is super-critical, $Z = 41/5.67 = 7.23$.

$$Z/b^{2.5} = 7.23/15.59 = 0.46$$

$dc/b = 0.42 \quad d_c = 0.42 \times 3 = 1.26$

From depth calculations, $d_f = 1.4$ feet $1.26 < 1.40$, therefore flow is sub-critical.

Using d_c to find area, A = (3 + 1.26 x 2) 1.26 = 6.96 sq. ft.

$$V_c = \frac{Q}{A} = 41/6.96 = 5.89 \text{ fps}$$

By comparing critical velocity, V_c, to the velocity obtained for lower value n we see that under good maintenance and during the dormant growing season, the channel could flow near critical depth. It would, therefore, seem advisable to alter the elements of the channel to obtain a more stable flow condition. The above computations are summarized in Exhibit IX-7.

9.4 Summary of Open Channel Design Procedure

The following are general procedures for the design of open channels: Numbers in parentheses refer to columns on formatted worksheet.

1) Fill in Q, S, n_Q and n_V (Exhibit IX-2) and V_{max} (Exhibits IX-3 or IX-4)

2) Quantify value of $Ar^{2/3}$ for Discharge (Q) and Velocity (V)

3) Divide Q(1) by V_{max} (12) = minimum area of channel that will flow within limit set by V_{max}.

4) Select channel by $Ar^{2/3}$ for V, with $A \geq A_{min}$.

5) Calculate channel velocity from area derived in Step 4. $V = Q/A$

6) Find d_f from Table 9-1, using $Ar^{2/3}$ for Q

7) Calculate T, $T = b + b + 2\ d_{fc}$

8) Calculate V for discharge

9) Calculate Z, $Z = Q/\sqrt{g}$

10) Calculate $Z/b^{2.5}$

11) Using Exhibit IX-5, find d_c/b

12) Multiply $d_c/b \times b$ for d_c

13) Using d_c, calculate V_c, $V_c = \dfrac{Q}{d_c\,(b + d_c\,C)}$

14) Compare the values in Exhibit IX-7:

	Col.		Col.	
If,	17	$<$	7,	flow is sub-critical for discharge
	17	$>$	7,	flow is super-critical for discharge
	17	=	7,	flow is critical for discharge
	18	$<$	13,	flow is super-critical for velocity

Col.		Col.
18	>	13, flow is sub-critical for velocity
18	=	13, flow is critical for velocity

9.5 Capacity of Natural Channels

Determining the capacity of a natural channel is necessary when that stream will flow through an area where damage can occur when the channel capacity is exceeded. In addition, it is often necessary to determine the effect on channel capacity resulting from cleaning and landscaping the banks to obtain a more stable and aesthetic effect or from modifying the channel structure to obtain a more stable flow condition.

While natural channels are in a constant process of changing both their grade and course, a goal of stormwater management is to prevent acceleration of these changes as the land use of the tributary area changes. Such changes are caused by variations in:

1. Rate or duration of flow
2. Sediment load
3. Channel geometry

It is the change in channel geometry which can most significantly affect the flow capacity of the channel. Determination of flow capacity in a natural channel involves considerable judgment by the design engineer. The results cannot be determined with as great a certainty as for an artificial channel. Variations in the cross-section of the stream, alignment and roughness of the channel, and the changing quantities of flowing water make the determination of capacity an approximation, at best.

The following general procedure is not exact and gives only the bankfull capacity of each reach of stream without consideration of backwater effects. From this can be determined,the rate of flow at which flooding would occur in any one reach and where bottlenecks may occur. The cross-section and profile data obtained can be used in a more sophisticated analysis of the flood profile if such is needed.

If accurate mapping is available, much of the necessary information can be obtained from it; if not, a detailed field survey will be required. In either event, field work will be required to determine the stream cross sections, data on control points and channel roughness.

9.5.1 General Procedure for Determining Capacity

1. From a base line established parallel to the principle course of the stream, obtain sufficient stream cross sections, at right angles to the centerline in each reach, to determine the average cross section. The stream cross section should be taken to sufficient width to include the major storm floodway.

2. Plot the cross sections at a scale that will permit the cross-sectional area and wetted perimeter to be obtained.

3. Plot a profile of the stream along the centerline of flow at design stage. A sufficiently accurate centerline can be determined by inspection. The elevation of the bottom of the channel (except for potholes) and the elevation at which the stream would leave the channel at each bank should be shown.

4. Points of entry of major tributaries, significant changes in grade of the channel bottom, significant change in stream cross section, and bridges or culverts that would obstruct design flow should be used as control points.

5. Divide the profile into reaches between each control point.

6. Draw a preliminary hydraulic grade line on the profile for each reach. The hydraulic gradient should be parallel to the bottom of the channel for design flow conditions with changes in slope occurring at control points. The preliminary hydraulic grade line should be set with flow at bankfull conditions or at an elevation at which flow could occur without damage to abutting land uses.

7. Plot the preliminary hydraulic gradient on each cross section.

8. Determine the area and hydraulic radius of each section under the hydraulic gradient line and average these values for the reach. An alternative method is to superimpose the sections of a reach, using the hydraulic gradient as a common line, and sketch an average section by judgment. Then from this average section, determine the average area and hydraulic radius for the reach. If the latter method is used, the average section sketched can be used for further study as long as the slope of the preliminary hydraulic grade line is not changed.

9. From the area and hydraulic radius so determined, the slope of the hydraulic gradient, and Manning's "n" value of the reach, a preliminary velocity of flow and hydraulic capacity of each reach can be determined.

10. Using the preliminary hydraulic grade for the reach, determined as headwater or backwater conditions, determine capacities of culverts or bridges that may obstruct the channel. The capacities of culverts can be determined by the method outlined in Chapter 8.

After the capacity of the natural stream has been determined, it should be compared with the design storm runoff. Deficiencies in the capacity of any reach should be examined to define an acceptable solution.

TABLE 9 - II

METHODS OF CONTROLLING EROSION IN OPEN CHANNELS

Type of Lining	Advantages	Disadvantages
Portland Cement or Asphalt Concrete	Permanent when properly constructed and maintained	Costly - speeds up runoff
Crushed or Dumped Rock	Allows for natural infiltration and draining of ground	Expensive; eyesore when not maintained; hard to place
Gabions	Permanent - no limit to range of flow - allows for natural infiltration - aesthetically pleasing	Expensive; hard to place
Drop Structures	Does not speed up runoff; controls velocities satisfactorily	Design problem; maintenance problems; can be unsightly, costly
Riprap	Permanent when properly placed and maintained	Expensive; hard to place

First, the culverts or bridges should be reviewed to determine the headwater that will result from the design storm. If this will not result in damage, change the hydraulic gradient of the upstream reach and recalculate this channel capacity with the backwater from the structure. If the backwater will result in flooding and damage, consideration could be given to increasing the size of the structure.

Next, if a few reaches of the stream channel do not have sufficient capacity to carry the design storm, they should be reviewed to determine the cause. Deficiencies along particular reaches may be eliminated without adversely affecting the entire stream.

9.6 Control of Channel Erosion

Runoff flows in open channels may cause accelerated erosion. Such erosion can be controlled by limiting velocities, changing the channel lining, and reshaping the channel to spread flow.

Ideally, open channels should have neither excessive erosion nor deposition of sediments, although the occurrence of both in moderate amounts is natural and unavoidable. Design velocities should generally be greater than 1.5 fps to avoid excessive deposition. The maximum allowable design velocities for bare channels are given in Exhibit IX-3, and for channels lined with vegetation, Exhibit IX-4. Other methods of controlling velocity or protecting the channel when allowable velocities for turf channels are exceeded are given in Table 9-II.

Vegetation should be established immediately following construction. Sodding can provide immediate protection from minor storms. Jute mats can also help establish seed growth. Seed and fertilizer should be applied to all areas disturbed by construction within 24 hours. This will take advantage of the available soil moisture in getting the seeding established. If seeding is not applied within 24 hours during the growing season, of if construction is done during the dormant season, a mulch should be applied after seeding to control moisture. Temporary seeding is recommended when soil is wet and spreading will be delayed.

Quite often during the first year of construction, or until vegetation is established, the channel bottom is covered with several inches of sediment. Also, deposits frequently occur on the inside of curves, where laterals and surface water enter the channel, above culverts, and near upper and lower ends of construction work. Extra channel depth or capacity can usually be added at selected locations during construction at a cost less than that of removing the sediment.

Consideration should be given to anticipated maintenance methods prior to specifying side slopes. Side slopes should be flat enough for soil stability, erosion control, and the maintenance equipment to be used. For mowing with conventional equipment, side slopes of 3:1 are the steepest recommended. Side slopes of 4:1 are desirable for wheeled tractors. Special equipment can be used on steep or long slopes to reduce disturbance of channel banks.

The point of channel entry by sub-surface storm sewers should also be protected against erosion. In most cases, riprap, dumped rock, or end walls should be used, or some combination of all methods. Backfill should be carefully placed and tamped.

9.7 Bibliography

1. Design Charts for Open-Channel Flow, U.S. Department of Commerce, Bureau of Public Roads, Washington, D.C., August, 1961.

2. Design of Roadside Drainage Channels, U.S. Department of Commerce, Bureau of Public Roads, Washington, D.C., May, 1965.

3. Open-Channel Hydraulics, Ven Te Chow, McGraw-Hill Book Company, New York, New York, 1959.

4. Handbook of Applied Hydrology, Ven Te Chow, Editor-in-Chief, McGraw-Hill Book Company, New York, New York, 1964.

5. Engineering Field Manual for Conservation Practices, U.S. Department of Agriculture, Soil Conservation Service, 1969.

6. Urban Storm Drainage Criteria Manual, Volume 2, Denver Regional Council of Governments, Wright-McLaughlin Engineers, Denver, Colorado, March, 1969.

7. Storm Drainage Design Manual, Erie and Niagara Counties Regional Planning Board, Harza Engineering Co., July, 1972.

8. Storm Water Management Design: A Manual of Procedures and Guidelines. Maryland Department of Natural Resources, Roy F. Weston, Inc., April, 1976.

9. "A Sensible Alternative to Stream Channelization", by E. A. Keller and E. K. Hoffman, Public Works, October 1976, pp. 70-72.

OPEN CHANNEL FLOW SYMBOLS

Symbol	Units	Description
A	sq.ft.	Area of cross section of flow
b	ft	Bottom width of trapezoidal channel
c		Side slope of channel, c:1
D	ft.	Hydraulic depth
d_c	ft.	Critical depth
d_f	ft.	Depth of flow
d_m	ft.	Mean depth of flow (A/T)
F		Froude number $= V/\sqrt{g d_m}$
g	ft/sec^2	Acceleration of gravity = 32.2
K		Channel conveyance $= 1.49/n \ Ar^{2/3}$
L	ft.	Length of channel reach
n		Manning roughness coefficient
Q	c.f.s.	Rate of discharge
r	ft.	Hydraulic radius = A/wp
s	ft./ft.	Slope of channel
s_c	ft./ft.	Critical slope
T	ft.	Top width of water surface in a channel
V	fps	Mean velocity of flow
V_c	fps	Critical velocity
wp	ft.	Wetted perimeter - length of line of contact between the flowing water and the channel
Z		Section factor

PRINCIPLE EQUATIONS

$$V = \frac{1.49}{n} r^{2/3} s^{1/2} \qquad Q = AV \qquad Q = \frac{1.49}{n} A r^{2/3} s^{1/2} \qquad Z = Q/\sqrt{g}$$

Trapezoid

$$A = (b + c d_f) d_f$$

$$wp = b + 2(d_f^2 + c d_f^2)^{0.5}$$

$$T = b + 2 c d_f$$

$$D = \frac{(b + c d_f) d_f}{b + 2 c d_f}$$

Triangle

$$A = c d_f^2$$

$$wp = 2(d_f^2 + c d_f^2)^{0.5}$$

$$T = 2 c d_f$$

$$D = 1/2 d_f$$

EXHIBIT IX-2

MANNING ROUGHNESS COEFFICIENTS, n[1]

	Manning's n Range
I. Closed conduits:	
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):	
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 in. corrugation (field bolted)	0.03
C. Vitrified clay pipe	0.012-0.014
D. Cast-iron pipe, 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	0.012-0.014
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
II. Open channels, lined (straight alinement)[2]	
A. Concrete, with surfaces as indicated:	
1. Formed, no finish	0.013-0.017
2. Trowel finish	0.012-0.014
3. Float finish	0.013-0.015
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, side 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

	Manning's n Range
III. Open channels, excavated (straight alinement[2], natural lining):	
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
IV. Highway channels and swales with maintained vegetation (values shown are for velocities of 2 and 6 f.p.s.):	
A. Depth of flow up to 0.7 foot:	
1. Bermudagrass, Kentucky bluegrass, buffalograss:	
a. Mowed to 2 inches	0.07-0.045
b. Length 4-6 inches	0.09-0.05
2. Good stand, any grass:	
a. Length about 12 inches	0.18-0.09
b. Length about 24 inches	0.30-0.15
3. Fair stand, any grass:	
a. Length about 12 inches	0.14-0.08
b. Length about 24 inches	0.25-0.13
B. Depth of flow 0.7-1.5 feet:	
1. Bermudagrass, Kentucky bluegrass, buffalograss:	
a. Mowed to 2 inches	0.05-0.035
b. Length 4 to 6 inches	0.06-0.04
2. Good stand, any grass:	
a. Length about 12 inches	0.12-0.07
b. Length about 24 inches	0.20-0.10
3. Fair stand, any grass:	
a. Length about 12 inches	0.10-0.06
b. Length about 24 inches	0.17-0.09

[1] Estimates are by Bureau of Public Roads unless otherwise noted.

[2] 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 additional loss of energy.

[3] 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.

MANNING ROUGHNESS COEFFICIENTS, n[1]

	Manning's n Range
V. Street and expressway gutters:	
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
VI. Natural stream channels:	
A. Minor streams (surface width at flood stage less than 100 ft.):	
1. Fairly regular section:	
a. Some grass and weeds, little or no brush................	0.030-0.035
b. Dense growth of weeds, depth of flow materially greater than weed height...........	0.035-0.005
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 trees within channel, with branches submerged at high stage, increase all above values 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:	
a. Winter...........................	0.05-0.06
b. Summer...........................	0.06-0.08
5. Medium to dense brush:	
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 trees, little undergrowth:	
a. Flood depth below branches.......	0.10-0.12
b. Flood depth reaches branches.....	0.12-0.16
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. Values of n may be somewhat reduced. The value of n for larger streams of most regular section, with no boulders or brush, may be in the range of............................	0.028-0.033

PERMISSIBLE VELOCITIES FOR CHANNELS WITH ERODIBLE LININGS, BASED ON UNIFORM FLOW IN CONTINUOUSLY WET, AGED CHANNELS[1]

Soil type or lining (earth; no vegetation)	Maximum permissible velocities for: Clear water	Water carrying fine silts	Water carrying sand and gravel
	(f.p.s.)	(f.p.s.)	(f.p.s.)
Fine sand (noncolloidal)	1.5	2.5	1.5
Sand loam (noncolloidal)	1.7	2.5	2.0
Silt loam (noncolloidal)	2.0	3.0	2.0
Ordinary firm loam	2.5	3.5	2.2
Volcanic ash	2.5	3.5	2.0
Fine gravel	2.5	5.0	3.7
Stiff clay (very colloidal)	3.7	5.0	3.0
Graded, loam to cobbles (noncolloidal)	3.7	5.0	5.0
Graded, silt to cobbles (colloidal)	4.0	5.5	5.0
Alluvial silts (noncolloidal)	2.0	3.5	2.0
Alluvial silts (colloidal)	3.7	5.0	3.0
Coarse gravel (noncolloidal)	4.0	6.0	6.5
Cobbles and shingles	5.0	5.5	6.5
Shales and hard pans	6.0	6.0	5.0

[1]As recommended by Special Committee on Irrigation Research, American Society of Civil Engineers, 1926.

Source: Design Charts for Open Channel Flow, U.S. Department of Commerce, Bureau of Public Roads

PERMISSIBLE VELOCITIES FOR CHANNELS LINED WITH UNIFORM STAND OF VARIOUS GRASS COVERS, WELL MAINTAINED

Cover	Slope[2] range	Permissible[1] velocity on: Erosion resistant soils	Easily eroded soils
Kentucky bluegrass	0-5	7	5
Smooth brome	5-10	6	4
Tall Fescue	Over 10	5	3
Grass mixture[2]	0-5 5-10	5 4	4 3
Lespedeza sericea			
Weeping lovegrass			
Red Fescue			
Red top	0-5	3.5	2.5
Alfafa			
Crabgrass			
Common lespedeza[4]			
Sudangrass[4]	0-5	3.5	2.5

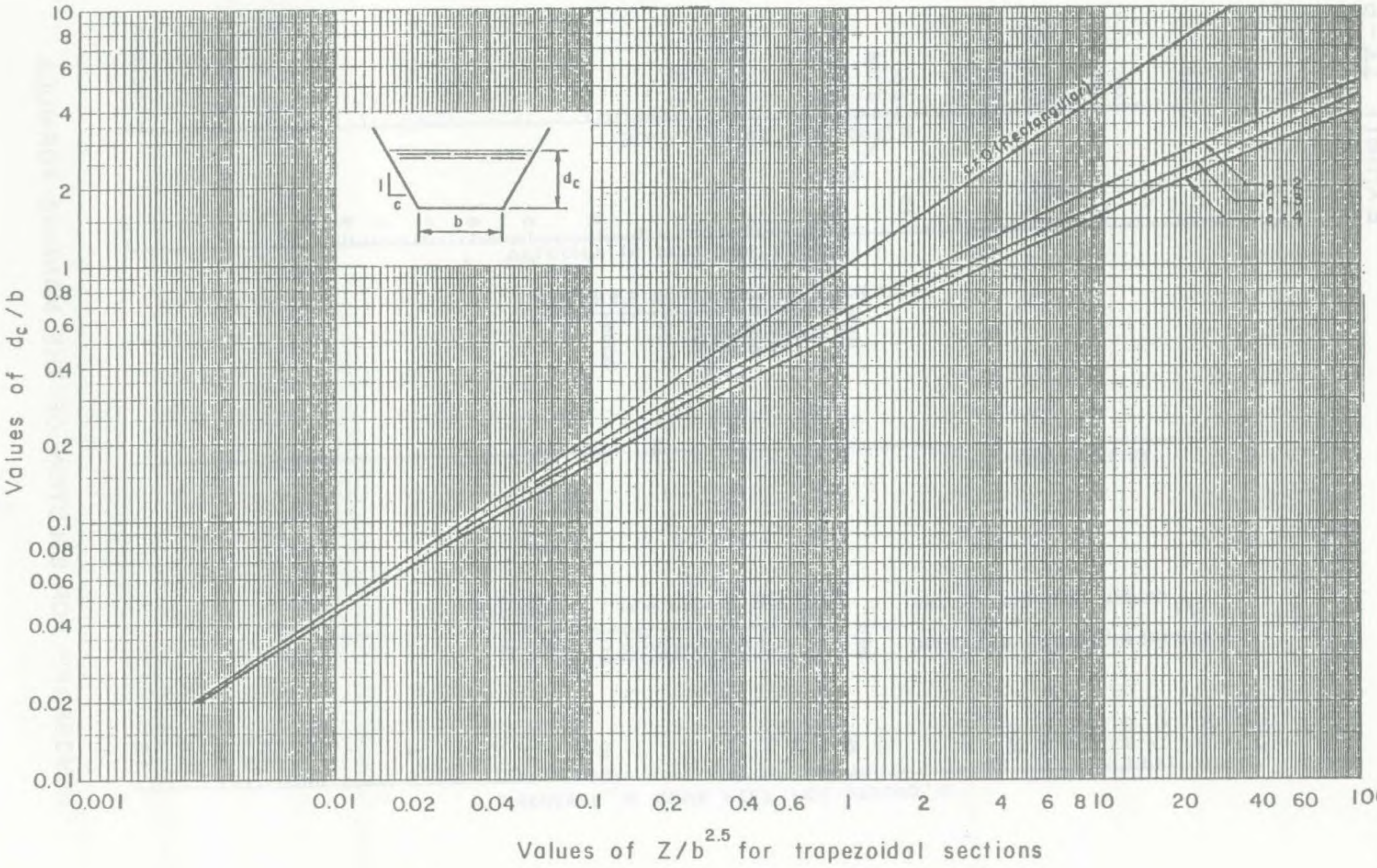
[1]Use velocities exceeding 5 feet per second only where good covers and proper maintenance can be obtained. Ersoion resistant soils include those listed in Exhibit IX-3 with a maximum velocity of 5 feet per second.

[2]Do not use on slopes steeper than 10 percent except for vegetated side slopes in combination with a stone, concrete, or highly resistant vegetative center section.

[3]Do not use on slopes steeper than 5 percent except for vegetated side slopes in combination with a stone, concrete, or highly resistant vegetation center section.

[4]Annuals - use on mild slopes, less than 5 percent, or as temporary protection until permanent covers are established.

Reference: Soil Conservation Service Engineering Field Manual, pp. 7-14.

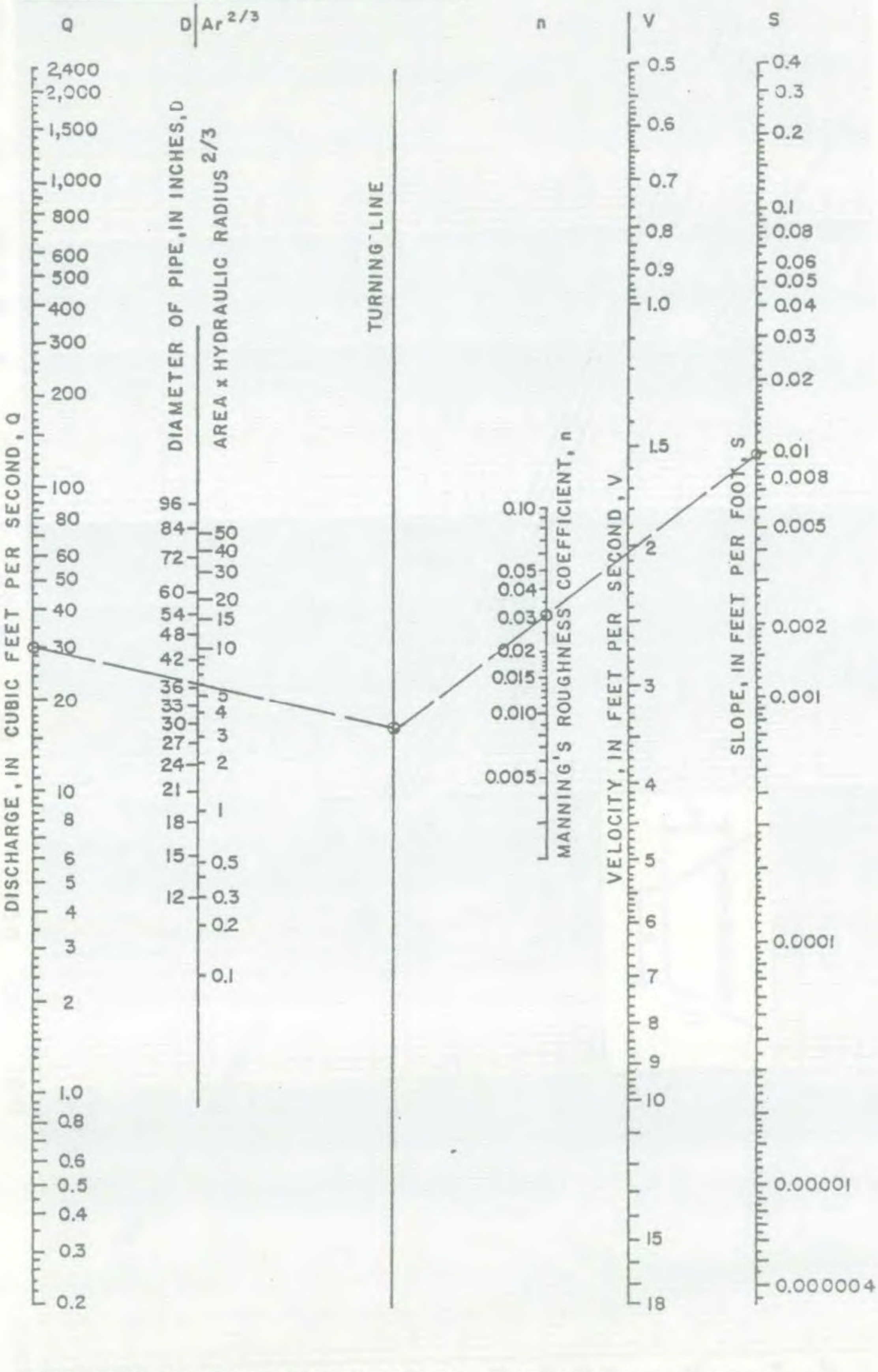


Curves for Determining Critical Depth

SOU[illegible] . "Open Channel Hydraulics", V.T. Chow

EXHIBIT IX-5

Solution of $Ar^{2/3}$ for Example 2 ;



NOMOGRAPH FOR SOLUTION OF THE MANNING FORMULA

EXAMPLE 3: OPEN CHANNEL COMPUTATIONS

DESIGNER ________

PROJECT ____________________ DATE ________

	DISCHARGE								VELOCITY					CRITICAL FLOW				
	1 Q	2 n_Q	3 s	4 $Ar^{2/3}$	5 b,c	6 T	7 d_f	8 V	9 n_V	10 $Ar^{2/3}$	11 A	12 $V_{max.}$	13 V	14 Z	15 $Z/b^{2.5}$	16 d_c/b	17 d_c	18 V_c
1	41	0.027	0.01						0.022			6						
2				7.43						6.05								
4					3,2													
5											6.97		5.88					
6							1.40											
7						8.6												
8								5.05										
9														7.23				
10															0.46			
11																0.42		
12																	1.26	
13																		5.89

$Ar^{2/3} = nQ/1.49\sqrt{s}$ $V = Q/A$ $Z = Q/\sqrt{g}$

EXHIBIT IX-7

10. RUNOFF CONTROL METHODS

10.1 Introduction

The runoff control criteria stated in Chapter 3 necessitates the use of stormwater runoff control facilities in many development situations. While the success of such facilities for accomplishing a desirable level of runoff control cannot be denied, it is often found these same facilities have the potential for adding to neighborhood blight or a threat to public health and safety.

It is not necessary that the "cure" be worse than the "disease." Stormwater storage facilities can be functional and wholly unobtrusive. If desired, or in some cases, their presence can offer an added amenity to the urban environment. This positive impact can be achieved by adherence to four basic steps in the implementation of stormwater storage facilities. These are:

1. Proper selection of runoff control mechanism
2. Proper design of facility
3. Construction of facility in strict adherence to design
4. Regular maintenance program and designated responsibility for maintenance.

This chapter discusses these steps which result in a rewarding and often cost-saving approach to stormwater management.

10.2 Structural and Nonstructural Approaches to Runoff Control

The runoff control requirements specified in Chapter 3 may be achieved in a variety of ways. A basic differentiation is the structural runoff control mechanism versus a nonstructural approach.

10.2.1 Nonstructural Approach to Runoff Control

The nonstructural approach relies on land use management techniques and site design methods to reduce the impact of accelerated storm runoff or to minimize the quantity of storm runoff. Such approaches may be broadly classified as watershed management.

Watershed management is directed at land use in areas outside the flood plain but within naturally occurring, geographically defined drainage areas. Watershed management emphasizes control of storm-water close to its source to reduce undesirable impacts on lower, servient lands.

Watershed management is not a euphemism for land use control. Such control is both unrealistic and unnecessary. However, the rationale of considering runoff impact relationships is valid. Negative runoff impacts can be minimized by orienting roads, buildings, and open space to the unique topographic and drainage characteristics of a site. Chapter 2, Planning for Stormwater Management, considers in greater detail non-structural approaches to runoff control.

10.2.2 Structural Approaches to Runoff Control

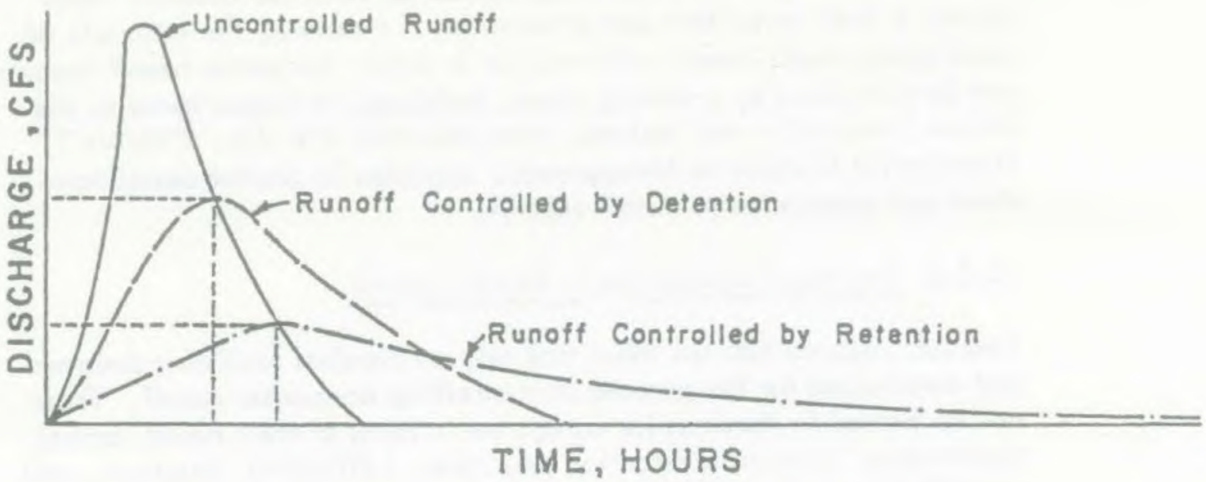
Structural approaches are those that rely on physical facilities designed and constructed for the purpose of controlling stormwater runoff. These can be placed in three major categories in terms of their runoff control mechanisms: detention/retention structures, infiltration structures, and conduit structures. While both structural and nonstructural approaches are effective tools for stormwater management, the most effective runoff control will result when both approaches are considered and applied collectively. Chapter 10 confines itself to a more detailed discussion of structural solutions to stormwater runoff control.

10.3 Detention/Retention Structures

Structures for detention or retention of stormwater may be considered together since the major control structures function the same for each. The principal difference is that while the control mechanism for detention or dry basin may be entirely passive, there should be some manner of movable gate provided in a retention or wet basin so that it can be drained for occasional maintenance.

The results of applying detention or retention storage methods for peak discharge control are illustrated by the typical hydrographs shown on the following page. The objective of both methods is to reduce peak rate of discharge by storage and controlled release. The difference in discharge rates shown is a function of the control devices discussed in Subsection 10.8 rather than the type of structure.

Infiltration of the stored stormwater will occur to some degree depending on underlying soil characteristics. However, in most cases, the amount of infiltration is minimal and is not considered as a part of the structural design. Thus, the total area under the hydrographs representing total volume of runoff will be substantially the same for all three cases.



TYPICAL DETENTION AND RETENTION HYDROGRAPHS

Land requirements are associated with any storage facilities. The demand for land is a major economic factor. For this reason, consideration of the multi-use concept for either wet or dry detention structures is strongly encouraged. Some types of detention and retention structures are discussed.

Detention and retention structures can be categorized as: dry basins, permanent (wet) ponds, storage tanks, and multi-use storage areas such as rooftops, parking lots, roadway embankments and other shallow holding areas. Design criteria of the various structural types are discussed in Sub-section 10.4. A general discussion of the specific types of detention and retention storage follows.

10.3.1 Dry Basins

Dry basins are surface storage areas created by constructing a typical excavated or embankment basin. There is no normal pool level and a specific controlled release feature is included to control the rate of discharge. The detention flow control structure is usually a multi-stage device, and the retention flow control structure is usually a single-stage device.

Dry basins are the most widely used structural method of stormwater management. The soil permeability and water storage potential are not as important with dry basins as with wet ponds. Therefore, dry basins have the greatest potential for broad applications. They can be utilized in small developments because they can be designed and constructed as small structures. Dry basins are often less costly than wet ponds because they do not require extensive design and construction considerations. They can be designed for multi-use purposes such as recreation and parks.

10.3.2 Wet Ponds

Wet ponds are permanent ponds where additional storage capacity is provided above the normal water level and special features for controlled release are included. Historically, wet ponds have proven extremely effective in abating increased runoff and channel erosion from urbanized areas. They are a major Soil Conservation Service land treatment practice.

Some problems often encountered with wet ponds are: site reservation (land requirements), permanent easements, complexity of design and construction, safety hazards and maintenance problems. Because of large land requirements, and the necessity of maintaining a permanent pool of water, wet ponds have a broader application for instream control where large watershed areas are involved compared to their use as onsite facilities for small urban areas. However, the recreational and aesthetic benefits of permanent wet ponds are very often considerable and may be justifiable in certain on-site applications.

10.3.3 Tank Storage

Tank storage is an underground tank or chamber, either pre-fabricated or constructed in place, which has a special controlled release feature. This method is most applicable where land area is very valuable, such as in industrial and commercial areas. Construction cost and operation costs which may include pumps make this method relatively expensive and, therefore, undesirable in most cases. Storage trenches, a variation on basic tank storage, are rock-filled underground storage tanks. The exception is that storage is provided within the void spaces between granular material, rather than in concrete structures.

10.3.4 Rooftop Storage

Rooftop storage is surface storage provided on flat rooftops designed with provisions for temporary ponding and with special roof-drain-controlled release features. Rooftop storage utilizes the built-in structural capability of rooftops to store certain amounts of rainfall.

Existing structures conforming to local building codes meet the requirement of being able to support a specified snow and live loading. This loading allowance can be utilized for stormwater storage without additional support. Modification of the roof drains to a controlled runoff device is usually all that is required for existing structures.

Rooftop storage can usually be incorporated into the design of new buildings. This method is probably one of the most convenient and economical to design and construct. It usually performs a retention function since the method is to hold the stored water for a relatively long time while draining gradually. For this reason, directing the drained water to lawn areas and infiltration trenches where it can percolate is possible.

The main disadvantages of rooftop storage are the inspection and maintenance requirements. Routine inspections are difficult when the installations are not readily accessible. Clogging of the roof drains make routing inspections a necessity. Another disadvantage of this method is the possibility of the roof drain flow control devices being removed after construction in the event the roof develops some leaks.

10.3.5 Parking Lot Storage

Parking lot storage is surface storage where shallow ponding is designed to flood specifically graded areas of the parking lot. Controlled release features are incorporated into the surface drainage system of the parking lot. Parking lot storage is a convenient multi-use structural control method where impervious parking lots are planned. Design features include small ponding areas with slotted controlled release structures and/or pipe-size reduction, and increased curb heights. This method can easily be incorporated into a site development at approximately the same cost as that of a conventional parking lot.

The major disadvantage is the inconvenience to users during the ponding function. This inconvenience can be minimized with proper design consideration. Clogging of the flow control device and icy conditions during cold weather are maintenance problems. Parking lot design and construction grades are critical factors. For these reasons, the functional effectiveness of parking lot storage is questionable. This method is intended to control the runoff directly from the parking area, and is usually not appropriate for storing large runoff volumes.

10.3.6 Special Fill Impoundments

Special fill impoundments are those where fills such as roads, parking lots, and other similar special use embankment fills are modified and designed to cause surface ponding and controlled release of stormwater runoff. These may be either wet or dry facilities. Special fill impoundments take full advantage of the multi-use concept.

10.4 Evaluation and Selection of Alternative Detention/Retention Structures

It is not possible to state which detention or retention facility is best in any given design situation. However, evaluation criteria are provided to assist the designer in the selection process. It is recommended that the features of each structure be examined with respect to the stages of facility development. The four basic stages of facility development are as follows:

1. Consideration of Alternatives

    a. Types of runoff control facilities
    b. Economic Considerations
    c. Multi-use features

d. Design complexity
e. Effective life

2. Site selection and design

   a. Land costs
   b. Site investigation requirements
   c. Soil limitations
   d. Accessibility to facility
   e. Aesthetic appeal and siting convenience

3. Construction

   a. Installation costs
   b. Complexity of construction
   c. Structural integrity
   d. Construction inspection

4. Operation and Maintenance

   a. Operation costs
   b. Maintenance plans and cost
   c. Ownership and maintenance responsibility
   d. Maintenance inspection needs

A 1972 survey of engineering firms (Practices in Detention of Urban Storm Water Runoff) revealed dry basins and wet ponds are prevalent over all other structural control methods. Detention of rainfall on parking lots ranked next, followed by detention of rainfall on rooftops.

10.5 Design Criteria for Detention/Retention Structures

The following criteria are recommended as standards of design. The criteria are not intended to provide a "cook book" method of design, but rather to place limits on design of structures so performance objectives can be achieved. Drawings of typical structures are included. Again, these drawings are not to be taken as the actual design configuration for all applications, but are included as a guide for design.

10.5.1 On-Site Detention (Dry) Basins

1. Discharge (release) rates:

   Maximum release rate shall be determined as stated in the recommended stormwater management policy. The peak discharge shall be determined based on undeveloped conditions using the appropriate runoff prediction method for the design area.

2. Volume determination:

Required storage volume shall be determined by the graphical storage method or the storage-indication method, discussed in Sub-section 10.7, for the critical storm recurrence interval, and for the proposed land use (inflow hydrograph).

3. Surface slopes:

The side slopes of the detention/retention basin, unless paved or riprapped, shall not exceed 4:1 to insure a maintenance capability.

The bottom of the basin shall be sloped to the flow control device. A method of carrying low flow through or around the basin shall be provided. The following minimum slopes are recommended to properly drain the basin:

Paved surfaces - 1%

Paved channels - 0.5%

Grassed surfaces - 2%

4. Maximum Water Depth and Embankment Width:

The maximum water depth should be about six feet, and the minimum top width of the embankment should be eight feet.

5. Discharge Control Facilities:

Control devices may be either single-stage or multi-stage devices. They shall be designed such that the permissible release rate is not exceeded at the highest water level.

Flow control structures should preferably be constructed of reinforced concrete either pre-fabricated or poured in place. The control structures shall be designed and constructed such that the public health, safety and welfare is fully protected with due consideration given to the structure being susceptible to plugging by debris. A 50 per cent increase in the net area of grated inlets is recommended to allow for possible blocking of the inlets by debris. A typical grated inlet is shown in Exhibit X-1.

Discharge velocities shall be controlled to prevent scouring of the downstream channel. The discharge control structure should in general be located at the bottom of the basin near the downstream end, although various other locations are permissible. In any event, the outlet device shall be at the absolute lowest point of the basin.

6. Detention Period

Detention period should be the minimum possible within the constraints of the runoff control standards.

7. Provisions for Upstream Tributary Areas

No storage volume will be required for off-site upstream areas. Flows from off-site areas in excess of the allowable discharge from the on-site area may be routed either around or through the detention/retention basin. Provision for excess flow routed through the storage facility shall be included in the design of the emergency spillway. The emergency spillway shall be designed for overflow rates based on the runoff predicted from the major design storm.

8. Barriers to access

Fencing around the storage facility in a manner which attempts to prevent access is generally undesirable from a maintenance and operation standpoint. In addition, experience has shown fences themselves present an inviting challenge to neighborhood children attracted to the treasures within. However, barriers to access may be desirable to delimit the area reserved for stormwater control. When required, these barriers should be designed with consideration given to the aesthetics of the site and nearby area. Landscaping and grading or natural wood fencing can offer suitable barriers.

9. Erosion and Sediment Control

Provisions for erosion and sediment control during construction should be included in the design of the storage facility. The four major design considerations of erosion and sediment control during construction are:

a. Disturb as little of the site as possible

b. Cover the soil as soon as possible

c. Control the rate of runoff by using temporary energy dissipators and control structures where necessary

d. Let the sediment settle out of the stormwater before leaving the site.

10. Aesthetics:

A storage facility is an integral part of the adjacent environment and, therefore, should serve as an aesthetic improvement to the area.

The use of landscape principles is encouraged and, in some cases, may be required for certain sites. In any event, the planting and preservation of desirable trees and other vegetation should be a part of the storage facility design.

11. Multi-purpose Uses:

The design criteria for multi-purpose basins such as recreation areas are in general the same as listed above with the following exceptions:

a. Side slopes flatter than 4:1 are recommended.

b. The deeper portions of the storage areas should be located in the more remote areas of the site.

c. Where the depth of the downstream outlet permits, it is recommended the storm sewer system be lowered such that the flow control structures can be located below the bottom surface of the basin. Typical design configurations of the on-site detention basins are included in Exhibits X-l and X-2. Orifice pipe control is shown in Exhibit X-2 where flow enters the basin when the upstream sewer system becomes surcharged.

12. Inlet Design:

The inlet should be designed such that the inflow velocities are controlled to tolerable magnitudes through the use of energy dissipation devices. The allowable inflow velocities shall be determined for the type of soil cover provided.

13. Permanent Maintenance Easements:

A permanent maintenance easement 10 feet in width shall be provided around the perimeter of the basin, and proper access to the facility for maintenance vehicles shall be provided.

10.5.2 On Site Retention (Wet) Ponds

General Criteria

1. General Criteria for dry basin design shall also apply to wet ponds except as modified herein.

Specific Criteria

2. Permament Pool Area and Depth

The maximum permament pool area shall not exceed 10 per cent of the upstream watershed area. The minimum depth from normal pool water level to bottom of side slope should be two feet, and at least 25 per cent of the pond area should be a minimum of ten feet deep. It is recommended the shallow perimeter of the pond extend from the bottom of the side slope for a distance of eight feet into the pond area. This is a safety factor in the event of accidental or intentional encroachment into the pond.

3. Eutrophication

A suitable means should be utilized to prevent water from becoming stagnant. Natural springs and aeration devices such as fountains are appropriate. Ponds constructed in permeable soils shall be sealed by a suitable method.

4. Embankment Width

The top of the embankment width shall be ten feet for nonvehicular traffic and 12 feet for vehicular traffic. The retaining structure shall be designed in accordance with the best accepted design practices.

5. Draining

Provision shall be included for completely draining the pond to allow periodic cleaning, inspection, and other maintenance. Drain facilities may be an integral part of the flow control structure or a separate structure.

6. Detention Period

The detention period for permanent ponds shall be the same as stated for dry basins except the detained volume shall be that amount of stored runoff above the normal pool water level.

7. Emergency Spillway

An emergency spillway shall be included which shall have a capacity for the runoff from a 100-year storm.

8. Outlet Channel

The outlet channel shall include adequate provision for energy dissipation and erosion protection (See Chapter 9, Open Channels).

9. Sediment

Probable quantities of sediment from the drainage area should be estimated for the expected life of the pond, and provisions for occasional sediment removal should be included in the design. Wet ponds may be over-excavated to accommodate siltation from construction activities during development.

10. Multi-purpose Ponds

Permanent ponds designed for multi-use purposes should meet the specific requirements of the uses intended in addition to the storm-water management requirements stated herein.

11. Hydraulic Gradient

The hydraulic gradient of the storage facility shall be indicated on the detailed plans both for the initial design storm and the 100-year major storm.

12. Pumping Facilities

The use of pumping facilities for on-site storage applications is discouraged for economy and reliability reasons. Common problems with pumped storage include high operation and maintenance costs, increased construction costs, increased design costs, and pump and power failures.

Where pumping facilities are proposed, the following design information shall be submitted for approval:

a. Certified pump head-discharge curves

b. Plans and sections of pumping structure indicating piping and pump arrangement details

c. Provisions for prevention of pump clogging

d. Power requirements and provisions for standby power facilities

e. Flood protection considerations

f. Safety and security provisions

g. Estimated construction and operation costs

A minimum of two pumps, each with capacity to pump the maximum release rate, is recommended to provide 100% standby pumping capability in the event of mechanical pump failure.

13. Permanent Easement

A permanent easement minimum 25 feet in width should be provided around the perimeter of the wet pond to act as a buffer zone to adjacent properties. The easement is measured from the water's edge for the maximum storage elevation or the outside bottom of the embankment side slope.

14. Shoreline Protection and Embankment Protection

Shoreline protection shall be provided where necessary to prevent wave action erosion. Embankment protection shall be provided to prevent erosion. A typical wet pond half profile is shown in Exhibit X-3.

10.5.3 Underground Storage Tanks

1. Materials

Underground storage tanks may be constructed in lieu of surface storage facilities. Storage tanks may be constructed of reinforced concrete either pre-fabricated, poured-in-place, or of corrugated steel. The structural design shall be in accordance with current design practice. A typical concrete storage tank is shown in Exhibit X-4.

2. Pipe Size

The minimum size drain pipe shall be 6 inches.

3. Access

An access hatch or manway shall be provided for inspection and maintenance. All openings shall be properly secured to minimize safety hazards.

4. Capacity

The required storage volume, discharge release rate, and detention period shall be as determined for surface storage facilities. A flow control device such as a simple weir or orifice shall be included.

5. Overflow

Overflow provisions shall be included to accommodate the less frequent storms up to and including the 100-year storm runoff.

6. Draining

The storage tank shall include provisions for completely draining the tank. The minimum slope of the tank bottom shall be 0.50 per cent.

Pumping facilities, where proposed, shall be designed in accordance with the requirements set forth for wet ponds.

10.5.4 Rooftop Storage

1. Maximum Water Depth -

   The maximum water depth for rooftop storage shall be three inches.

2. Discharge Period

   The rooftop shall be completely drained within the minimum time permitted under the runoff control standards (Chapter 3).

3. Live Load

   The roof structure shall be designed for a minimum live loading of 30 pounds per square foot.

4. Slope

   A minimum roof pitch of 0.25 inch per foot to the outlet device shall be provided to assure complete drainage.

5. Overflow

   Overflow drains shall be provided to accommodate major storms, and shall be located above the maximum water depth. Roof scuppers are to be provided in parapet walls.

6. Waterproofing

   The building structure shall be designed to provide a watertight roof. Additional layers of roofing membrane or coating are recommended to provide a watertight seal.

7. Flow control device

   It is recommended the flow control device be in compliance with the local building code and the National Plumbing Code. Several types of fixed and variable capacity flow control devices (roof drains) are available commercially. One type is "Flo-Set" roof drains manufactured by Josam Manufacturing Company, Michigan City, Indiana. Drain pipes and downspouts will be of standard design.

8. Combination Designs

   The design of other storage control methods in combination with rooftop storage, such as infiltration or storage trenches, is appropriate.

10.5.5 Parking Lot Storage (Storage of Runoff in Depressions)

1. Maximum Water Depth

    The water depth in the storage area shall not exceed seven inches.

2. Discharge Period

    The parking lot shall be completely drained in the minimum time permissible within the constraints of the runoff control standards (Chapter 3).

3. Outlet Structure

    The outlet structure shall be located at the absolute lowest point or points in the storage area.

4. Storage location

    The storage area should be located in the more remote, least used, portion of the parking facility. The ponded area at the maximum water depth shall be at least 50 feet distant from building structures.

5. Slope

    The maximum surface slope of the storage area shall be 4 per cent and the recommended minimum slope is 1 per cent.

6. Overflow

    Provisions shall be included for overflow of runoff above the design storm, such as uncontrolled drainage structures near the ponding area or surface outlets. When the latter is provided, special consideration shall be given to control of runoff velocities. Damage from soil erosion shall be minimized.

7. Storage Volume

    The required storage volume shall be determined in accordance with the same design criteria as for dry basins.

8. Alternate Designs

    Consideration shall be given to the use of semi-paved/semi-grassed areas for remote parking areas. These can include infiltration methods and thereby reduce total runoff.

TABLE 10 - I

SUMMARY OF DESIGN CRITERIA
FOR
ONSITE DETENTION/RETENTION STRUCTURES

Control Method	Maximum Side Slope	Maximum/Minimum Water Depth	Top Width of Embankment	Permanent Easement Widt
Detention (Dry) Basin	4:1	Maximum 4-6 Feet	8 Feet	10 Feet
Retention (Wet) Ponds	4:1	Minimum Pool 10 Feet	10 Feet 12 Feet (Vehicular Traffic)	25 Feet
Underground Storage Tanks	-	-	-	-
Rooftop Storage	-	Maximum 3 Inches	-	-
Parking Lot Storage	-	Maximum 7 Inches	-	-
Roadway Embankment Basins	4:1	Maximum 4-6 Feet	Varies	Varies

The use of pervious pavement materials is possible in certain applications, although widespread use is discouraged unless appropriate means to prevent clogging the pervious areas are provided.

10.5.6 Special Fill Impoundments

General Criteria

Special Fill Impoundments shall, in general, be designed in accordance with the criteria stated herein for detention/retention basins. In addition, special fill impoundment areas shall meet the specific requirements of the primary use or uses intended.

Topographic and soil characteristics of individual proposed sites are major design considerations in determining the feasibility of special fills for use as a stormwater management facility.

Specific Criteria for Roadway Embankment Impoundments

1. Discharge Control Facilities

   The recommended flow control device is a specially designed culvert structure. The culvert structure shall be designed in accordance with the criteria set forth in Chapter 8 of this manual. In addition, a minimum of two feet of freeboard shall be provided above the maximum water depth for the design storm runoff volume.

2. Overflow Spillway

   The roadway and embankment may serve as the overflow spillway. Erosion protection shall be provided from the upstream point of the embankment face to the downstream toe of the embankment. The overflow spillway shall be designed for overflow rates based on the runoff predicted from a 100-year storm rainfall.

10.5.7 Summary of Design Criteria

A summary of the basic design criteria parameters is given in Table 10-1. They are intended to establish general limits of design and are not all-inclusive. Hopefully, these criteria will be refined and further expanded to suit specific applications. In the final analysis, engineering judgment and actual experience are important factors of any design. The detailed criteria should be based on a thorough investigation of site conditions and design limitations applied to all stages of facility development. The Soil Conservation Service Engineering Field Manual is a valuable source for typical design of embankments, ponds, spillways, and outlet structures and should be consulted for detailed information not specified herein.

10.6 Infiltration for Runoff Control

Infiltration methods seek to restore the naturally occurring process of the hydrologic system by permitting stormwater to percolate into the ground. The degree to which infiltration of stormwater can be utilized depends on the physical characteristics of the soil or soils and the groundwater system of the area. Obviously, infiltration of stormwater lessens the amount of rainfall that becomes runoff. Hydrologically, infiltration reduces the total volume of direct runoff and decreases the peak discharge rate. The infiltration process also enhances groundwater recharge.

Current emphasis appears to be toward the use of the infiltration methods only where soil suitability and site conditions are favorable. Soils having slow permeability and shallow depths to bedrock are, understandably, poor locations for infiltration methods. Sediments, oils, and other debris can cause problems by clogging the soil surface resulting in loss of storage capacity. Therefore, installation of infiltration structures near areas where these products are present is discouraged. Infiltration structures usually have a short term life expectancy and maintenance is expensive. Thus, unless adequate inspection and maintenance can be provided, the use of structural infiltration methods is discouraged.

In most instances, infiltration methods are not capable of handling large amounts of runoff due to limits of the soil infiltration capacity. Infiltration control methods, therefore, are usually limited to handling relatively small sources of runoff such as roof drains, small parking lots, tennis courts, and the like. Infiltration methods may also be incorporated as one part of a large runoff control system utilizing several different techniques. Some specific infiltration structures are dry wells, infiltration trenches, and storage trenches.

10.6.1 Dry Wells

Dry wells vary in depth from six feet to several hundred feet, depending on the depth of the permeable soil strata and the depth to bedrock. Shallow wells may range from three feet to several feet in diameter. Deep wells ususally have a diameter less than three feet. Dry wells should be filled with crushed stone or washed gravel of two inch size. Storage volume may be computed by assuming one-third of the storage volume is within the voids between the stones. Therefore, the dry well excavated volume must be three times the required storage volume. Dry wells are most applicable for storing runoff from rooftops and other areas free of sediment and debris. No control release structure is required. They may discharge directly into unsaturated soils or the groundwater. Dry wells discharging into an aquifer are subject to regulation and approval by the state.

10.6.2 Infiltration Trenches

Infiltration trenches require the same degree of site investigation and soil consideration as dry wells. They normally have a depth less than six feet. Groundwater should be a minimum of 10 feet below the bottom of the trench. Storage volume is determined as for dry wells. The excavated trench should be filled with 2-inch crushed stone or washed gravel. The trench should include an observation well for inspection.

Vegetative buffers at least 20 feet in width are recommended upstream when contributing runoff may be bearing sediment. The use of infiltration trenches is discouraged unless adequate provisions are included for inspection and maintenance.

10.6.3 Storage Trenches

Storage trenches are similar to underground storage tanks, except that the stored runoff is in the voids between the 2-inch stone rather than in a concrete or steel structure. With this exception, storage trenches should be designed in accordance with the criteria for storage tanks (Section 10.5.3). Storage trenches are subject to clogging by sediment as are infiltration trenches, and will require maintenance.

10.7 Conduit Structures

Conduit structures include storm drainage pipes, open ditches (natural and man-made), natural streams, and channelized streams. They serve primarily as a conveyance system to transport stormwater from one point to another. Conduits are used to direct flows to a particular point in the stormwater management system and may be used to bypass flows. Flow bypassing is a viable method in certain applications and in some instances is unavoidable. When stormwater flows are bypassed, the impact at the point of re-entry into the drainage system should be investigated. Details of channel design and storm sewer design are discussed in Chapters 7 and 9 of this manual.

Conduit storage as a structural control method is underground storage provided by enlarged storm drain pipes, or storm inlet structures that include storage capacity and a flow control device. It compares closely with tank storage except for construction requirements. Conduit storage is generally applied to urban high land value areas where storage basins and ponds are undesirable, such as small industrial and commercial areas. Storm pipe manholes provide easy access for maintenance purposes. This method is generally expensive and impractical for runoff control of large areas or for storm runoff from high intensity storms where emergency overflow must be provided.

Conduit structures (oversized or enlarged storm sewer pipes) shall be designed in accordance with the criteria specified for underground storage tanks, except as modified herein.

Special consideration shall be given to the structural strength and load-carrying capacity of the conduit as well as the bearing capacity of the soil. Perforated pipe may be used where soil conditions are favorable for infiltration methods.

10.8 Methods for Determining Storage Capacity

This section examines several methods for estimating the volume requirements of different types of stormwater storage facilities. The criteria used to choose the appropriate runoff prediction method (Chapter 5) may also be used to determine the method of estimating storage requirements. Generally, if the Peak Discharge Method of runoff calculation is appropriate for a given situation, it may be assumed that Graphical Flow Routing Methods are similarly applicable. If the Tabular Method or Unit Hydrograph Method are used for estimating runoff then the Storage Indication Flow Routing Method is most accurate for determining storage capacity.

10.8.1 Graphical Flow Routing Method

The graphical method presented herein is developed by the Soil Conservation Service, presented in Chapter 7 of the "Urban Hydrology for Small Watersheds," Technical Release No. 55. It is based on average storage and routing effects using the storage-indication method of routing. The graphs relate inflow (Q_i) and release rate (Q_o) to storage requirements for single stage weir or pipe outlet structures. Emergency spillway flow (overflow) is not considered in this method.

The proper graph to use will depend on the release rate and the ratio of release rate to inflow rate as shown in Table 10-II which follows.

Use of these graphs will result in rough approximation since this method is based on several general assumptions. The results of the Graphical Flow Routing Method should be interpreted accordingly.

For any application where the graphical method is not appropriate, a more accurate flow routing method, such as the storage-indication method, is required to determine storage requirements.

10.8.2 Storage-Indication Flow Routing Method

Flow routing problems are solved by using the continuity equation. The continuity equation is based on the concept of conservation of mass. For a given time interval the volume of inflow minus the volume of outflow equals the change in volume of storage. The continuity equation expressed in this simplest form is:

$$\Delta t \ (\overline{I} - \overline{O}) = S$$

where:

Δt = a time interval

$\overline{I}$ = average rate of inflow during the time interval

$\overline{O}$ = average rate of outflow during the time interval

S = change in volume of storage during the time interval

The rate of inflow is determined by the inflow hydrograph. The rate of discharge is determined by the elevation versus discharge characteristics of the reservoir outlet structure. The change in storage volume is represented by the elevation versus storage curve of a reservoir. For routing studies, the inflow hydrograph is determined once a design storm and watershed parameters have been established. Also, the elevation-storage curve is determined by the reservoir site. Therefore for a particular

TABLE 10 - II

APPLICATION OF GRAPHICAL STORAGE DETERMINATION

Release Rate, CSM (cfs/sq. mi.) Pipe Flow	Weir Flow	Ratio of Release Rate to Inflow Rate	Graph Selection
30-300	30-150	Up to 0.35	Exhibit X-5
300	150	Greater than 0.35	Exhibit X-6

The general procedure for using these graphs is as follows:

Using the Graph in Exhibit X-5:

1. Enter graph with total direct runoff depth V_r (watershed inches) for the "developed" conditions using the Peak Discharge Method.

2. Find point on the appropriate curve for the design release rate (maximum discharge rate, Q_o) corresponding to the runoff depth.

3. Find the required storage in watershed inches (Vs).

4. Calculate the storage in acre-feet by multiplying the required storage (inches) by the watershed area in acres and dividing by 12.

Using the Graph in Exhibit X-6

1. Calculate the peak inflow rate (Q_i) for the "developed" conditions using the Peak Discharge Method.

2. Calculate the ratio of design release rate (Q_o) to inflow rate (Q_i) for the same units.

3. Enter the graph with Q_o/Q_i and from the appropriate point on the curve, read the value for the ratio V_s/V_r.

4. Calculate the required storage (V_s) by mutiplying the above ratio by the volume of runoff (V_r) in inches predicted for the "developed" conditions using the Peak Discharge Method.

site, design storm, and drainage area, the only variable in the continuity equation is the elevation-discharge curve. The elevation-discharge curve is dependent on the hydraulic characteristics of the flow control device (outlet structure).

Flow control devices can be either fixed or variable. For purposes of stormwater management the most reliable is the fixed type, either an orifice, a simple weir or a combination of the two. The theoretical flow characteristics for these types of devices are as follows:

Circular orifice, (submerged discharge)(1)

$$Q = Ca \sqrt{2gh}$$

where:

Q = orifice discharge in cfs

C = coefficient of discharge

a = orifice cross-sectional area in sq. ft.

g = gravitational acceleration constant = 32.2 ft/sec^2

h = height of water surface over center of orifice.

When the storage water surface falls below the top of the orifice opening, the flow characteristics are equivalent to those of a weir.

Weir discharge M (2)

$$Q = CLH^{3/2}$$

where:

Q = weir discharge in cfs

C = weir coefficeint of discharge

L = length of weir in feet

H = hydraulic head above weir crest in feet.

Values for the constants to be used in the above equations for different configurations of weir may be found in the reference given. Orifice pipe and weir discharge curves are shown in Exhibits X-7 and X-8, respectively.

---

(1) Reference: Handbook of Hydraulics, Fifth Edition, King & Brater, pp. 4-3.

(2) Reference: Handbook of Hydraulics, Fifth Edition, King & Brater, pp. 5-3 et. seq.

Multistage outlet control structures are possible by including both the orifice and weir in one structure. A common configuraton used in detention structures is a pipe orifice as a primary discharge near the bottom of a reservoir with a simple weir located a few feet above the orifice or near the top of the structure as a secondary discharge. When both control devices are used together, the resulting discharge is the sum of the individual discharges for a given storage elevation.

Several techniques have been developed for solving flow routing problems. One method which is relatively simple and gives accurate results for many applications is the Storage-Indication Method. This method is recommended for use when the graphical storage method is not applicable.

For sufficiently small time intervals, (Δt) the continuity equation can be expressed:

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{S_2 - S_1}{\Delta t} \quad (1)$$

Subscripts "1" and "2" represent the beginning and end of the time interval. Usually, the complete inflow hydrograph is known or can be determined, as well as the initial storage and outflow values. Therefore, the reamining unknown values are the outflow at the end of the time interval (O_2) and the storage volume (S_2). The above equation is rearranged as follows:

$$\frac{I_1 + I_2}{2} + \frac{S_1}{\Delta t} - \frac{O_1}{2} = \frac{S_2}{\Delta t} + \frac{O_2}{2} \quad (2)$$

The left side of this equation can be determined for successive increments of time, and therefore, the right side of the equation can be quantified. Storage-indication curves are developed to determine outflow (O_2) for known values of $\left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right)$.

The general procedure for flow routing by the Storage-Indication Method is as follows:

1. Develop the inflow hydrograph. Hydrographs can be developed using the tabular hydrograph method, by synthetic hydrograph procedures or by actual stream measurement.

2. Estimate storage requirements, make a preliminary design of a storage facility and develop an elevation-storage curve for the storage site. Data may be obtained from field survey and topographic maps. (The graphical flow routing method is helpful in preliminary design of the storage facility).

3. Develop an elevation-discharge curve for the reservoir discharge control structure. This is based on the hydraulic characteristics of the flow control device. (The size of the flow control device is based on the design discharge rate and the design high water level).

4. Select the time interval (Δt). The value of Δt may need to be decreased if the storage-indication curve developed in the following step exceeds the equal value line represented by $O_2 = S_2/\Delta t + O_2/2$.

5. Develop the storage-indication curve by plotting O_2 versus $\left\{\frac{S_2}{\Delta t} + \frac{O_2}{2}\right\}$.

   Check the choice of Δt as explained in the preceding step. The suggested format for storage-indication computation and operation tables is included to facilitate calculations of this and subsequent steps.

6. Tabulate inflow values for each routing interval (Δt) from the design inflow hydrograph, and calculate the average inflow $\overline{I}$ by averaging successive values.

   $$\overline{I} = \frac{I_1 + I_2}{2} \quad \text{(Use operations table)} \quad (3)$$

7. Perform the routing by using the operations table and the storage indication curve.

   $$\frac{S_1}{\Delta t} - \frac{O_1}{2} = \frac{S_1}{\Delta t} + \frac{O_1}{2} - \frac{O_1}{} \quad (4)$$

The sum of the values from equations (3) and (4) above for each time interval (Δt) quantifies the right side of equation (2).

When inflow equals zero, all values across the operations table are zero. From this point on, average inflow $\overline{I}$ is determined from Step Six, and values for $\frac{S_1}{\Delta t} + \frac{O_1}{2}$ and O_1 are taken from preceding values of $\frac{S_2}{\Delta t} + \frac{O_2}{2}$ and O_2 of the preceding routing interval.

Referring to the operations table, the value of $\frac{S_2}{\Delta t} + \frac{O_2}{2}$ is obtained by summing columns five and six and subtracting column seven. The outflow value (O_2) is determined using the developed storage-indication curve (Step five).

8. Develop the outflow hydrograph by plotting O_2 versus time from the operations table.

10.8.3 Examples Using Storage Indication Flow Routing Method

The example calculations below illustrate the storage-indication method of flow routing through structures.

Example:

A storage reservoir is to be placed on Sub-area 7 for the typical drainage area presented in Section 5.3.3, Tabular Method.

Given:

Five-year developed inflow hydrograph using the tabular method. Sub-area 7 drainage area equals 0.20 square mile.

Time of concentration, Tc = 0.75 hours

Travel time = 0.0 hours

Runoff = 2.26 inches

Maximum Design Release Rate = 35.2 cfs (1-year present condition)

Required:

Determine the required storage volume using the Storage-Indication Flow Routing Method such that the maximum design release rate is not exceeded.

1. Inflow Hydrograph by Tabular Method:

Sub-area 7 Hydrograph Time (Hours)

8.0	9.0	10.0	11.0	11.5	11.75	12.0	12.25	12.5	12.75	13.0	13.25	13.50	13.75	14.0	14.25	14.50	15.0
0	0.8	2.7	7.6	16.0	59	138	165	126	88	61	44	34	27	23	20	18	15

The inflow hydrograph is shown in Exhibit X-9.

2. Preliminary volume requirements are estimated by using the graphical flow routing method.

V_s = 10.3 Acre - ft.

7. The flow routing is performed as shown in the operations table. A partial operations table is included below to indicate the routing operation.

TABLE 10 - III

PARTIAL OPERATIONS TABLE

(3) Time	(4) Inflow	(5) $\bar{I}$	(6) $\frac{S_1}{\Delta t} + \frac{O_1}{2}$	(7) O_1	(8) $\frac{S_2}{\Delta t} + \frac{O_2}{2}$	(9) O_2
0	16.0	11.4	0	0	0	0
15	59[a]	37.5[b]	0	0	37.5 [c]	1.6 [d]
30	138	98.5	37.5	1.6	134.4	8.0

(a) Step 6 above.

(b) Step 6 above

(c) Columns 5 + 6 minus Column 7

(d) From Storage - Indication curve (Exhibit X-11)

Note: In this example, it is assumed no outflow occurs prior to 11.5 hours (hydrograph time based on the Soil Conservation Service Type II Storm) when the inflow equals 16 cfs. In actuality, some outflow does occur prior to this time, but the value is too small to warrant its calculations.

In most cases, the beginning discharge values of inflow are small and may be ignored in the storage-indication flow routing method without significant loss in accuracy. An inspection of the design inflow hydrograph will determine the appropriate beginning value to use. Where a high degree of accuracy is required, computer analysis of flow routing through structures is recommended.

8. The storage outflow, O_2 (Column 9 of Exhibit X-12) is plotted versus Hydrograph Time, (Column 1) to determine the configuration of the outflow hydrograph as shown in Exhibit X-13.

Exhibit X-13 includes the inflow hydrograph and the storage volume determination.

The storage volume is determined graphically. The area between the inflow and outflow hydrographs represents the required storage volume in cfs-hour units. To convert cfs-hours to cubic feet, multiply by 3,600; i.e., 1 cfs-hour = 3,600 cubic feet.

In the above example, the required storage volume equals approximately 11.0 acre-feet, and the maximum outflow rate (O_2) is 36.2 cfs (Exhibit X-12) which is greater than the maximum design release rate of 35.2 cfs. Therefore, adjustments must be made in the design of the outflow control structure to reduce the discharge rate.

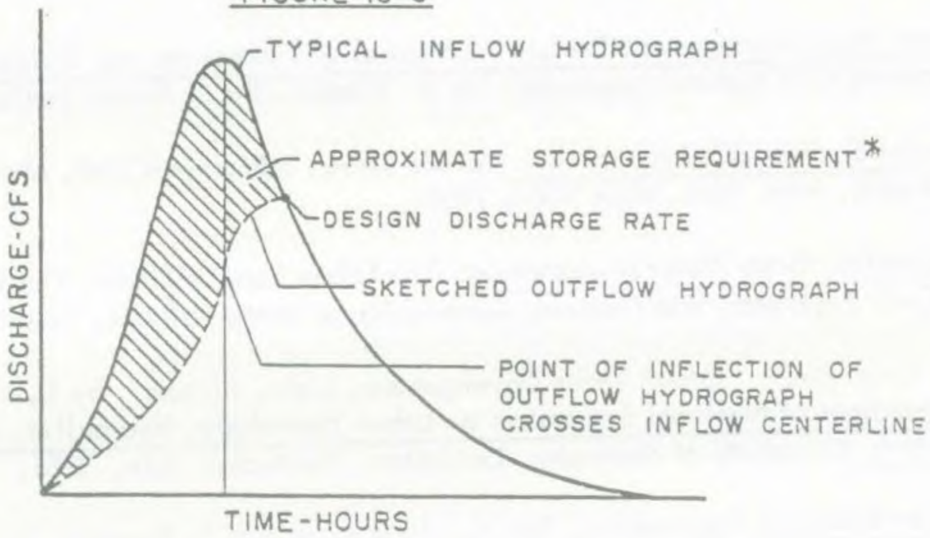
The same principles and procedures of flow routing by the storage-indication method can be applied to channel routing as well as reservoir routing. The effects of channel storage can be determined, if necessary, for a particular design situation. As with reservoir routing, the stage-storage and stage-discharge relationships must be determined for the channel in question.

As the preceding flow routing example calculations indicate, manual calculations of the storage-indication method can become quite lengthy. Computer analysis can minimize computation time and is recommended whenever there are several storage facilities and/or channel reaches to be analyzed within a watershed.

10.8.4 Graphical Approximations of Storage Requirements

Preliminary determinations of storage volume may be found using various graphical methods. One such method is a graphical version of the Mass-Curve Method of flow routing through structures described in Chapter 17 of the National Engineering Handbook, Section 4, "Hydrology." Another approximate method is by sketching the outflow hydrograph on a graph together with the inflow hydrograph similar to the hydrographs shown in Exhibit X-13. The design discharge rate (peak of outflow hydrograph) is plotted on the falling limb of the inflow hydrograph, the point of inflection on the rising limb of the outflow hydrograph crosses the centerline of the inflow hydrograph, and the beginning point (zero flow) is the same for both hydrographs. The subtended area is the approximate storage volume of the proposed facility. The procedure is illustrated graphically below.

FIGURE 10-3



Graphical Storage Volume Determination

by Sketching Outflow Hydrograph

*Convert cfs-hour units to cubic feet as stated on Exhibit X-13.

10.9 Bibliography

1. Practices in Detention of Urban Stormwater Runoff, Special Report No. 43, American Public Works Association, Chicago, Illinois, 1974.

2. "An Evaluation of Practical Experience in Storm Water Management," by George L. Stem, Proceedings of National Symposium on Urban Hydrology and Sediment Control, University of Kentucky, Lexington, Kentucky, July, 1975.

3. "Section 4, Hydrology", Soil Conservation Service, National Engineering Handbook, U.S. Department of Agriculture, U.S. Government Printing Office, Washington, D.C., January, 1971.

4. Urban Hydrology for Small Watersheds, Technical Release No. 55, Soil Conservation Service Engineering Division, U.S. Department of Agriculture, January, 1975.

5. Tabular Method of Flood Routing - 24-Hour Type II Storm Distribution, TSC Technical Note - Engineering - UD-20, Soil Conservation Service Regional Technical Service Center, U.S. Department of Agriculture, Upper Darby, Pennsylvania, January, 1972.

6. Computer Program for Project Formulation - Hydrology, Technical Release No. 20, Soil Conservation Service Central Technical Unit, U.S. Department of Agriculture, Washington, D.C., May, 1965.

7. Engineering Field Manual for Conservation Practices, U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., 1969.

8. Storm Water Management Design: A Manual of Procedures and Guidelines. Maryland Department of Natural Resources, Roy F. Weston, Inc., Pennsylvania, April, 1976.

9. Handbook of Applied Hydrology, Ven Te Chow, Editor-in-Chief, McGraw-Hill Book Company, New York, New York, 1964.

10. Residential Storm Water Management, The Urban Land Institute, The American Society of Civil Engineers, The National Association of Home Builders, Washington, D.C., 1975.

11. "A Case Study: Green Trails Development, Lisle, Illinois", by R. William Lindley, Proceedings of National Symposium on Urban Hydrology, Hydraulics, and Sediment Control, University of Kentucky, Lexington, Kentucky, July, 1976.

12. Water-Resources Engineering, Ray K. Linsley, Joseph B. Franzini, Second Edition, McGraw-Hill Book Company, New York, New York, 1972.

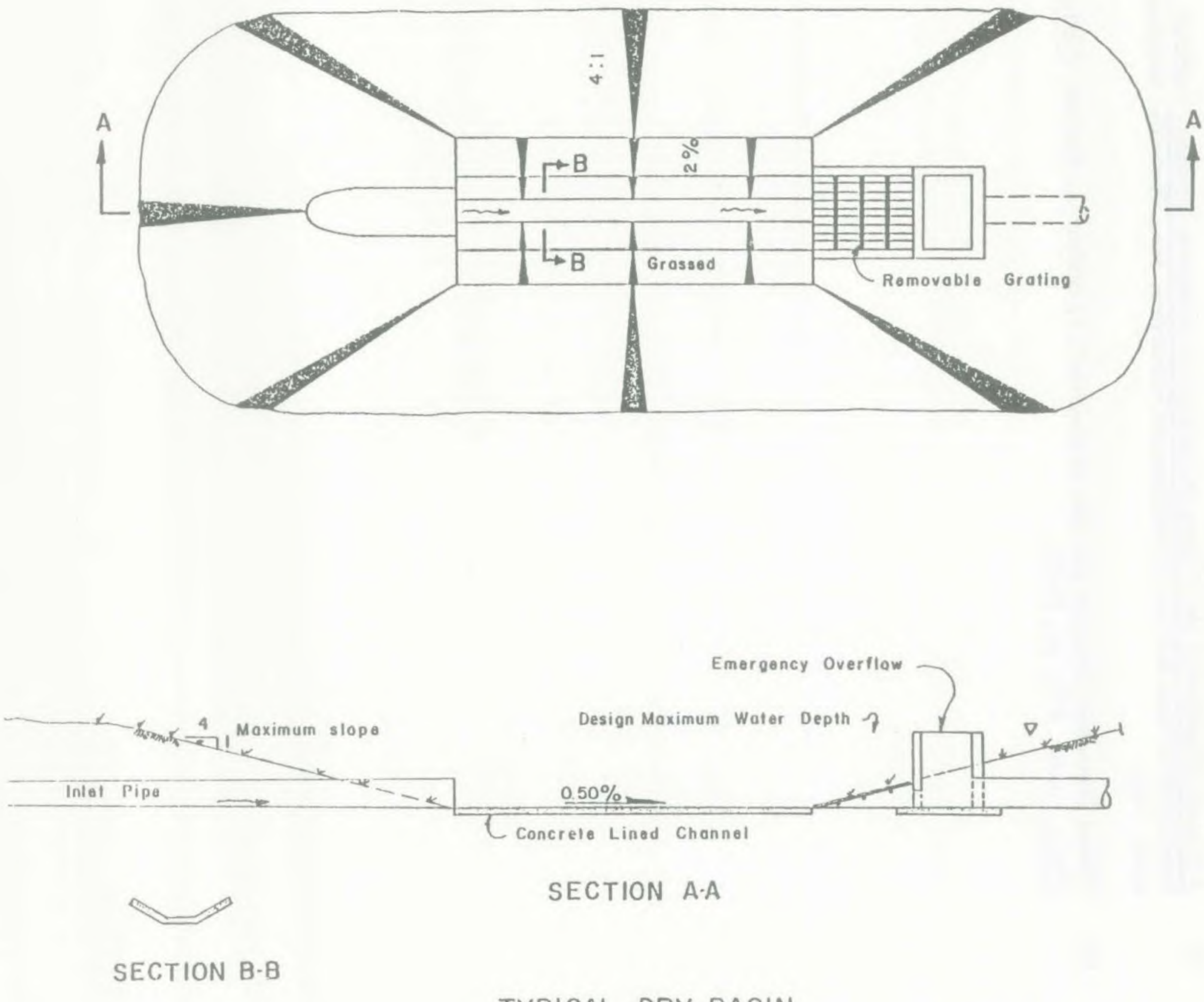
13. A Manual on Storm Water Management for Evaluating and Mitigating the Effects of Land Use Changes on Runoff, The Maryland-National Capital Park and Planning Commission, Roy F. Weston, Inc., Pennsylvania, June, 1974.

14. "Urban Storm Runoff Controls" by David L. Daugherty, Proceedings of Planning and Design for Urban Runoff and Sediment Management, University of Kentucky, Lexington, Kentucky, 1973.

15. Lakes and Ponds, Technical Bulletin 72, The Urban Land Institute, Washington, D.C., 1976.

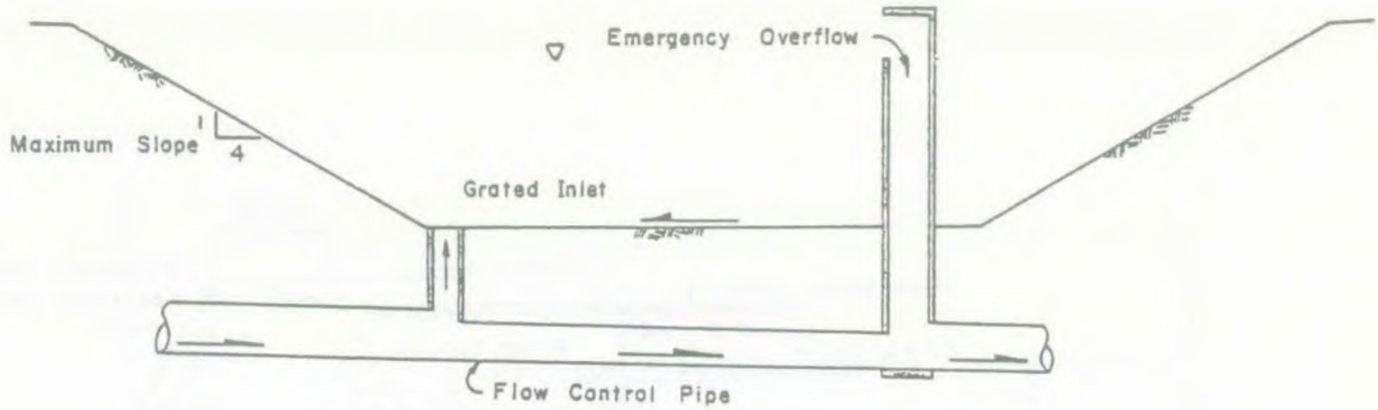
16. A Manual of Residential Storm Water Management Development Standards, National Association of Home Builders, NAHB Research Foundation, Inc., Rockville, Maryland, April, 1973.

17. "Urban Stormwater Detention and Flow Attenuation", by Herbert G. Poertner, Public Works, August, 1976, pp. 83-85.

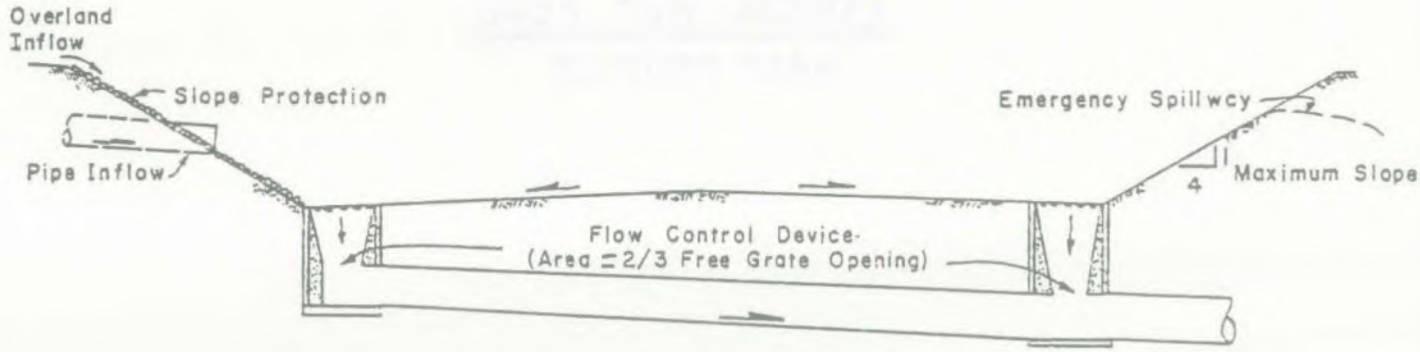


TYPICAL DRY BASIN

EXHIBIT X-I

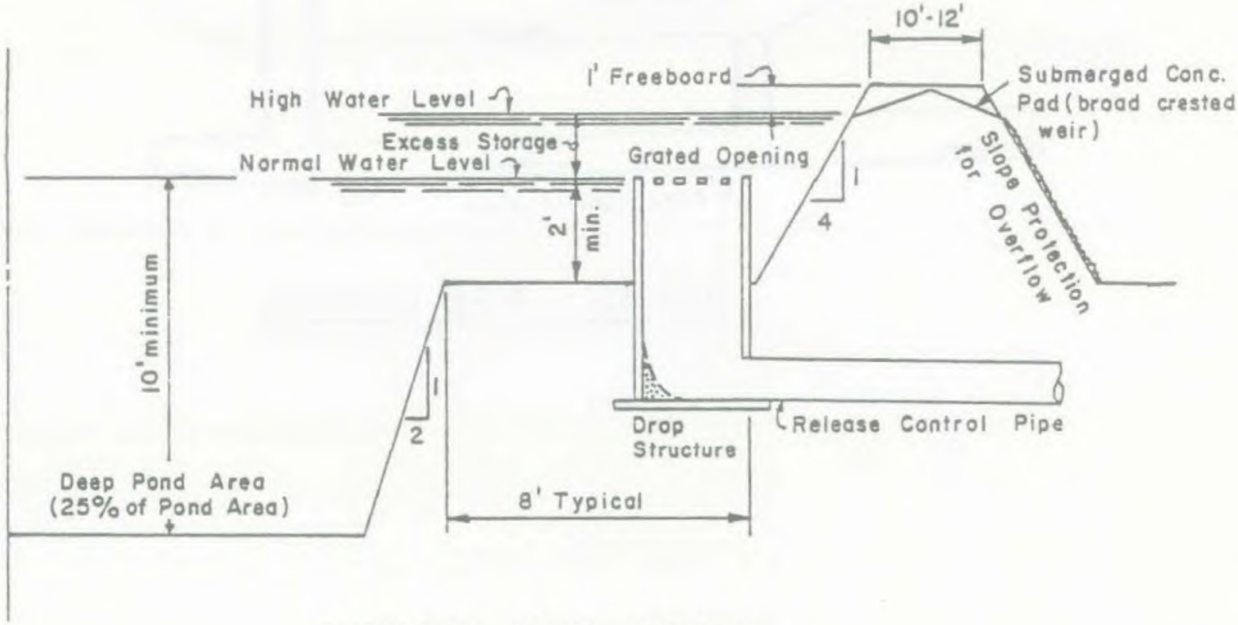


ORIFICE PIPE CONTROL



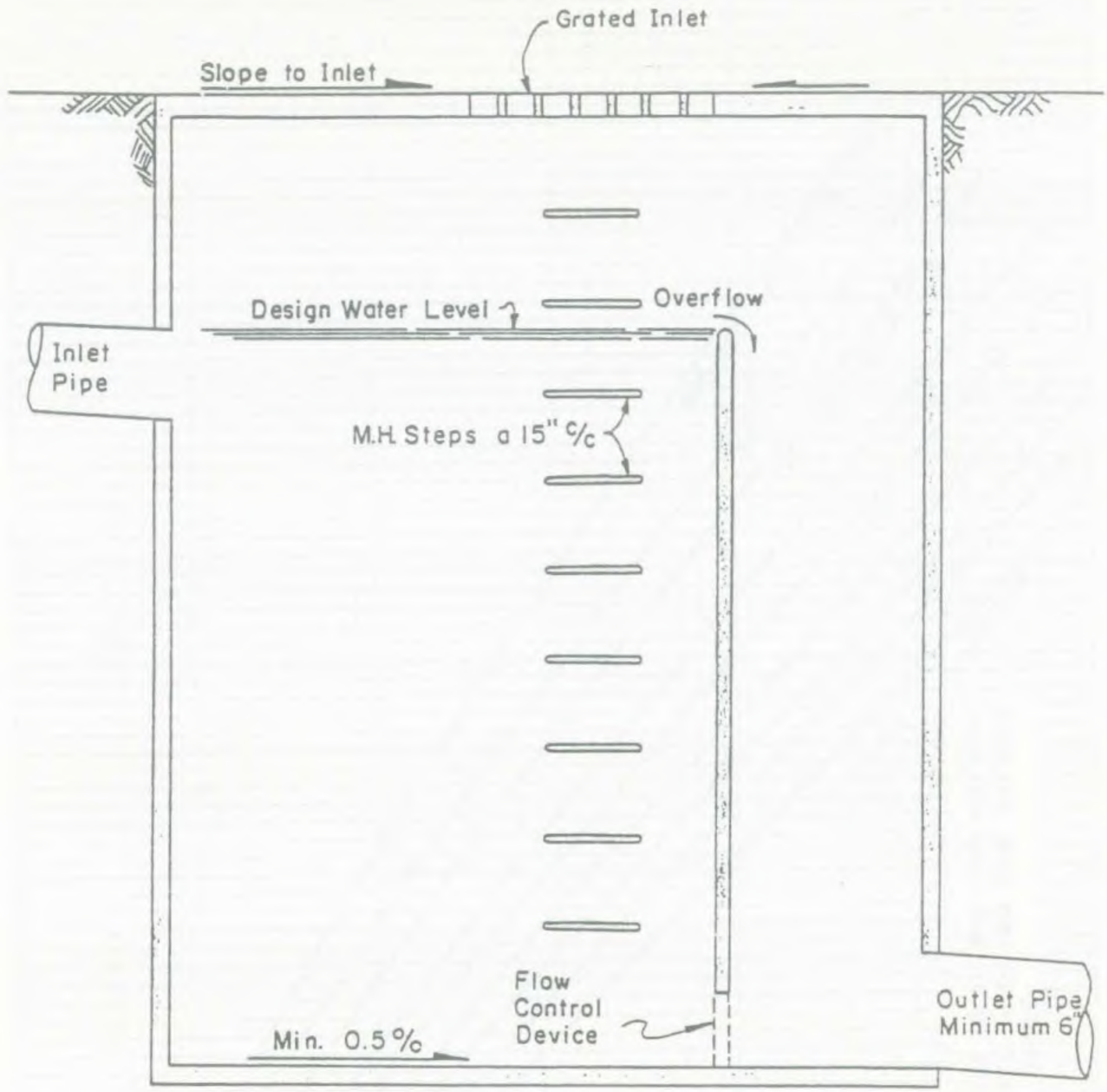
CONTROLLED RATE INLETS

TYPICAL DRY BASIN
ALTERNATE DESIGN PROFILES



TYPICAL WET POND

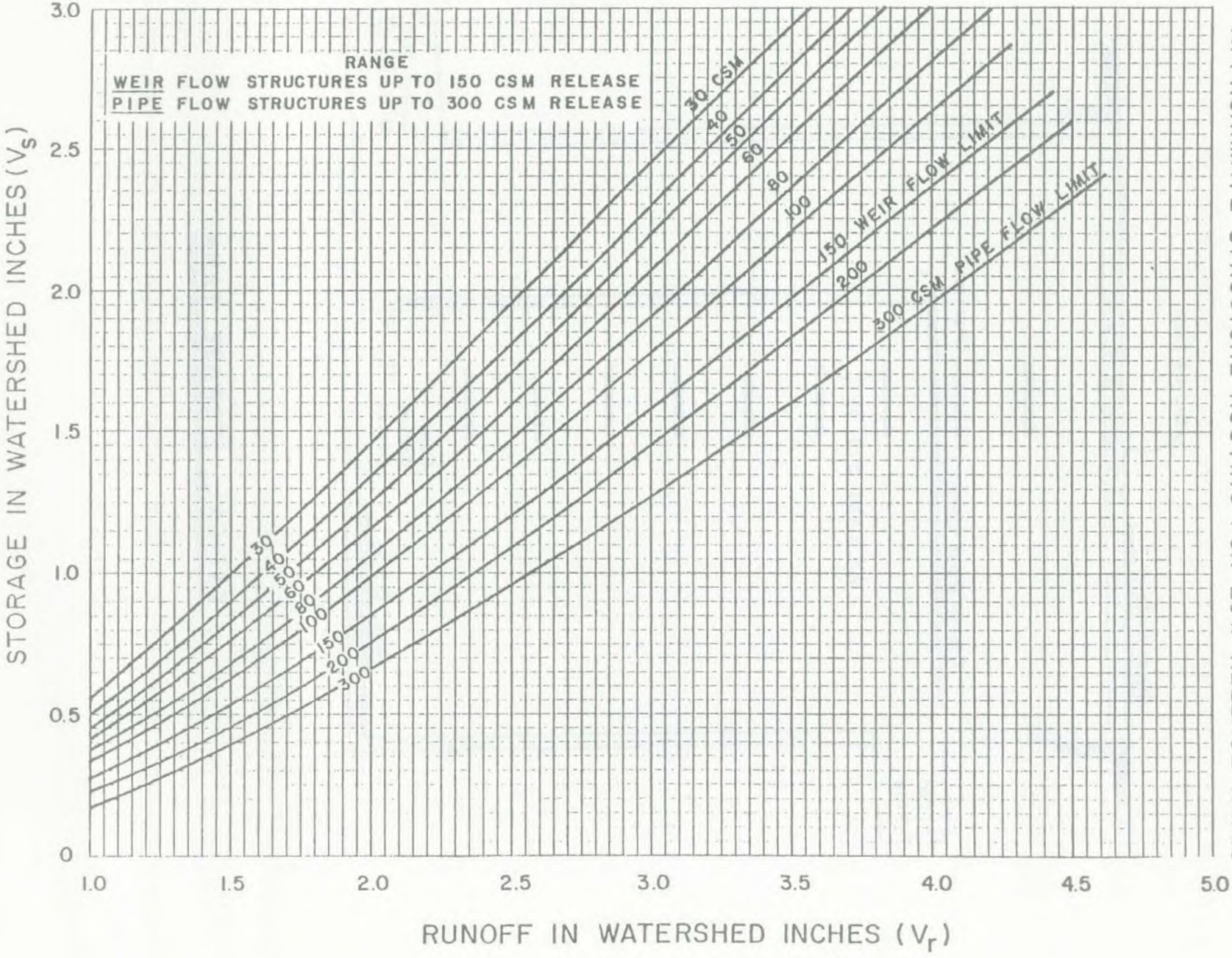
HALF PROFILE



TYPICAL STORAGE TANK

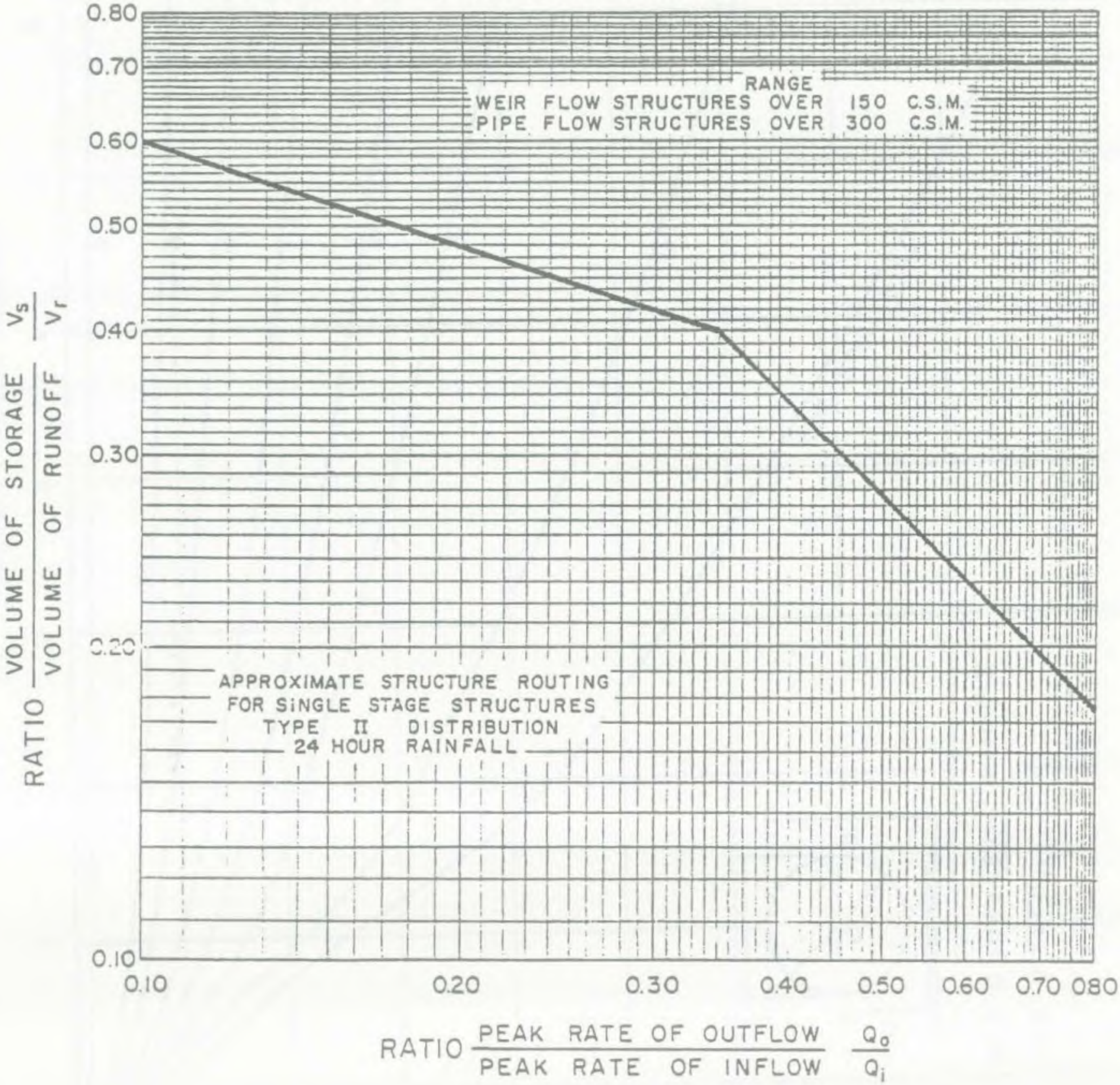
EXHIBIT X-5

APPROXIMATE STRUCTURE ROUTING FOR LOW RELEASE RATES



Source: S.C.S. Urban Hydrology for Small Watersheds, T.R. No 55

APPROXIMATE STRUCTURE ROUTING FOR HIGH RELEASE RATES



Source: S.C.S. Urban Hydrology for Small Watersheds, T.R. Nº 55

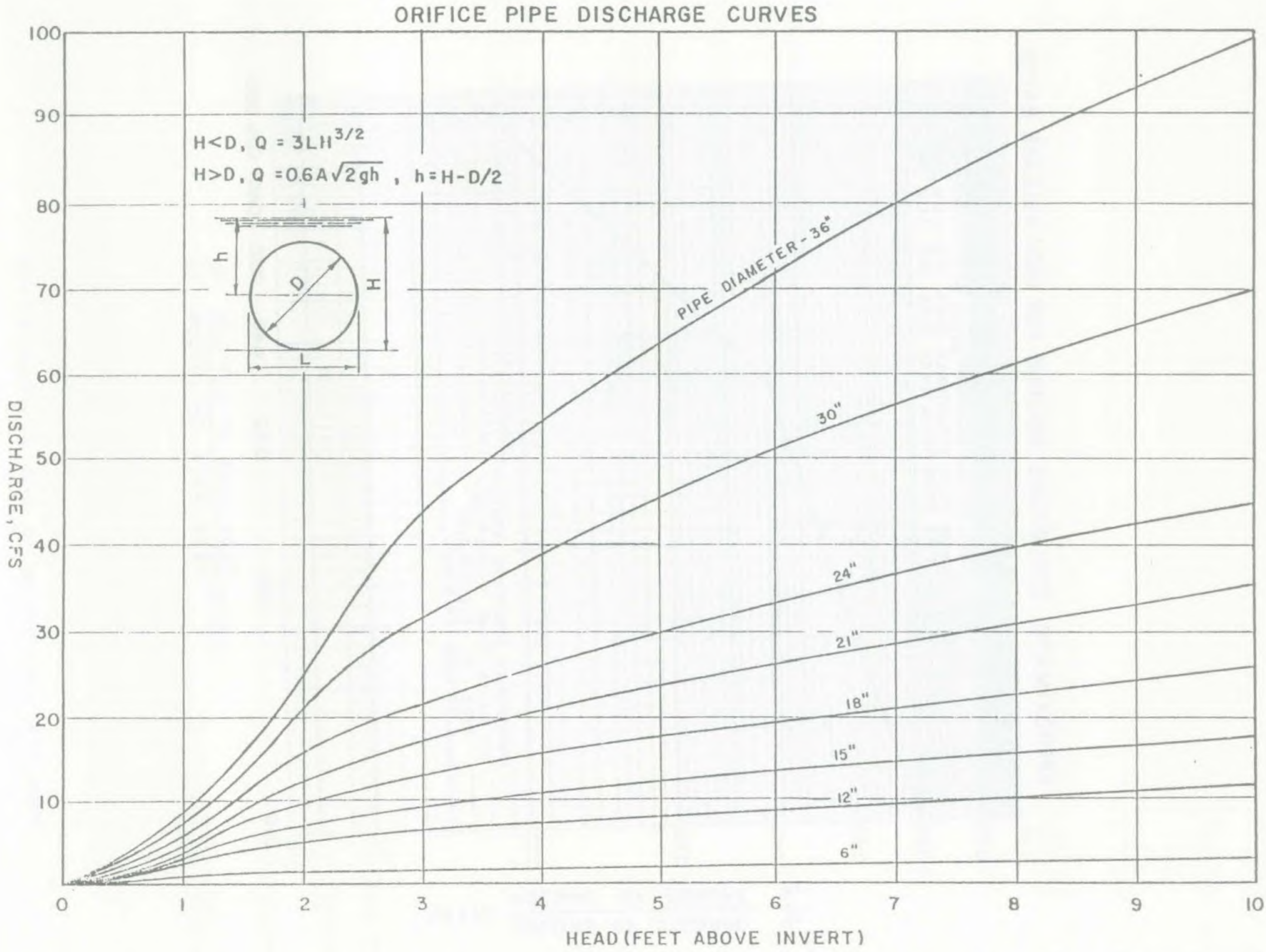
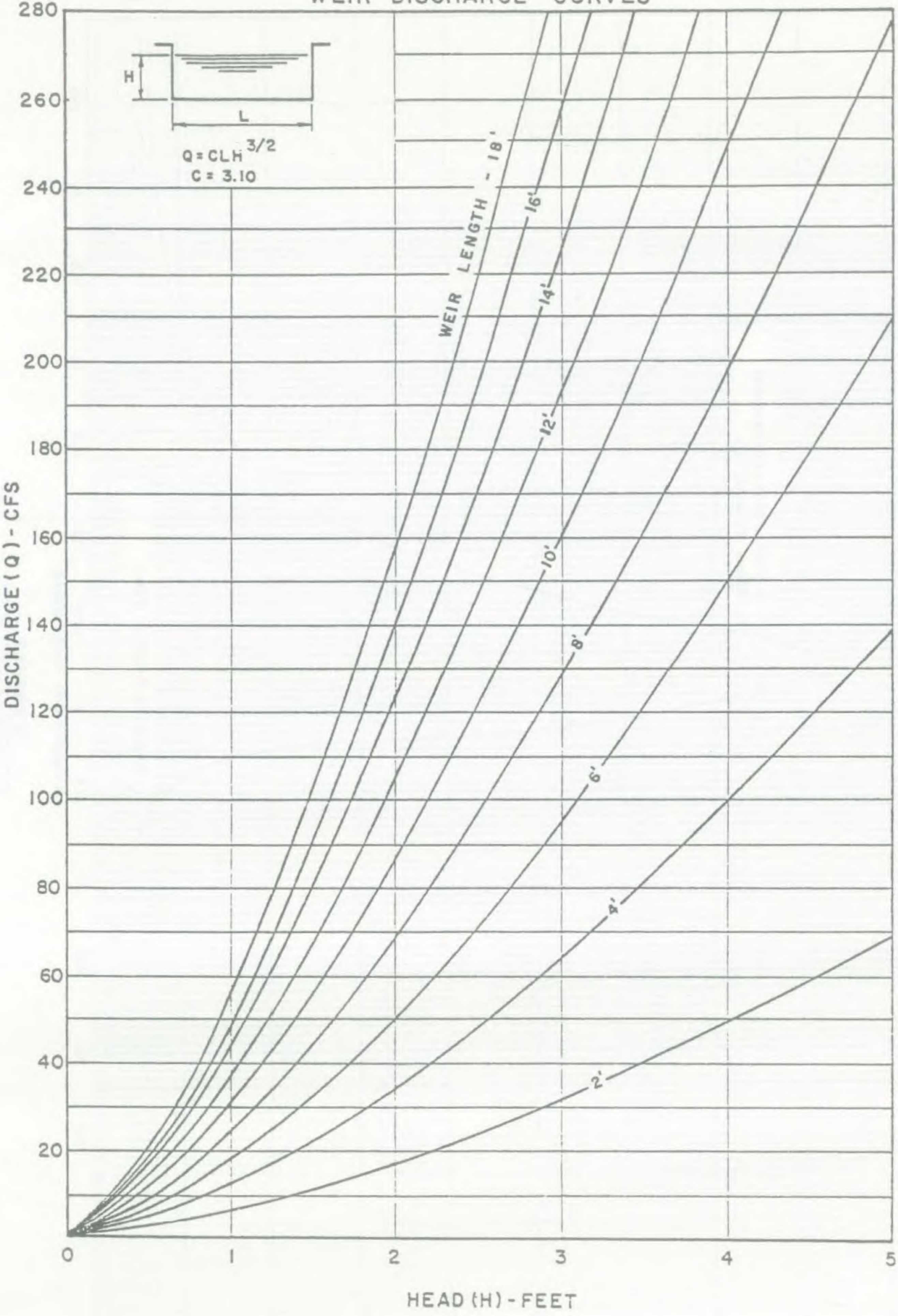
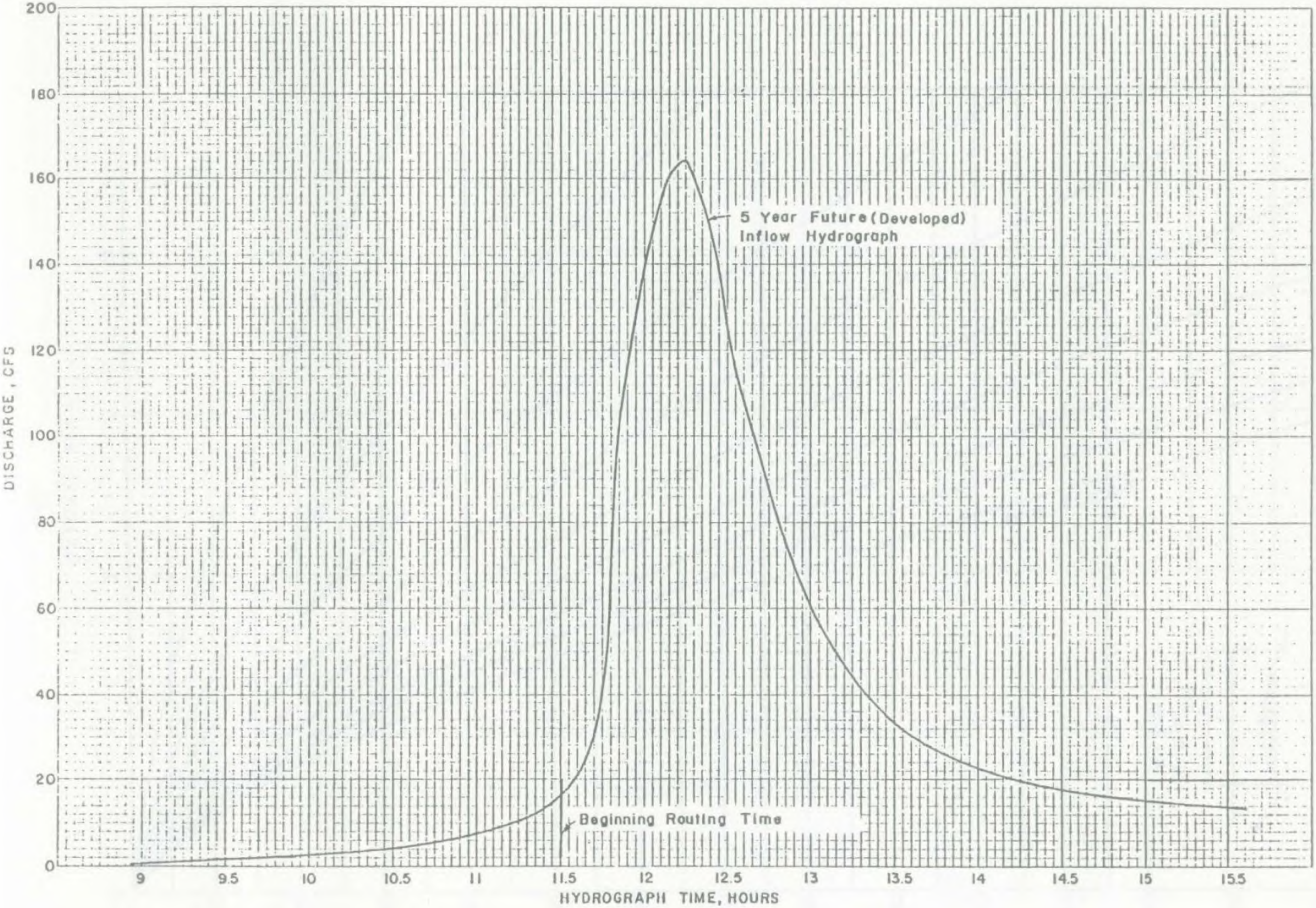

ORIFICE PIPE DISCHARGE CURVES
H<D, Q = 3LH^3/2
H>D, Q = 0.6A√2gh , h = H-D/2
h
D
H
L
PIPE DIAMETER - 36"
30"
24"
21"
18"
15"
12"
6"
DISCHARGE, CFS
HEAD (FEET ABOVE INVERT)
0
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100

EXHIBIT X-7

WEIR DISCHARGE CURVES





EXAMPLE

SUBAREA 7
INFLOW HYDROGRAPH

EXHIBIT X-9

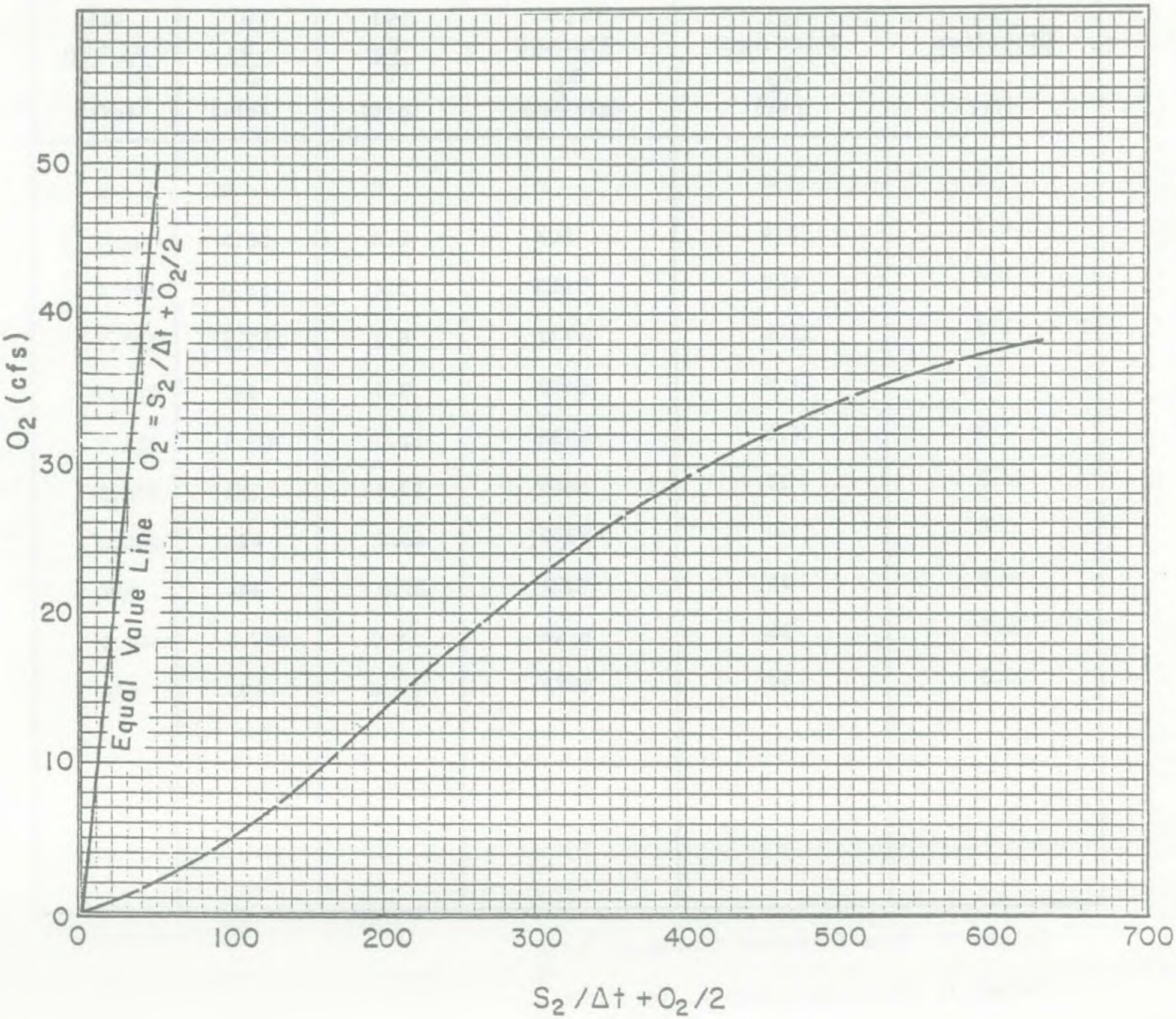
EXAMPLE STORAGE-INDICATION COMPUTATION TABLE

(1) Elevation (ft)	(2) Discharge O_2 (cfs)	(3) Storage S_2 (cfs-min)	(4) $\frac{O_2}{2}$ (cfs)	$\Delta t = 15$ min. (5) $\frac{S_2}{\Delta t}$ (cfs)	(6) $\frac{S_2}{\Delta t} + \frac{O_2}{2}$ (cfs)
0	0	0	0	0	0
0.5	2.4	908	1.2	60.5	61.7
1.0	6.8	1815	3.4	121.0	124.4
1.5	12.4	2722	6.2	181.5	187.7
2.0	18.0	3630	9.0	242	251.0
2.5	24.0	4538	12.0	302.5	314.5
3.0	27	5445	13.5	363	376.5
3.5	32	6352	16.0	424	440.0
4.0	34	7260	17.0	484	508
4.5	36	8168	18.0	544.5	562.5
5.0	38	9075	19.0	605	624

Column 2: From Elevation-Discharge Curve

Column 3: From Elevation-Storage Curve

Columns 4, 5 & 6 are self explanatory



Example Storage-Indication Curve, Subarea 7

EXAMPLE STORAGE-INDICATION OPERATIONS TABLE

(1) Hydrograph Time Hrs.	(2) Routing Interval	(3) Time minutes	(4) Inflow cfs	(5) Ave. Inflow cfs $\bar{I}$	(6) $S_1/\Delta t + O_1/2$ cfs	(7) O_1 cfs	(8) (5)+(6)-(7) $S_2/\Delta t + O_2/2$ cfs	(9) O_2 cfs
11.5		0	16.0	11.4	0	0	0	0
11.75	1.	15	59	37.5	0	0	37.5	1.6
12.0	2.	30	138	98.5	37.5	1.6	134.4	8.0
12.25	3.	45	165	151.5	134.4	8.0	277.9	20.2
12.5	4.	60	126	145.5	277.9	20.2	403.2	29
12.75	5.	75	88	107	403.2	29	481.2	33.2
13.0	6.	90	61	74.5	481.2	33.2	522.5	35.0
13.25	7.	105	44	52.5	522.5	35.0	540.0	36
13.50	8.	120	34	39.	540.0	36.0	543.	36.2
13.75	9.	135	27	30.5	543	36.2	537.3	35.8
14.0	10.	150	23	25.	537.3	35.8	526.5	35.1
14.25	11.	165	20	21.5	526.5	35.1	512.9	34.5
14.50	12.	180	18	19.0	512.9	34.5	497.4	34.0
14.75	13.	195	16.5	17.25	497.4	34.0	480.6	33.0
15.00	14.	210	15.0	15.75	480.6	33.0	463.4	32

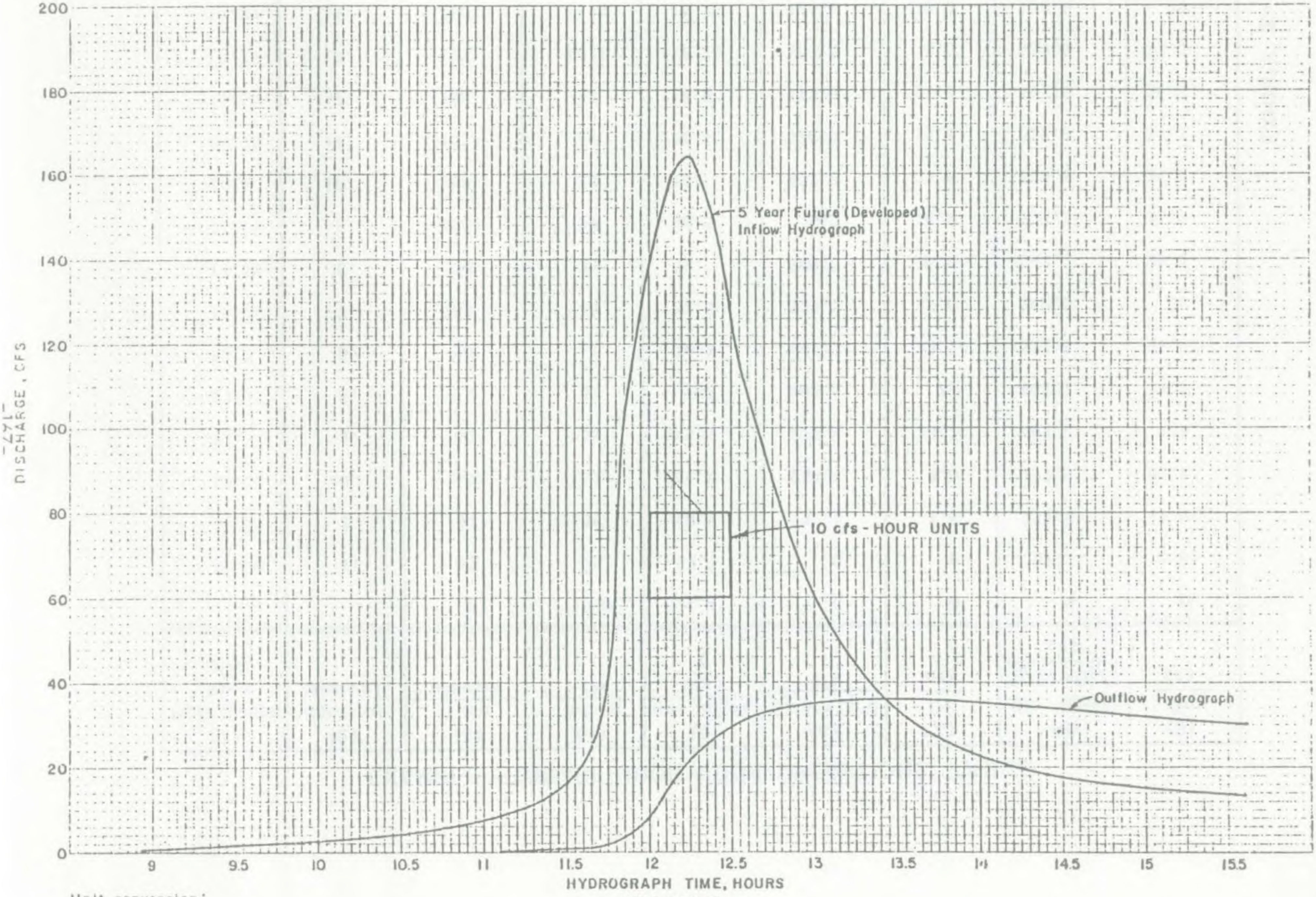


EXHIBIT X-13

Unit conversion;
1 unit = 10 cfs - Hour
= 10 00 ,000 ft. st e

EXAMPLE
SUBAREA 7
RA O

11. FORMATTED WORK SHEETS

11.1 Introduction

This section presents blank formatted work sheets to be used in design of stormwater drainage facilities. Application of the design work sheets is shown and discussed in the general design procedures and example calculations of the respective sections of the manual.

It is intended that the final design of the stormwater drainage facilities be submitted on the work sheets together with necessary supportive data for review and approval by the appropriate agency.

11.2 Listing by Design Manual Section

The following is a list of the work sheets included herein and the appropriate design manual section:

Description	Section
Peak Discharge and Total Runoff Volume Work Sheet	5. Runoff
Tabular Hydrograph Computations	5. Runoff
Pavement Drainage Computations	6. Streets & Inlets
Storm Sewer Design Computations	7. Storm Sewers
Culvert Size Design	8. Culverts
Open Channel Computations	9. Open Channels
Storage-Indication Computation Table	10. Runoff Control
Storage-Indication Operations Table	10. Runoff Control

PROJECT

PAVEMENT DRAINAGE COMPUTATIONS

DESIGNER ______

STORM RETURN PERIOD ______ YEAR

DATE ______

										Gutter Flow				Grate Opening Inlet (Sag)				Curb Opening Inlet (Continuous Grade)								
INLET LOCATION (DESIGN POINT)	LENGTH OF GUTTER FLOW, FEET	DRAINAGE SUBAREA WIDTH	A, INLET DRAINAGE AREA	C, RUNOFF COEFFICIENT	CA	TIME OF CONCENTRATION	I, RAINFALL INTENSITY	ΔQ, SUBAREA DISCHARGE	$Qa = \Delta Q$ + INLET CARRYOVER FLOW	GUTTER SLOPE	CROSS SLOPE	SPREAD	DEPTH AT CURB (d)	$\frac{Qa/p}{H\text{(WEIR FLOW)}}$	$\frac{Qa/A}{H\text{(ORIFICE FLOW)}}$	MAXIMUM HEAD (H)	TYPE OF FLOW	Qa/La CURB OPENING	100% PICKUP LENGTH (La)	$\frac{L}{La}$	a/d CURB OPENING	Q/Qa CURB OPENING	TOTAL FLOW INTERCEPTED (Q)	CARRYOVER FLOW	% PICKUP	TYPE OF INLET
		FT	ACRES		ACRES	MIN	IN/HR	CFS	CFS	FT/FT	FT/FT	FT	FT			FT			FT				CFS	CFS		

-171

PROJECT

STORM SEWER DESIGN COMPUTATIONS

DESIGNER________

LOCATION:

DATE________

							DESIGN FREQUENCY ______ YEAR									n = ____	ELEVATIONS		PRESSURE ELEVATION CALCULATIONS								
																				PRESSURE RISE							
INLET LOCATION (DESIGN POINT)	RUNOFF COEFFICIENT C	DRAINAGE AREA A	CA	ΣCA	INLET OR CONDUIT TIME	TIME OF CONCENTRATION T_c	RAINFALL INTENSITY I	RUNOFF ΣCAI	OTHER CONTROLLED RUNOFF	TOTAL RUNOFF, Q	PIPE LINE DESIGNATION	PIPE DIAMETER	LENGTH, L	SLOPE	VELOCITY HEAD $V^2/2g$	FRICTION SLOPE, S_f OR FLOW DEPTH $[d_n]$	GUTTER AT INLET	INLET or MANHOLE BOTTOM	ELEVATION DOWNSTREAM SIDE OF INLET or MANHOLE	CHART NUMBER	VELOCITY HEAD COEFFICIENT, K	PRESSURE RISE	ELEVATION UPSTREAM SIDE OF INLET or MANHOLE	PIPE LOSS, $h_f = L \times S_f$	ELEVATION AT NEXT UPSTREAM INLET	VELOCITY, V	INLET NUMBER
		ACRES			MIN.	MIN.	IN/HR	CFS	CFS	CFS		IN	FT	FT/FT	FT							FT		FT			

CULVERT SIZE DESIGN

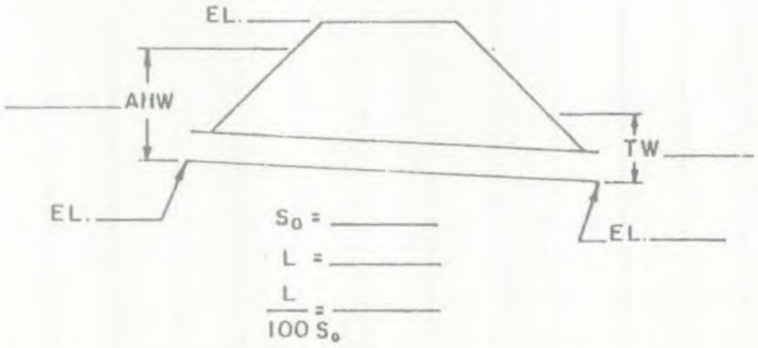
PROJECT:

DESIGNER ____________

DATE ____________

CULVERT DESCRIPTION (ENTRANCE TYPE)	SIZE	Q	HEADWATER COMPUTATIONS: INLET CONT.		OUTLET CONTROL $HW = H + h_o - LS_o$								Controlling HW	OUTLET CONT. Size OK if HW >		OUTLET VELOCITY	COMMENTS
			$\frac{HW}{D}$	HW	Ke	H	d_c	$\frac{d_c + D}{2}$	TW	h_o	LS_o	HW		$D + \frac{(1+Ke)V^2}{2g}$	.75 D		

SKETCH



DATA

Q_1 = ____ TAILWATER ELEV. ____ Q_1 = ____ YR. STM.

Q_2 = ____ TAILWATER ELEV. ____ Q_2 = ____ YR. STM.

FINAL DESIGN

SIZE ____ INLET ELEV ____ SKEW ____ HEADWALL ____

COVER ____ OUTLET ELEV ____ LENGTH ____ SLOPE ____

VELOCITY ____ PROTECTION ____

OPEN CHANNEL COMPUTATIONS

DESIGNER ________
DATE ________

PROJECT ____________________

DISCHARGE								VELOCITY					CRITICAL FLOW				
1 Q	2 n_Q	3 s	4 $Ar^{2/3}$	5 b,c	6 T	7 d_f	8 V	9 n_V	10 $Ar^{2/3}$	11 A	12 $V_{max.}$	13 V	14 Z	15 $Z/b^{2.5}$	16 d_c/b	17 d_c	18 V_c

$Ar^{2/3} = nQ/1.49\sqrt{s}$ $V = Q/A$ $Z = Q/\sqrt{g}$

STORAGE-INDICATION COMPUTATION TABLE

				$\Delta t =$ min.	
(1) Elevation (ft)	(2) Discharge O_2 (cfs)	(3) Storage S_2 (cfs-min)	(4) $\frac{O_2}{2}$ (cfs)	(5) $\frac{S_2}{\Delta t}$ (cfs)	(6) $\frac{S_2}{\Delta t}+\frac{O_2}{2}$ (cfs)

STORAGE-INDICATION OPERATIONS TABLE

(1) Hydrograph Time Hrs.	(2) Routing Interval	(3) Time / minutes	(4) Inflow / cfs	(5) Ave. Inflow / cfs / $\overline{I}$	(6) $S_1/\Delta t + O_1/2$ / cfs	(7) O_1 / cfs	(8) (5)+(6)-(7) / $S_2/\Delta t + O_2/2$ / cfs	(9) O_2 / cfs

APPENDIX A

HYDRAULIC DESIGN OF STORM SEWERS

This appendix contains charts for determining the pressure changes of storm sewer junctions as well as design procedures for both open channel and pressurized conduit design.

Source: "Urban Storm Drainage Criteria Manual, Volume 1," Denver Regional Council of Governments, Wright-McLaughlin Engineers, Denver, Colorado, March, 1969.

8. HYDRAULIC DESIGN OF STORM SEWERS

Final hydraulic design of a storm sewer system should have as its objective the design of a balanced system in which all portions will be used to their full capacity without adversely effecting the drainage of any area. Although design flows are dependent upon assumptions that do not represent the actual conditions under which the sewer will usually operate, the designer must not be tempted by the inherent limitations of the basic flow data to become "sloppy" in hydraulic design. (2)

Hydraulic calculations will usually be based upon the design initial runoff. It may be advisable to also analyze lower flows for sediment carrying capacity and higher flows for the possibility of the hydraulic grade exceeding the ground elevation with resulting discharge from the sewer. (2)

8.1 Rules of Thumb

Numerous rules of thumb exist for designing storm sewer systems. These are probably used because there is a conspicuous lack of usable design data for situations actually encountered.

The design methods presented in this Manual should be utilized whenever applicable. From the number and variations of the rules of thumb, it can be seen how designs based upon them could vary substantially; and therefore, how likely the systems are to be incorrectly designed.

1. Set crown of pipes at same elevation. (2)
2. Set crown of smaller pipe above the invert of the larger downstream sewer by an amount not less than 0.8 of the vertical height of the larger sewer. (5)
3. Keep the 0.8 depth line continuous. (2)
4. Make invert drop equal to one-half the difference in sewer diameters for sewers smaller than 24 inches, and 3/4 the difference for sewers larger than 24 inches. (2)
5. Use increased coefficient of roughness such as n = 0.015 instead of 0.013. (2)
6. Allow a drop of 0.1 feet in a through manhole, of 0.2 feet in the presence of one lateral bend, and 0.3 feet for two laterals. (2)

8.1 Continued

7. Set the 0.75 depth line continuous. (6)

8. If a sewer changes direction in a manhole without changing size, a drop of 0.04 ft. should be provided. (7)

8.2 Pipe Friction Losses

Numerous formulas for flow in conduits are available. Manning's, Kutter's, or Hazan-Williams' are all applicable under certain conditions. Care should be exercised to verify that the actual condition applies to the formula being utilized.

A key to the correctness of any formula is the selection of a proper roughness coefficient. Numerous tests have been conducted with the aim of establishing roughness coefficients. (8, 9) Coefficients are listed elsewhere in this Manual in terms of Manning's n value.

Variation of roughness coefficients with depth of flow is well established and it is now recommended that this be reflected in design. (2) Figure 8-1 shows the hydraulic elements of circular conduits for both variable and constant values of Manning's n with depth.

8.3 Open Channel Design

The following design procedure is applicable to storm sewers flowing with a free water surface. Open channel flow is usually not economical when circular conduits are involved. Although it is theoretically possible for a pipe flowing 95% full to carry more than at 100% full for the same slope, it is from a practical standpoint impossible since any trash accummulation, junction, or other impediment would cause it to flow full.

8.3.1 Range of Applicability. Based on the assumption that Manning's n varies with depth, the capacity of a circular conduit at a given grade is the same at 91% and 100% ratios of d/D. Since it is impractical to design for the theoretical range where capacity exceeds that for the full conduit, open channel flow should only be assumed below d/D = 90%.

Figure 8-2 illustrates how conduit flow conditions may be roughly determined. The design procedure in the following paragraphs are based upon the same assumptions of straight water surfaces. Where sizes of conduits are sufficiently large, or other needs for a higher degree of accuracy exist, backwater or drawdown curves should be calculated.

8.3.2 Design Procedure. The basic approach to design of open channel flow in storm sewers should be to calculate the energy grade line along the system. (2, 10) The assumption is made that the energy grade line is parallel to the pipe grade, and that any losses other than pipe friction may be accounted for by assuming point losses at each manhole.

Once the discharge has been determined and a pipe size and slope assumed for a given section, the d/D and v/V_{full}, ratios can be determined from a graph of Hydraulic Elements for Circular Conduits such as Figure 8-1. Figure 8-1 applies only to circular cross sections.

The next step is to calculate the energy grade line.

$$H = Z + d + (V^2/2g) \qquad (8\text{-}1)$$

At each manhole the energy grade lines of all pipes should coincide, allowing for reasonable values of head loss to the junction. Under certain conditions, this would indicate an upstream invert lower than the downstream invert. Inverts should be set at the same elevations under such circumstances.

The usual method of stating head losses at manholes is in terms of a constant K times the velocity head of the conduit in question, $h_e = (K)V^2/2g$. A difficulty in design of systems is the determination of the value of K.

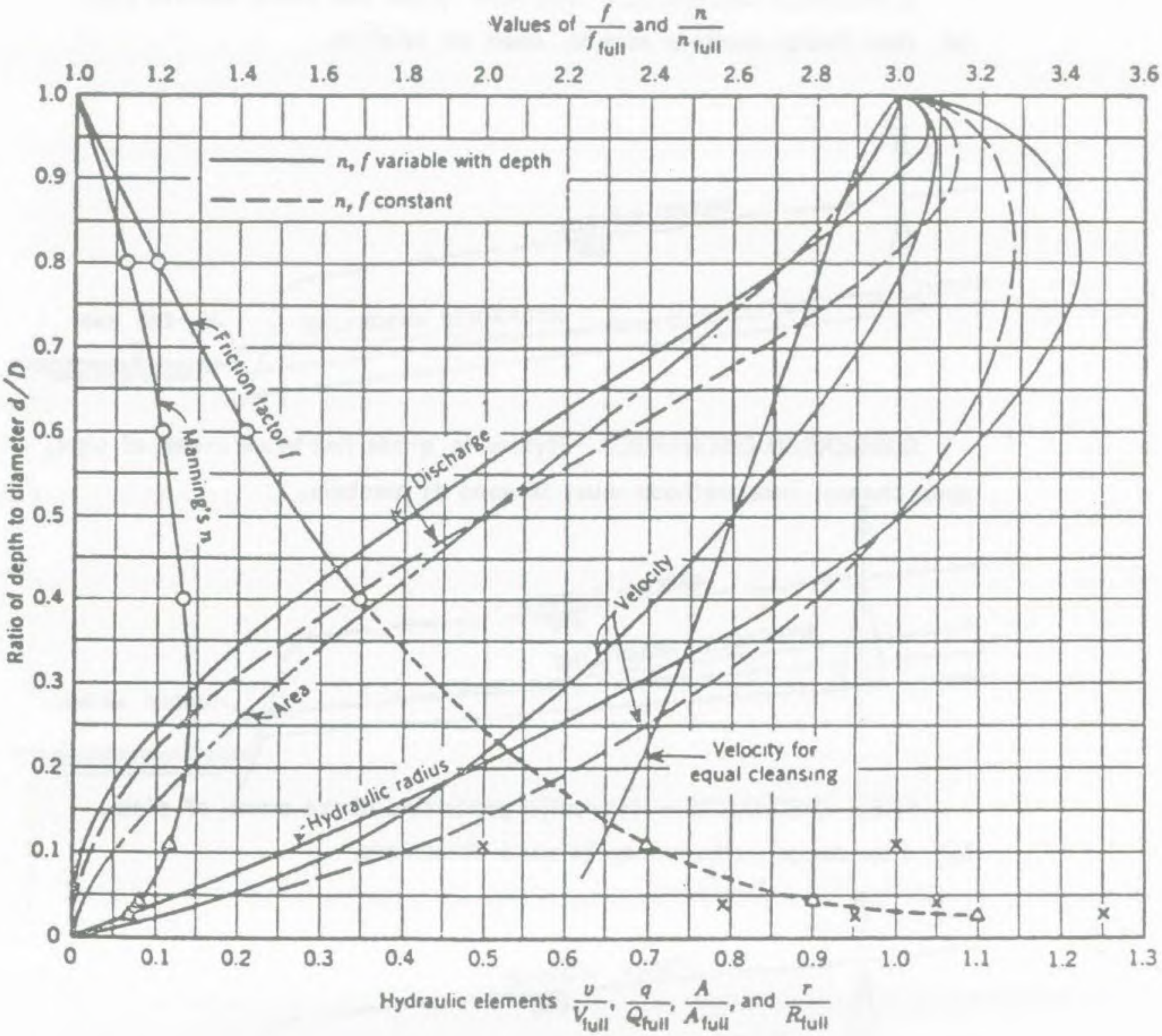
8.3.2a Simple Transitions in Pipe Size. Simple transitions in conduit size in a manhole with straight through flow may be analyzed by the following equation:

$$h_e = K \triangle (V^2/2g) \qquad (8\text{-}2)$$

The term $\triangle(V^2/2g)$ refers to the change in velocity head in the upstream and downstream conduits. The value of K varies from 0.1 for increasing velocity to 0.2 for decreasing velocity transitions if flow is sub-critical. For super-critical flow, greater values of K are probable, but have not been determined. (11)

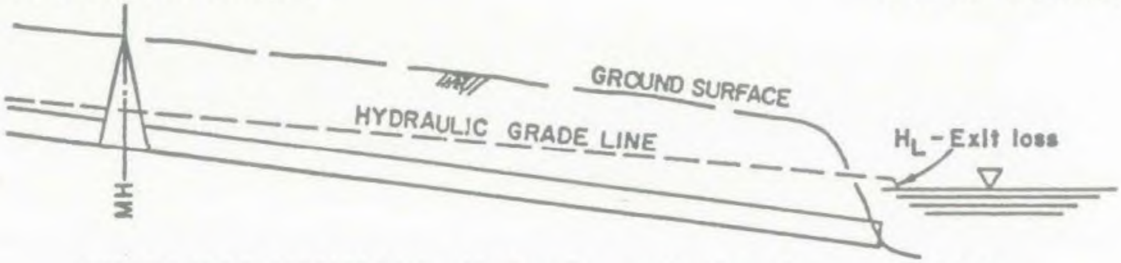
8.3.2b Bends. Reliable headloss coefficients through bends in open channel flow are almost entirely lacking. Reasonable assumptions may be made by utilizing existing information available on losses in bends in pressure conduits. (2)

8.3.2c Junctions. Values for head loss coefficients at junctions on storm sewers flowing as open channels are not readily available. Complicated methods for calculating head loss atcertain types of junctions are available and are justified for certain situations. A chapter in Street and Highway Drainage, Volume 2, Design Charts, (12) is devoted to Hydraulic Analysis of Junctions. Additional information is contained in references 13 and 14. Major junctions for which these methods are not applicable should be model tested.

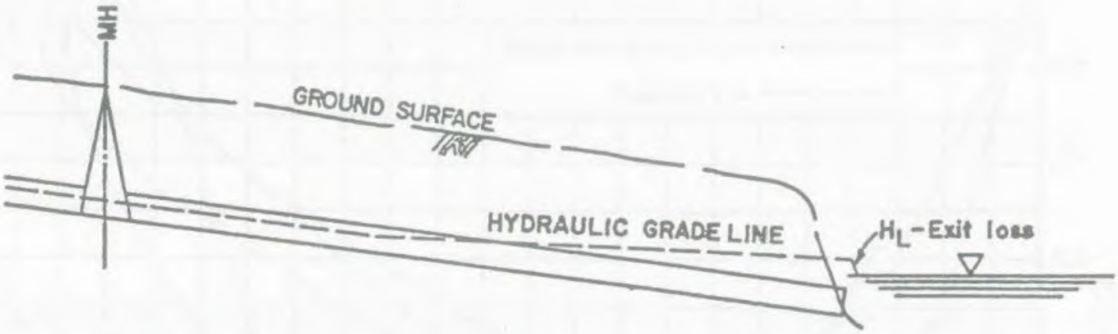


v = Actual velocity of flow (fps)		A = Area occupied by flow (ft.^{2})	
V_{full} = Velocity flowing full (fps)		A_{full} = Area of pipe (ft.^{2})	
q = Actual quantity of flow (cfs)		r = Actual hydraulic radius (ft.)	
Q_{full} = Capacity flowing full (cfs)		R_{full} = Hydraulic radius of full pipe (ft.)	

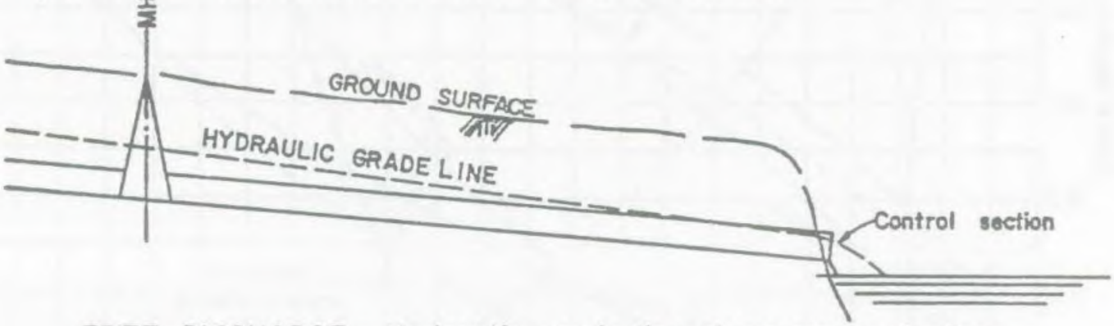
FIGURE 8-1. HYDRAULIC ELEMENTS OF CIRCULAR CONDUITS (2)



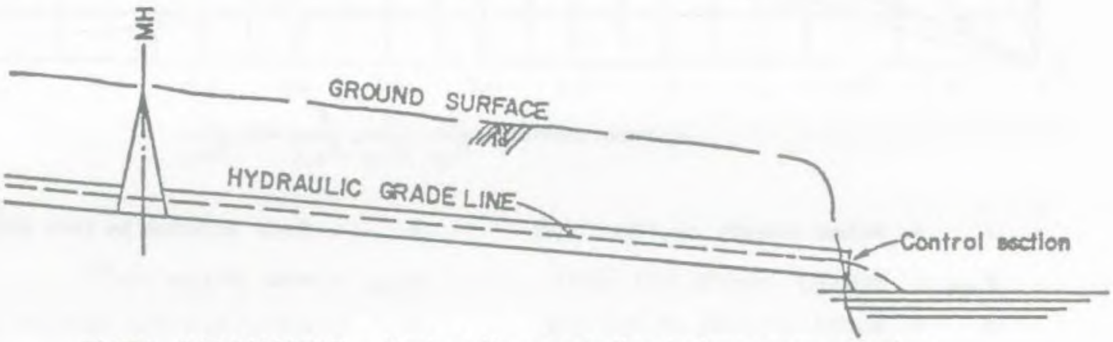
SUBMERGED DISCHARGE – Hydraulic grade line above crown of pipe, full flow design methods may be used at manhole.



SUBMERGED DISCHARGED – Hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole.



FREE DISCHARGE – Hydraulic grade line above crown of pipe, full flow design methods may be used at manhole.



FREE DISCHARGE – Hydraulic grade line below crown of pipe, open channel flow methods must be used at manhole.

FIGURE 8–2. DETERMINING TYPE OF FLOW

8.3.2d Storm Water Inlets. The design methods for culverts acting under inlet control presented in the Culverts & Inlets Part of this Manual are applicable when designing catch basin water depths where the connector pipe to the sewer is flowing partly full. Care should be exercised to verify that the required headwater depth to attain the design flow in the conduit does not come within 6 inches of the gutter elevation at the inlet.

8.4 Pressurized Storm Sewers

Storm sewers are sometimes planned as pressure conduits for the design storm runoff. The fact that such storm sewers usually have manhole and storm inlet appurtenances which provide a direct hydraulic connection to the street surface above means that special care must be taken in the hydraulic analysis of such sewers.

It is evident that if the hydraulic grade line rises above the ground surface, storm inlets will not function, and storm water will actually be inadvertently discharged from the storm sewer to the street surface via the inlets and manholes. This is a frequent cause of "popping" manhole covers. Sometimes manholes are designed for a higher than street level hydraulic gradient with limited storm inlet connections.

Often a gravity flow open channel storm sewer is planned which operates as a pressure storm sewer when a storm runoff occurs which is greater than that used for design purposes. Here the principles presented in this Chapter may be applied to obviate excessive and inadvertent flooding of a neighborhood. Over design of upstream storm inlets, which is encouraged as good practice, can lead to this latter situation..

The following design procedures are applicable where it is found advisable or necessary to have storm sewers flow full as pressure conduits. Checks must continually be made to verify if the conduit is in fact flowing full. Often the storm sewer will alternate between flowing full and open channel flow from one stretch to another. In this case, it will be necessary to establish type of flow and design accordingly.

The design procedures for pressure conduits presented in this section are largely derived from "Pressure Changes of Storm Drain Junctions," (15). The nomenclature used for explanation of the method for the design is summarized in Figure 8-3.

8.4.1 Allowable Pressures. Two major considerations limit the maximum pressure which may be allowed in a sewer. First, the structural limitations must not be exceeded for a given pipe. When considering structural limitations both the pipe and the joint must be analyzed. Second, the hydraulic grade line must be maintained below ground level, unless special consideration is taken to prevent water from escaping from sewers or to handle it once it does escape.

8.4.1 Continued

A further limitation is to allow a sufficient drop into inlets to allow them to function properly.

8.4.2 Determining Type of Flow. Before treating a conduit as flowing full, checks must be made to determine the type of flow. To do this, calculations must proceed upstream, varifying progressively that the hydraulic grade line is above the crown of the pipe.

8.4.2a Discharge Point. The discharge point of the sewer usually establishes a starting point. If the discharge is submerged, as when the water level of the receiving carrier is above the crown of the sewer, the exit loss should be added to the water level and calculations for head loss in the sewer started from this point as illustrated in Figure 8-2. If the hydraulic grade line is above the pipe crown at the next upstream manhole, full flow calculations may proceed. If the hydraulic grade line is below the pipe crown at the upstream manhole, then open channel flow calculations must be used at the manhole.

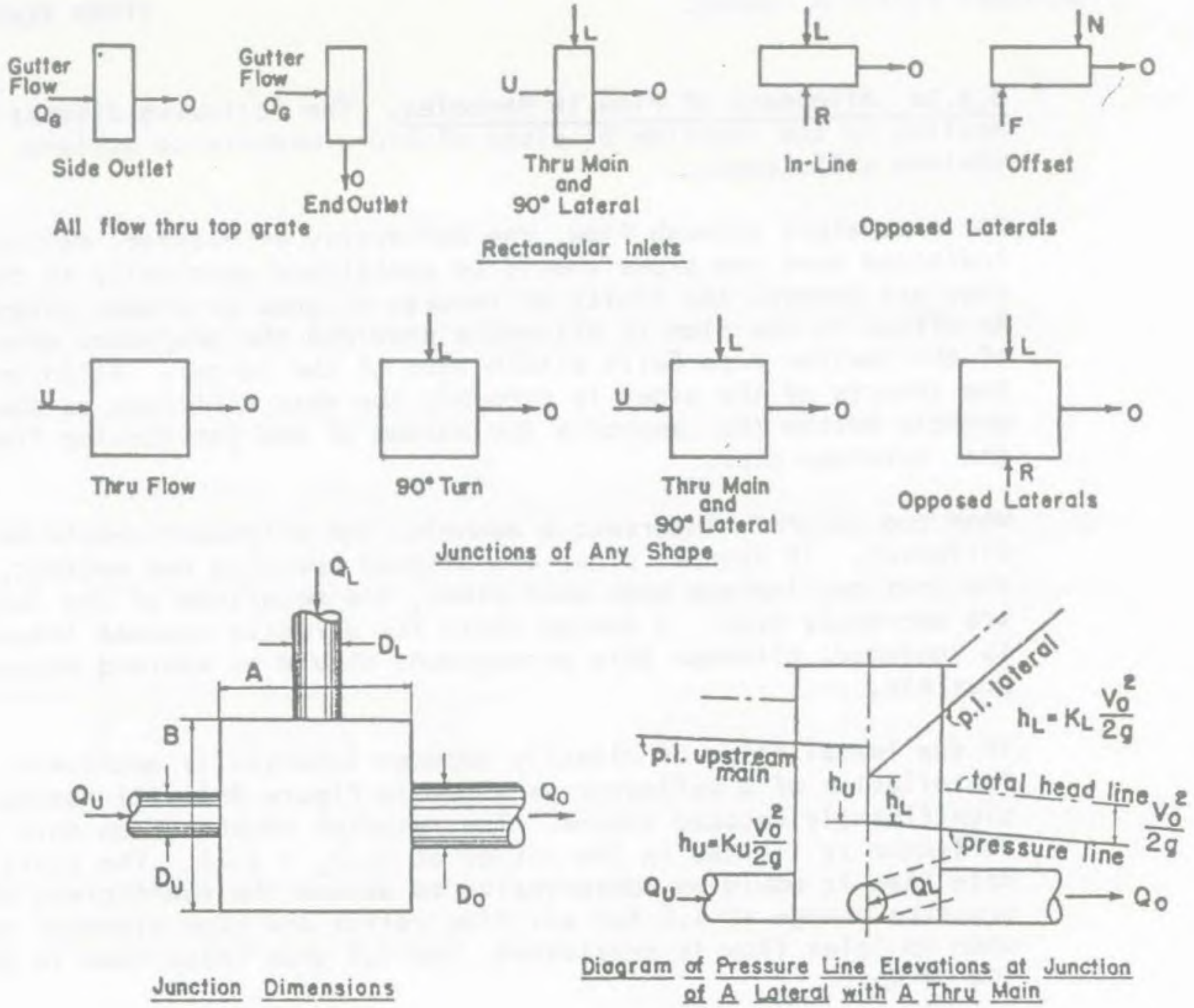
When the discharge is not submerged, a flow depth must be determined at some control section to allow calculations to proceed upstream. As illustrated in Figure 8-2, the hydraulic grade line is then projected to the upstream manhole. Full flow calculations may be utilized at the manhole if the hydraulic grade is above the pipe crown.

The assumption of straight hydraulic grade lines as shown in Figure 8-2 is not entirely correct, since backwaters and drawdowns exist, but should be accurate enough for the size pipes usually considered as storm sewers. If the designers feel that additional accuracy is justified, as with very large conduits or where the result will have a very significant effect on design, backwater and drawdown curves may actually be calculated.

8.4.2b Within System. At each manhole the same type procedure as outlined for the discharge must be repeated.

The water depth in each manhole must be calculated to verify that the water level is above the crown of all pipes. Whenever the level is below the crown of a pipe full flow methods are not applicable.

8.4.3 Manhole Construction. The construction details of manholes or systems designed to flow full should deviate somewhat from manholes for open channel flow systems.



Nomenclature

Q		rate of flow
D		diameter of pipe
A		dimension of junction in direction of outfall pipe
B		dimension of junction at right angles to outfall pipe
d		depth of water in inlet
S		slope of pipe
S_f		friction slope
Q_G		flow into inlet thru top grate
D_O	Q_O	dia. and flow in outfall
D_U	Q_U	dia. and flow in upstream main
D_L	Q_L	dia. and flow in left lateral
D_R	Q_R	dia. and flow in right lateral
D_N	Q_N	dia. and flow in near lateral } of opposed laterals; designation with reference to position relative to outfall end of junction.
D_F	Q_F	dia. and flow in far lateral } of opposed laterals; designation with reference to position relative to outfall end of junction.
$D_{h.v.}$	$Q_{h.v.}$	dia. and flow in lateral with higher-velocity flow } for in-line opposed laterals only
$D_{l.v.}$	$Q_{l.v.}$	dia. and flow in lateral with lower-velocity flow } for in-line opposed laterals only

Pressure change coefficients for inlet water depth and on upstream pipe pressure relative to the outfall pipe pressure.

- K_G water depth with all flow thru grate
- K_U upstream main pressure
- K_R or K_L lateral pipe pressure
- K_N near lateral pipe pressure } offset opposed laterals
- K_F far lateral pipe pressure } offset opposed laterals
- $\overline{K_U}$, $\overline{K_L}$ pressure coefficient at $Q_L = Q_O$
- M_U, M_L multipliers for $\overline{K_U}$ or $\overline{K_L}$ to obtain K_U or K_L

FIGURE 8-3. MANHOLE JUNCTION TYPES & NOMENCLATURE (15)

8.4.3a Alignment of Pipe in Manholes. The following discussion applies to the location of pipes within a manhole to achieve maximum efficiency.

For a straight through flow, the University of Missouri research indicated that the pipes should be positioned vertically so that they are between the limits of inverts aligned or crowns aligned. An offset in the plan is allowable provided the projected area of the smaller pipe falls within that of the larger. Aligning the inverts of the pipes is probably the most efficient as the manhole bottom then supports the bottom of the jet issuing from the upstream pipe.

When two laterals intersect a manhole, the alignment should be quite different. If lateral pipes are aligned opposite one another, so the jets may impinge upon each other, the magnitude of the losses are extremely high. A design chart for directly opposed laterals is included, although this arrangement should be avoided wherever possible.

If the installation of directly opposed laterals is necessary, the installation of a deflector as shown in Figure 8-4 will result in significantly reduced losses. The research conducted on this type deflector is limited to the ratios of $D_0/D_L = 1.25$. The tests indicate that it would be conservative to assume the coefficient of pressure change at 1.6 for all flow ratios and pipe diameter ratios when no inlet flow is considered, and 1.8 when inlet flow is over 10% of Q_0.

Lateral pipes should not be located directly opposite; rather, their centerlines should be separated laterally by at least the sum of the two lateral pipe diameters. Reference to the design figures show that head losses are definitely reduced as compared to directly opposed laterals, even with deflectors. Insufficient data has been collected to determine the effect of offsetting laterals vertically.

8.4.3b Shaping Inside of Manhole. The fact that jets issue from the upstream and lateral pipes must be considered when attempting to shape the inside of manholes.

The tests for full flow revealed that very little, if anything, is gained by shaping the bottom of a manhole to conform to the pipe invert. Shaping of the invert may even be detrimental when lateral flows are involved, as the shaping tends to deflect the jet upwards, causing unnecessary head loss. Limited shaping of the invert to handle low flows is necessary from a practical point of view.

8.4.3b Continued

Figure 8-4 details several types of deflector devices that have been found efficient in reducing losses at junctions and bends. In all cases, the bottoms are flat, or only slightly rounded, to handle low flows. Numerous other types of deflectors or shaping of the manhole interiors were tested by the University of Missouri. Some of these devices which were found insufficient are shown in Figure 8-5. The fact that several of these inefficient devices would appear to be improvements indicates that special shapings deviating from those in Figure 8-4 should be used with caution, possibly only after model tests.

8.4.3c Entrances. The tests showed that rounding entrances or the use of pipe socket entrances do not have the effect on reducing losses that might be expected. Once again, the effect of the jet from the upstream pipe must be considered. Specific reductions to the pressure change factors are indicated with each design chart.

8.4.4 Catch Basins. Certain specific design procedures are necessary when designing catch basins for storm water inlets on systems flowing full.

The design water surface should be at least 6 inches below the gutter grade at the inlet to allow the inlet to function properly. If there is any possibility of the hydraulic grade being above this level, the inlet should be considered not to accept any flow. Under unusual cases, the hydraulic grade may exceed the inlet elevation, allowing flow to escape from the system. Methods of dealing with this water must then be included in the design.

8.4.5 Design Procedures. General procedures for establishing quantities of flow and horizontal layout are the same for both open channel and pressure conduits. Once these criteria have been set, the following procedure allows computations for required pipe size and appurtenance construction to be pursued with the degree of accuracy justified by the cost of subsequent construction.

In storm drain systems flowing full, all losses of energy through resistance to flow in pipes or by changes in momentum and interference with flow patterns at manholes must be compensated by an accumulative series of rises of pressure along the system from its outlet to its initial upstream inlet, or until open channel flow conditions interrupt the calculations. Losses due to pipe flow resistance may be calculated by any of a variety of methods, so long as an appropriate roughness factor for the conduit is used.

The lack of a reasonable method for calculation of losses at junctions has long hindered the design of storm sewers flowing full. The charts

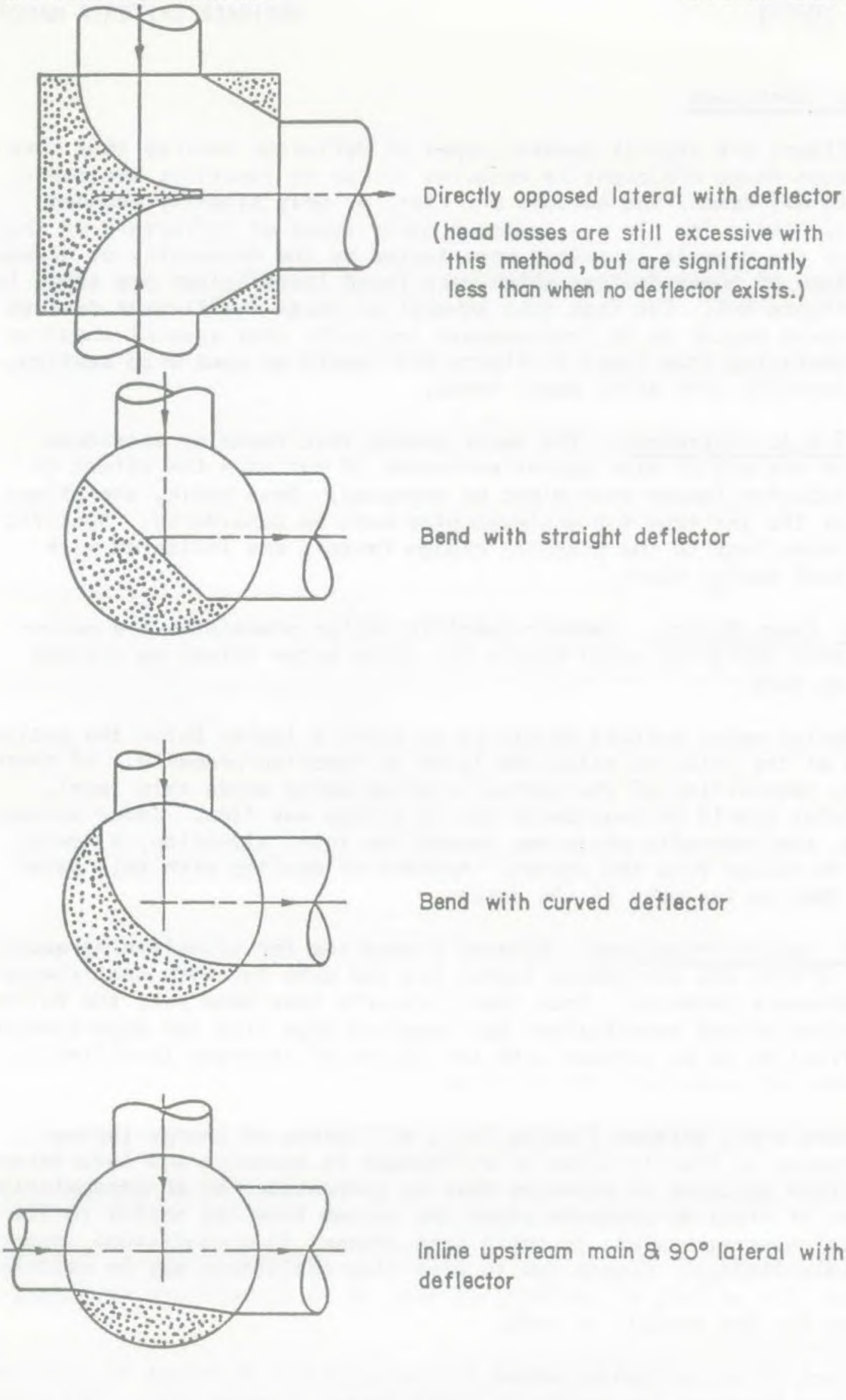
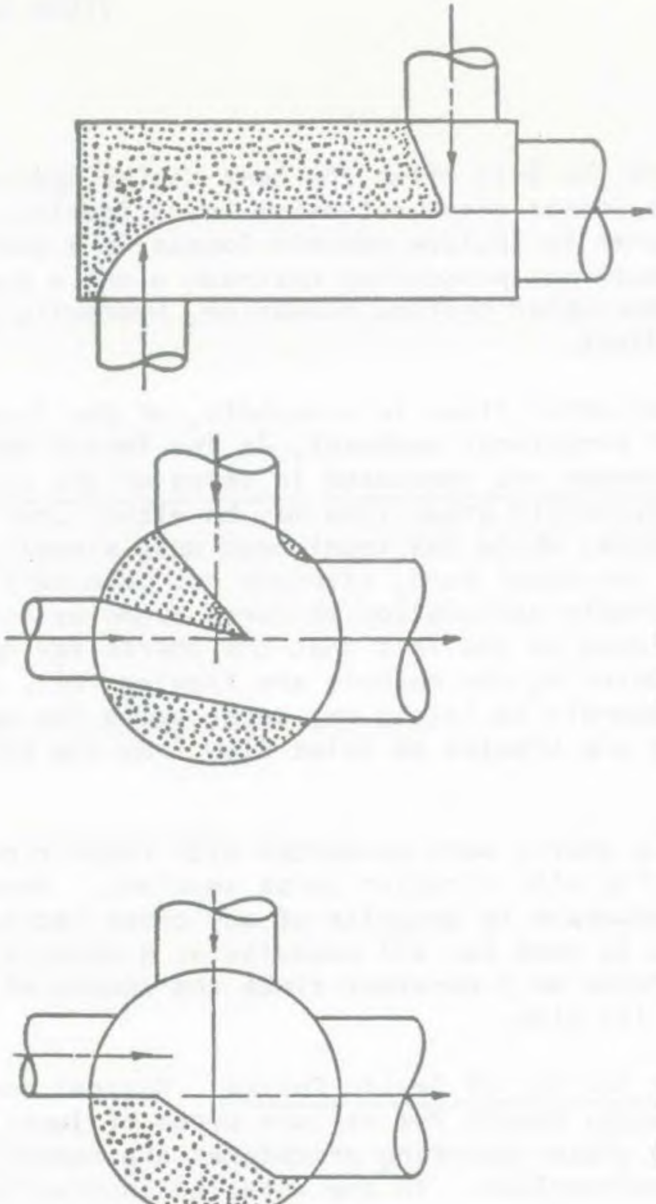


FIGURE 8-4. EFFICIENT MANHOLE SHAPING

The above methods of shaping the interior of a manhole were found efficient by University of Missouri tests.



Offset lateral with deflector

Inline upstream main & 90° lateral with divider

Inline upstream main & 90° lateral with deflector

FIGURE 8-5. INEFFICIENT MANHOLE SHAPING

The above methods of shaping the interior of a manhole were found inefficient by University of Missouri either due to increased head loss or tendency to plug with trash.

8.4.5 Continued

In this section, Figures 8-6, to 8-15 offer the best design approach the writer has been able to locate after extensive investigation. These charts enable a designer to include manhole losses in a progressive calculation of pressure elevations proceeding upstream along a storm sewer system, determining the water surface elevation, hydraulic grade line, and total energy gradient.

Since the elevation to which water rises in a manhole, or the internal pressure on the conduit for structural purposes, is the factor most important in design, the changes are expressed in terms of the total head line. The change in hydraulic grade line may be either positive or negative at a given manhole, while the total head must always drop due to losses. Therefore, the total head, although not necessary for calculations, may be a desirable calculation to carry along as a check. Special emphasis must be placed on the fact that the charts are only applicable when all pipes entering the manhole are flowing full, except that small pipe, such as laterals to inlets may enter above the water surface, in which case they are treated as inlet flow from the ground surface.

The tests for developing the charts were conducted with round pipes, and apply strictly to conduits with circular cross sections. However, the charts are relatively accurate to conduits of any cross section if the same type of conduit is used for all conduits at a manhole, and if the conduit areas are stated as a constant times the square of the dimension used to identify its size.

8.4.6 General Instructions for Use of Design Charts. Several operations are common to use of the design charts for various types of junctions. Instructions for performing these recurring procedures are consolidated in the following General Instructions. In the detailed instructions for use of the individual charts, references to these General Instructions are made by number (Gen. Instr. 1, etc.). The general Instructions are as follow:

1. Determine and tabulate the elevation of the outfall pipe pressure line at the branch point or inlet center (refer to Figure 8-3). This elevation is obtained by adding to the elevation of the pressure line at the preceding structure downstream the pipe friction loss.

2. Calculate the mean velocity head of the flow in the outfall pipe.

$$\frac{V_o^2}{2g}$$

3. Calculate the required flow rate and size ratios. Examples: Q_U/Q_O, Q_L/Q_O, Q_G/Q_O, etc. D_U/D_O, D_L/D_O, B/D_O, etc.

4. Estimate the depth of water, d, in a manhole with flow into the

8.4.6 Continued

manhole from a top inlet, either alone or combining with flow from an upstream pipe.

d = total depth of water, in ft.
  = (outfall pressure line elevation minus inlet bottom elevation) + (K) $V_0^2/2g$.
K = the pressure change coefficient for the inlet water depth. (This is estimated as detailed for each type of manhole. Such estimates are not necessary for manholes with in-line or offset opposed laterals.)

5. Use the coefficients K from the charts for manholes with square-edged entrance to the outfall pipe (entrance flush with box side, with square edges).

6. Use reduced coefficients K, where applicable, for a rounded entrance to the outfall pipe (rounded on 1/4 circle arc of approximate radius 1/8 D_0) or for an entrance formed by the socket end of a standard tongue-and-groove concrete pipe.

   Fig. 8-6 - insignificant effect, make no reduction.
   Fig. 8-7 - read directly from chart.
   Fig. 8-8 - reduce K_U by 0.1 for usual proportions of inlet flow; by 0.2 for Q_G about 0.5.
   Fig. 8-9 - reduce K_U and K_L in same manner as Fig. 8-8.
   Fig. 8-10- insignificant effect, make no reduction.
   Fig. 8-11- insignificant effect, make no reduction.
   Fig. 8-12, 13, 14- see specific instructions for each case.

7. Calculate pressure change.

   To calculate the change of pressure at a manhole, working upstream from the outfall pipe to an upstream pipe, the design chart applying to the type of junction involved is selected. The pressure change coefficient for a specific upstream pipe is read from the chart for the particular flow rate and size ratios already calculated. The pressure change is calculated from $h = K \times \frac{V_0^2}{2g}$. The coefficient is a dimensionless number, and therefore, the change of pressure will be in feet.

8.4.6 Continued

8. Apply the pressure change.

   The pressure change, in feet, for each upstream pipe is added to the outfall pipe pressure line elevation at the branch point to obtain the elevation of each pressure line for further calculations upstream along that pipe. In some cases the upstream pressure line at the branch point will be at a lower elevation than the downstream pressure line. Where this less common situation may occur with a particular type of junction, it is mentioned in the instructions for use of the specific chart.

9. Determine the elevation of the water surface.

   The elevation of the water surface in a manhole (with or without inlet flow) receiving flow from a pipe or pipes will correspond to that of the upstream in-line pipe pressure line. At a junction with offset opposed laterals, the water surface will correspond to the elevation of the far lateral pipe pressure line. At a junction with in-line opposed laterals, the water surface will correspond to the elevation of the pressure line of the higher-velocity lateral pipe.

   Verify that the water surface is above the crown elevation of all pipe connections to the structures that are being analyzed. Small pipes, such as laterals to inlets, which carry a small portion of the total flow, may reasonably be considered to effect a manhole in the same way as inlet flow from the ground surface.

The various cases are summarized below:

Case	Paragraph	Figure
Catch Basin with Inlet Flow Only	8.4.6a	8-6
→ Flow Straight Through Any Manhole	8.4.6b	8-7
Rectangular Manhole, Thru Pipe & Inlet Flow	8.4.6c	8-8
Rectangular Manhole with In-Line Upstream Main and 90° Lateral Pipe (with or without inlet flow)	8.4.6d	8-9
Rectangular Manhole with In-Line Opposed Lateral Pipes each at 90° to Outfall (with or without Inlet Flow)	8.4.6e	8-10
Rectangular Manhole with Offset Opposed Lateral Pipes each at 90° to Outfall (with or without Inlet Flow)	8.4.6f	8-11
Square Manhole at 90° Deflection	8.4.6g	8-12
Round Manhole at 90° Deflection	8.4.6h	8-12

8.4.6 Continued

8.4.6a Figure 8-6. Pressure change coefficients are presented in this Figure for use in determining the elevation of the water surface in a catch basin with all inflow entering through an inlet. Separate curves are included for the outfall pipe connected at the box end (short dimension) and the box side (long dimension). The coefficient K_G depends on the pipe position and the depth of water in the inlet.

To use the Figure:

1. Note whether outlet is at end or side.
2. Determine outfall pipe pressure line elevation--Gen. Inst. 1.
3. Calculate outfall velocity head--Gen. Instr. 2.
4. Estimate a value for water depth d.
   a. outfall pressure line elevation minus inlet bottom elevation plus $K_G \frac{V_0^2}{2g}$ equals d.
   b. Estimate K_G as follows:
      7.0 for end outlet, 5.0 for side outlet — pressure line to bottom not over 2 pipe diameters
      4.0 for end outlet, 3.0 for side outlet — for higher pressure lines
5. Calculate the estimated relative water depth d/D_0.
6. Enter Fig. 8-6 at this depth d/D_0 and read K_G from the curve for the particular outfall pipe location.
7. Calculate h_G as indicated on the diagram on the chart and by Gen. Instr. 7.
8. Add h_G to the elevation of the outfall pressure line at the inlet center to obtain the water surface elevation in the inlet.
9. From this water surface elevation subtract the elevation of the inlet bottom to obtain a more precise value for the water depth d.
10. Repeat the above procedure with the improved value of d from step (9), if necessary. Such repetition may not be necessary if the estimated d/D_0 of step (5) was reasonably accurate.
11. Check to be sure that the inlet water elevation is below the gutter elevation at the inlet so that inflow may be admitted.

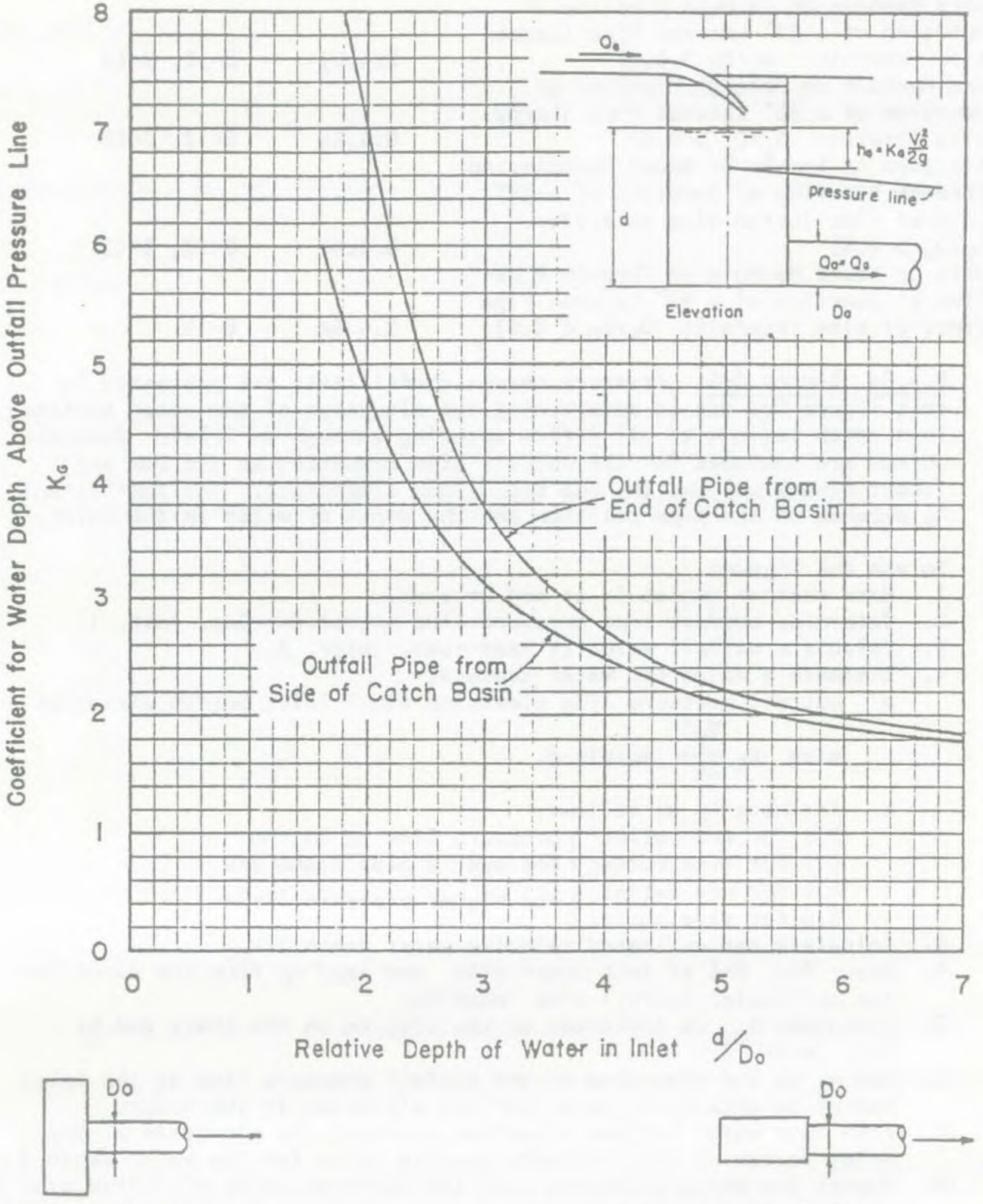


FIGURE 8-6. CATCH BASIN WITH INLET FLOW ONLY (15)

8.4.6b Figure 8-7. Pressure change coefficients are presented in this Figure for use in determining the elevation of the pressure line of an upstream in-line pipe relative to that of the outfall. The pipe centerlines must be parallel and not offset more than would permit the area of the smaller pipe to fall entirely within that of the larger if protected across the junction box along the pipe axis The shape of the junction in plan is not significant in determining the pressure change. The effect of junction size and outfall pipe entrance conditions are included in the chart, Negative pressure changes occur with an upstream pipe smaller than the outfall pipe. That is, at the junction center the upstream pressure line is below the outfall pressure line for this case. No flow other than that from the upstream in-line pipe may be involved where this Figure applies

To use the Figure:

1. Determine the outfall pipe pressure line elevation--Gen. Instr. 1.
2. Calculate the velocity head in the outfall--Gen. Instr. 2.
3. Calculate the size ratios D_U/D_O and A/D_U--Gen. Instr. 3.
4. Note whether the outfall pipe entrance is to be square-edged or rounded smooth. (note Gen. Instr. 6).
5. Enter Fig. 8-7 at the pipe size ratio D_U/D_O and read K_U at the curve for the proper value of A/D_U for a square-edged entrance condition, or at the dashed curve for a rounded entrance.
6. Calculate h_U (positive or negative)as indicated on the diagrams on the Figure and by Gen. Instr. 7.
7. Add a positive h_U to (or subtract a negative h_U from) the elevation of the outfall pressure line at the junction center to obtain the elevation of the upstream pipe pressure line at the same location.
8. The water surface elevation in the junction corresponds to that of the upstream pipe, whether above or below the outfall pressure line.
9. Check to be sure the water surface elevation in the junction is below the top of the junction box so that overflow may not occur.

Comments: For a square-edged entrance to the outfall pipe, values of A/D_U less than 1 do not appreciably reduce the values of K_U shown for $A/D_U = 1$. K_U increases for distances A/D_U greater than 3, but such values are not usual in storm drain construction. For rounded entrances the curve shown will apply with sufficient accuracy for all values of A/D_U up to 3.

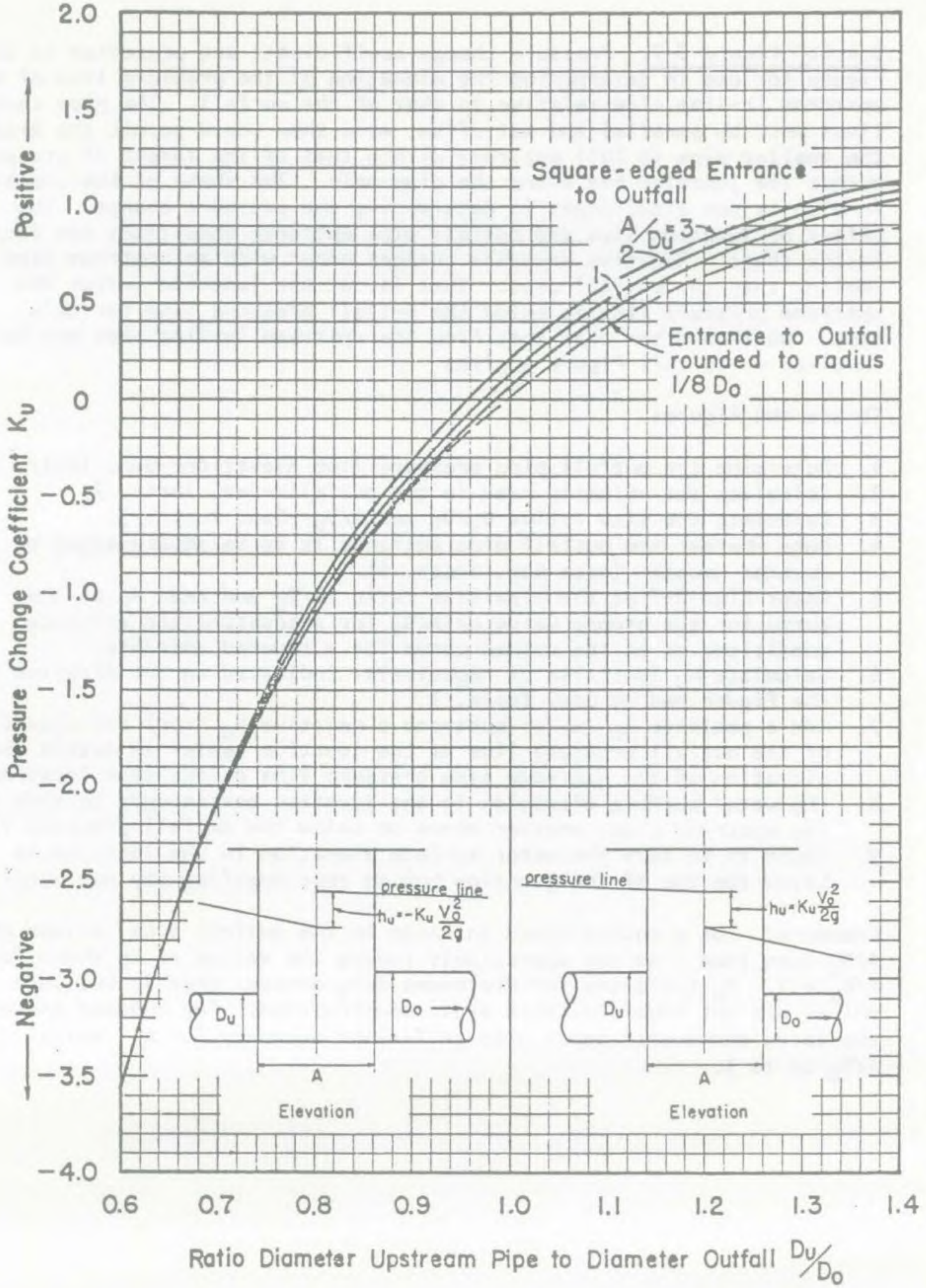
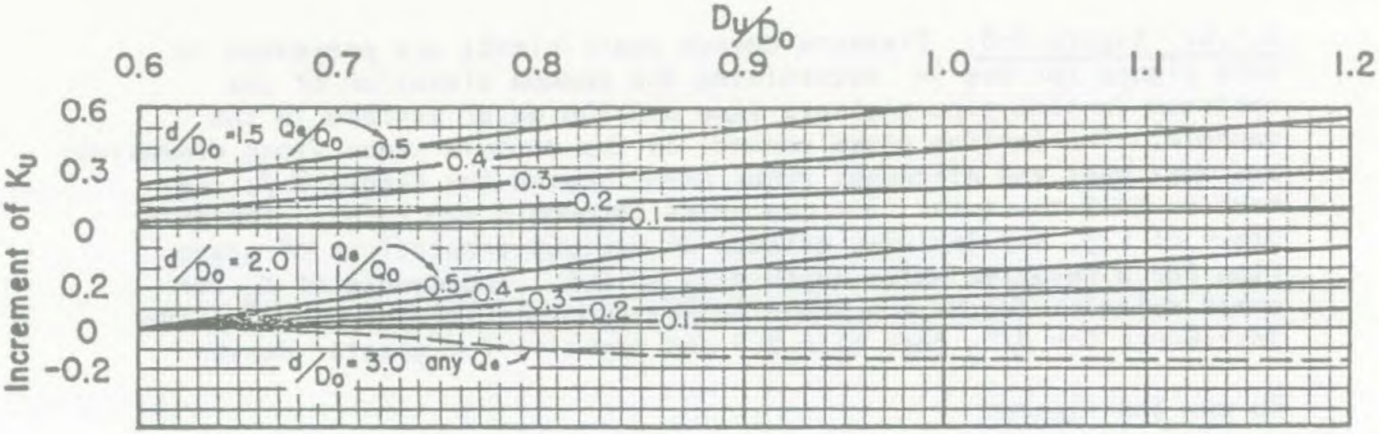


FIGURE 8–7. FLOW STRAIGHT THROUGH ANY MANHOLE(15)

8.4.6c Figure 8-8. Pressure change coefficients are presented in this Figure for use in determining the common elevation of the upstream in-line pipe pressure line and the water surface in the manhole. The in-line pipes connect at the manhole sides (long dimension) and must meet the alignment requirement stated for Figure 8-7. As much as half the total flow may enter through a top inlet. The main graph of Fig. 8-8 includes effects of various proportions of grate flow for a relative water depth d/D_O of 2.5. Increments of K_U for other relative depths are shown in the supplemental graphs; postiive increments for d/D_O less than 2.5 and negative for greater depths.

To use the Figure:

1. Determine the outfall pipe pressure line elevation--Gen. Instr. 1.
2. Calculate velocity head in the outfall--Gen. Instr. 2.
3. Calculate the ratios D_U/D_O and Q_U/Q_O--Gen. Instr. 3. (The inlet flow ratio $Q_G/Q_O = 1 - Q_U/Q_O$).
4. Estimate a value for the water depth d.
   a. Follow Gen. Instr. 4.
   b. Estimate $K = 3\ Q_G/Q_O$.
5. Calculate the corresponding relative water depth d/D_O.
6. If the estimated d/D_O is approximately 2.5 , enter the lower graph on Chart 4 at the pipe size ratio D_U/D_O and read K_U at the curve or interpolated curve for Q_U/Q_O; then proceed as in step (9).
7. If the estimated d/D_O is other than 2.5, follow step (6), then enter the upper graph on Fig. 8-8 at the given D_U/D_O and determine the increment of K_U required to account for the effects of the estimated relative water depth d/D_O.
8. Add K_U from step (6) and the increment from step (7) to determine the total value of K_U. Note that negative values of K_U may occur.
9. For a rounded outfall pipe entrance or one consisting of a pipe socket, reduce K_U according to Gen. Instr. 6.
10. Calculate h_U as indicated on the diagram on the Figure and by Gen. Instr. 7.
11. Add h_U to the elevation of the outfall pressure line at the inlet center to obtain the elevation of the upstream in-line pressure line at the same location. The water surface elevation will correspond.
12. From this water surface elevation subtract the elevation of the inlet bottom to obtain a more precise value for the water depth d.
13. Repeat the above procedure with the improved value of d from step (12), if necessary. Such repetition may not be necessary if the original estimated d/D_O of step (5) was reasonably accurate.
14. Check to be sure the water elevation is below the gutter elevation at the inlet so that inflow may be admitted.



Supplementary Chart for Modification of K_u
for Depth in Inlet other than $2.5\,D_o$

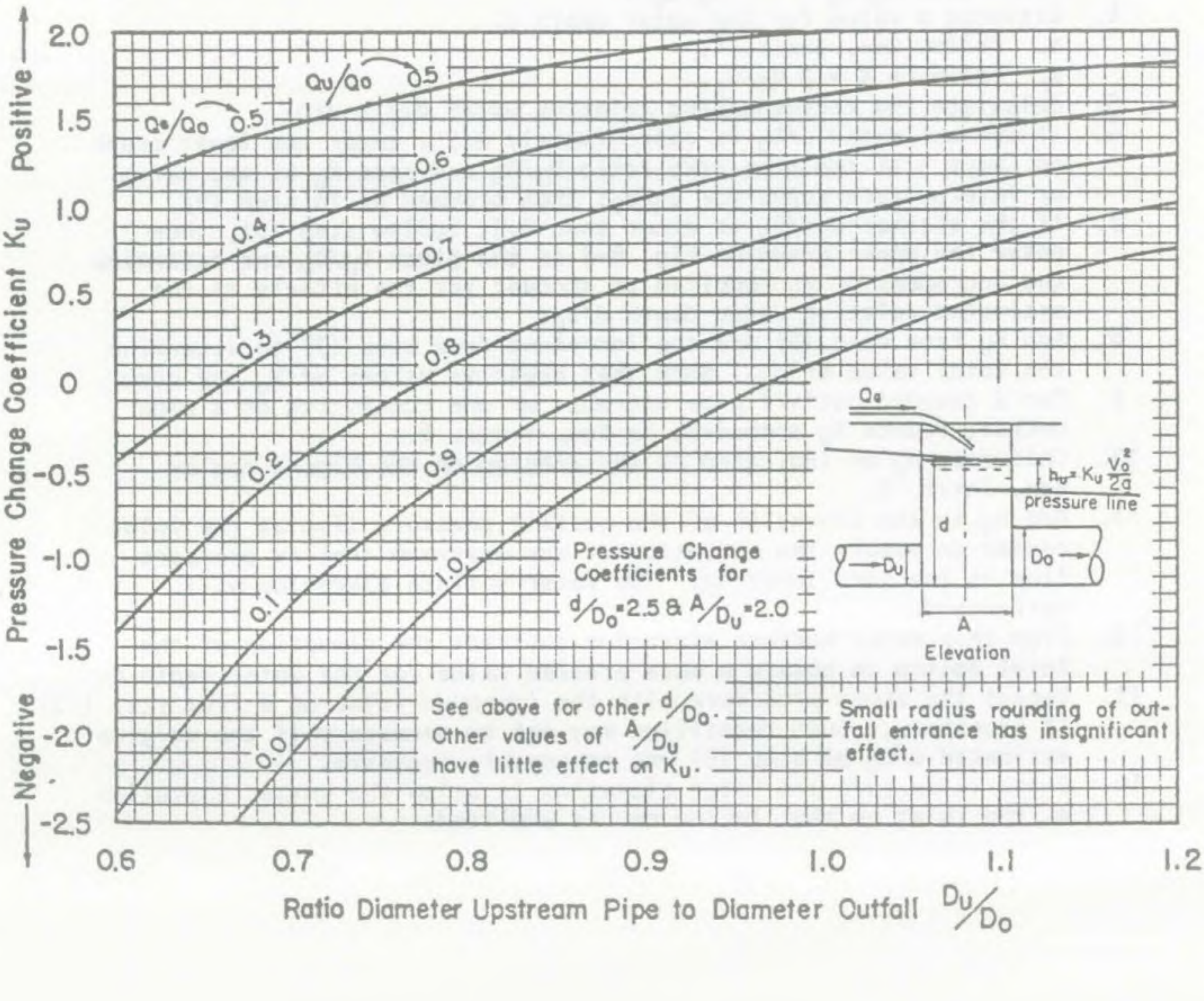


FIGURE 8-8. RECTANGULAR MANHOLE WITH THROUGH PIPELINE AND INLET FLOW (15)

8.4.6d Figure 8-9

Pressure change coefficients are presented in this Chart for use in determining the common elevation of the two upstream pipe pressure lines and the water surface in the manhole. Flow into the combination inlet and junction box is supplied by an upstream main, in-line with the outfall and flowing through the short dimension of the manhole, and a 90° lateral pipe connected at one end of the box, supplemented by flow through a top inlet. The main graph of Figure 8-9 applies directly for no flow into the manhole through the inlet. Increments of K_U and K_L for inlet flow conditions are shown in the supplementary graphs of the upper portion of the chart.

To use the Figure:

1. Determine the outfall pipe pressure line elevation--Gen. Instr. 1.
2. Calculate the velocity head in the outfall--Gen. Instr. 2.
3. Calculate the ratios D_U/D_O, Q_U/Q_O, and Q_G/Q_O--Gen. Instr. 3.
4. If no inlet flow is involved, enter the lower graph on Fig. 8-9 at the pipe size ratio D_U/D_O and read K_U (or K_L) at the curve or interpolated curve for Q_U/Q_O; then proceed as in step (10).
5. With inlet flow, estimate a value for the water depth d.
   a. Follow Gen. Instr. 4.
   b. Estimate $K = 1.5$.
6. Calculate the corresponding relative water depth d/D_O.
7. Enter the lower graph and obtain K_U (or K_L) as in step (4), this value applying for $Q_G/Q_O = 0$.
8. Enter the appropriate upper graph on Fig. 8-9, for the particular d/D_O nearest that estimated in step (6), at the given D_U/D_O and determine the increment of K_U (or K_L) at the curve for Q_G/Q_O. This increment accounts for the effects of inlet flow and is always a positive value, even when K_U of step (7) is negative.
9. Add K_U from step (7) and the increment from step (8) to obtain the total value of K_U. Note that in unusual cases the total value of K_U may be negative.
10. For a rounded outfall pipe entrance or one consisting of a pipe socket, reduce K_U and K_L according to Gen. Instr. 6.
11. Calculate h_U (also equal to h_L) as indicated by the diagram on the chart and by Gen. Instr. 7.
12. Add h_U to the elevation of the outfall pressure line at the branch point to obtain the elevation of the upstream in-line pipe pressure line at this point. The elevations of the lateral pipe pressure line and the water surface in the inlet will correspond.
13. From this water surface elevation subtract the elevation of the inlet bottom to obtain a more precise value for the water depth d.

8.4.6d Continued

14. Repeat the above procedure with the improved value of d from step (13), if necessary. Such repetition may not be necessary if the original estimated d/D_0 of step (6) was reasonably accurate.

15. Check to be sure the inlet water surface elevation is below the top of the inlet so that inflow may be admitted.

1-15-69

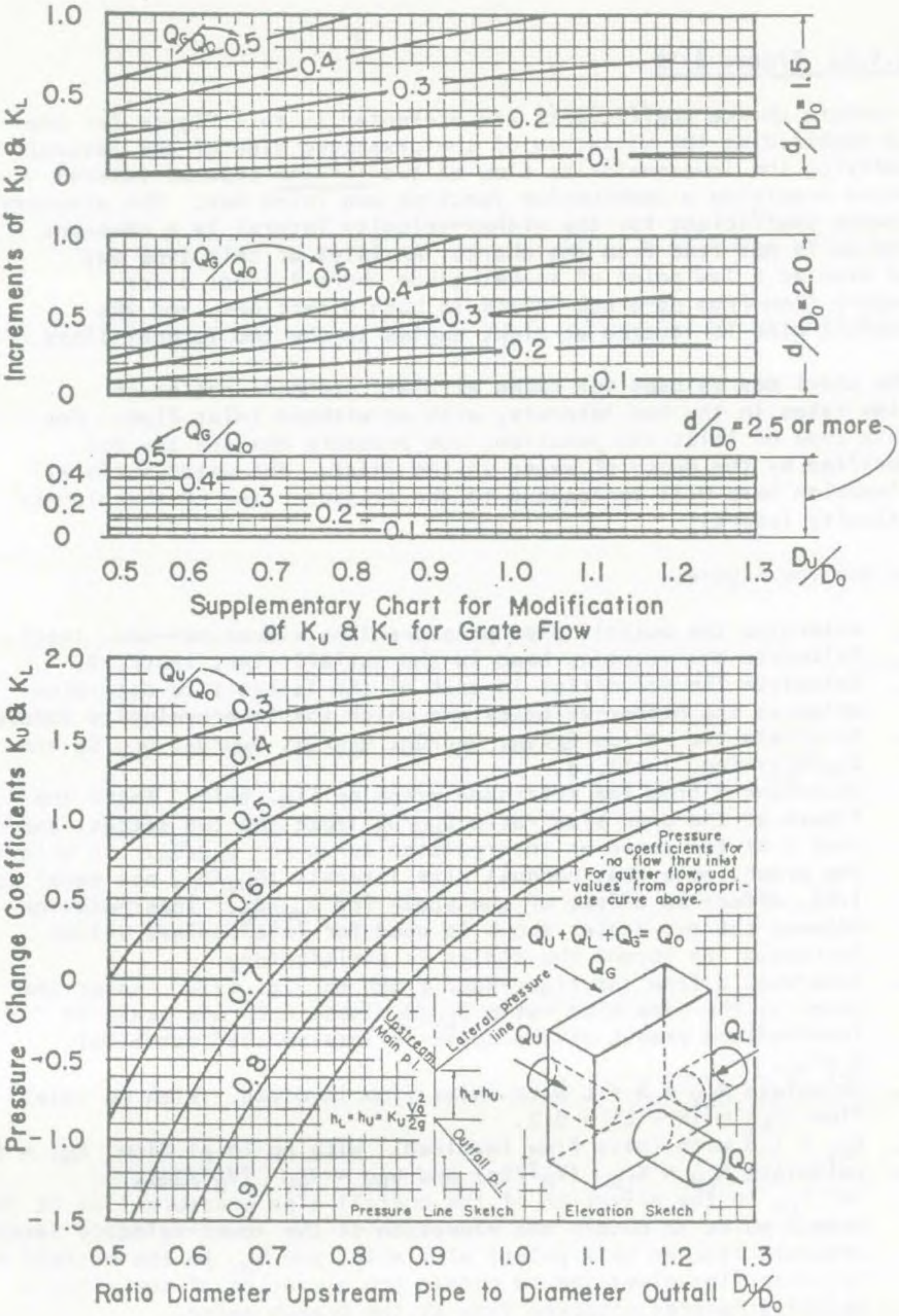


FIGURE 8-9. RECTANGULAR MANHOLE WITH IN-LINE UPSTREAM MAIN & 90° LATERAL PIPE (WITH OR WITHOUT INLET FLOW) (15)

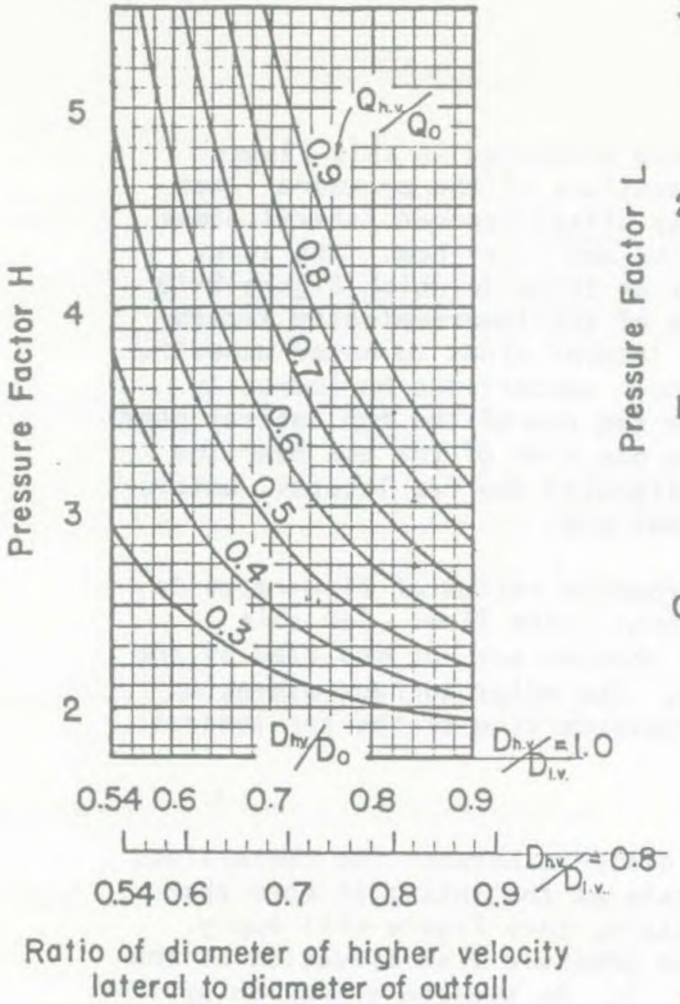
8.4.6e Figure 8-10

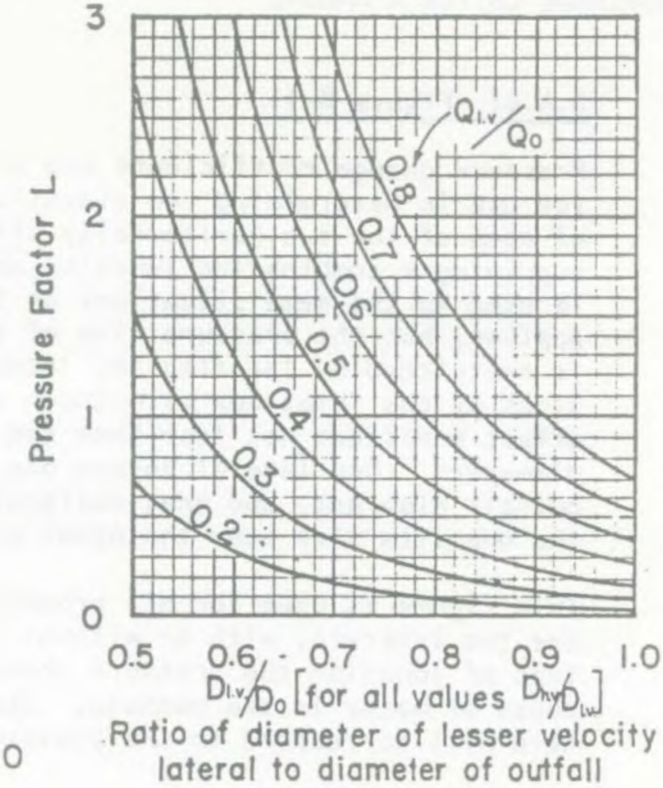
Pressure change coefficients are presented in this Figure for use in determining the elevation of the prsessure line of the lateral carrying the lower-velocity flow of two in-line opposed lateral pipes supplying a combination junction and inlet box. The pressure change coefficient for the higher-velocity lateral is a constant and so is not read from the chart. An inlet of this type may be used at a low point of street grade where lateral pipes supply flow from up-grade inlets in both directions, and the outfall pipe is located at right angles to the two lateral lines.

The chart may be used for cases with all probable ratios of flow rates in the two laterals, with or without inlet flow. For this type of inlet and junction, the pressure changes are not modified by the depth of water in the inlet. The water surface elevation here will correspond to the pressure line of the higher-velocity lateral.

To use the Figure:

1. Determine the outfall pipe pressure line elevation--Gen. Instr. 1.
2. Calculate the velocity head in the outfall--Gen. Instr. 2.
3. Calculate the velocities in each of the laterals to determine which is the higher-velocity and which the lower-velocity lateral.
4. Calculate the ratios Q_G/Q_0, Q_{HV}/Q_0, Q_{LV}/Q_0, D_{HV}/D_0, D_{LV}/D_0 and D_{HV}/D_{LV}--Gen. Instr. 3.
5. Determine H from the left-hand graph on Fig. 8-10. Enter the figure at the pipe size ratio D_{HV}/D_0 (note the two scales) and read H at the curve or interpolated curve for Q_{HV}/Q_0. In entering the graph, note that unequal size laterals (D_{HV}/D_{LV} not equal to 1.0), effect an offset of the scale for D_{HV}/D_0. Interpolation between the two scales shown is used for intermediate values. Extrapolation beyond the scales is satisfactory.
6. Determine L from the right-hand graph on Fig. 8-10. Enter the graph at the pipe size ratio D_{LV}/D_0 (note only one scale is involved)and read L at the curve or interpolated curve for Q_{LV}/Q_0.
7. Calculate $K_{LV} = H - L$ with inlet flow involved. With no inlet flow, $K_{LV} = (H - L) - 0.2$.
8. $K_{HV} = 1.8$ with inlet flow involved. With no inlet flow, $K_{HV} = 1.6$.
9. Calculate $h_{LV} = K_{LV} \ (V_0^2/2g)$ and $h_{HV} = K_{HV} \ (V_0^2/2g)$.
10. Add h_{LV} to the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the lower-velocity lateral pressure line at this point; similarly, add h_{HV} to the outfall pipe pressure line elevation to obtain the elevation of the higher-velocity lateral pressure line at the branch point.
11. Determine the water surface elevation in the inlet, which is equal to the lower of the two lateral pressure line elevations (that of the higher-velocity lateral).
12. Check to be sure the inlet water surface elevation is below the top of the inlet so that inflow may be admitted.





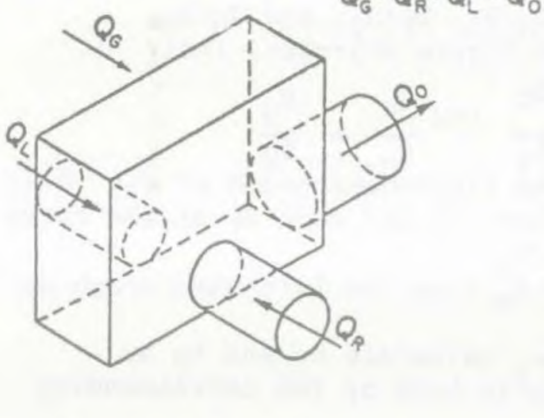
$D_{h.v.}$ = diameter of lateral with higher-velocity flow.

$Q_{h.v.}$ = rate of flow in lateral with higher-velocity flow.

$D_{l.v.}$ = diameter of lateral with lower-velocity flow.

$Q_{l.v.}$ = rate of flow in lateral with lower-velocity flow.

$Q_G + Q_R + Q_L = Q_O$



Elevation Sketch

To find K_R or K_L for the right or left lateral pipe with flow at a lesser velocity than the other lateral, read H for the higher velocity lateral D and Q, then read L for the lower velocity lateral D and Q; then: K_R (or K_L) = H − L

K_R or K_L for the lateral pipe with higher velocity flow is always 1.8

$$h_L = K_L \frac{V_O^2}{2g} \qquad h_R = K_R \frac{V_O^2}{2g}$$

FIGURE 8-10. RECTANGULAR MANHOLE WITH IN-LINE OPPOSED LATERAL PIPES EACH AT 90° TO OUTFALL (WITH OR WITHOUT INLET FLOW) (15)

8.4.6f Figure 8-11

Pressure change coefficients are presented in this Figure for use in determining the elevations of the pressure lines of each of the two horizontally offset opposed lateral pipes supplying a combination junction and inlet box. The inlet is used in the same situations as those to which Figure 8-10 applies, but the pressure rise of the lower-velocity lateral is restricted by locating the lateral pipes to enter opposite sides of the inlet box with their centerlines horizontally offset a distant not less than the sum of the two lateral pipe diameters. One lateral enters one side of the box near the outfall pipe end, and one, designated the far lateral, enters the opposite side near the other end.

This figure is used for all probable ratios of flow rates in the two laterals, with or without inlet flow. For this type of junction the pressure changes are not modified by the depth of water in the manhole. The water surface elevation here will correspond to the pressure line of the far lateral.

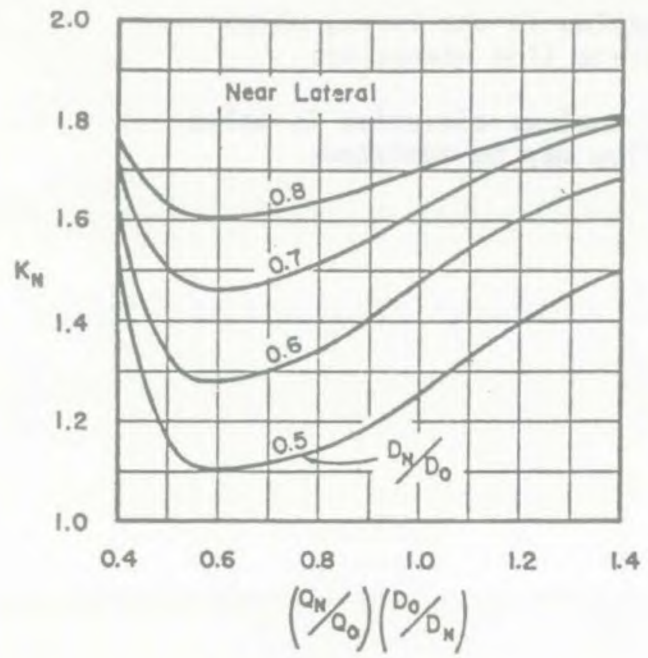
To use the Figure:

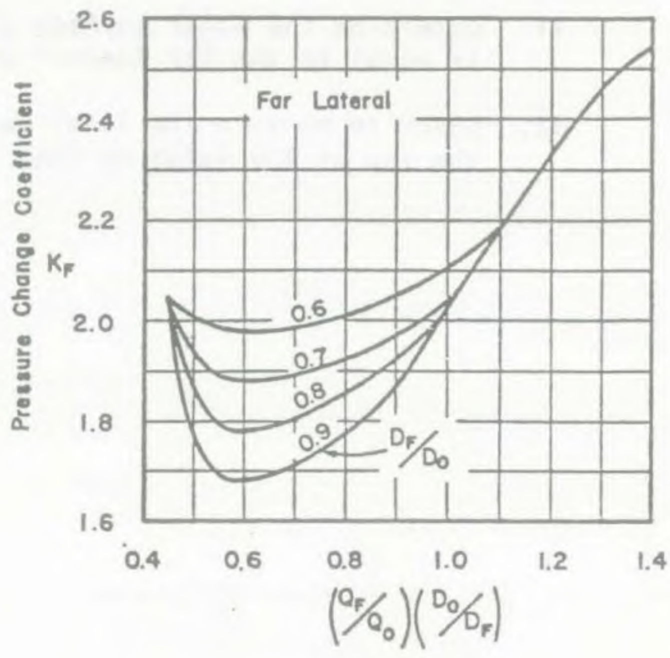
1. Determine the horizontal distance between the centerlines of the opposed flow laterals at the inlet; if more than the sum of the pipe diameters, this figure will apply.
2. Determine the outfall pipe pressure line elevation at the branch points--Gen. Instr. 1. An average elevation applicable to both is sufficiently precise.
3. Calculate the velocity head in the outfall--Gen. Instr. 2.
4. Calculate the ratios Q_F/Q_O, Q_N/Q_O, D_F/D_O, and D_N/D_O, observing the nomenclature of Figure 8-3--Gen. Instr. 3.
5. Calculate the factors $\frac{Q_F}{Q_O} \times \frac{D_O}{D_F}$ and $\frac{Q_N}{Q_O} \times \frac{D_O}{D_N}$.
6. For the far lateral, enter the right-hand graph of Fig. 8-11 at the abscissa value from step (5) and read K_F at the curve or interpolated curve for D_F/D_O.
7. For the near lateral, obtain K_N from the left-hand graph by a similar procedure.
8. For a manhole with inlet flow, calculate h_F and h_N by multiplying the outfall velocity head by the corresponding coefficient K_F or K_N.
9. For a junction without inlet flow, calculate h_F and h_N by multiplying the outfall velocity head by the corresponding reduced coefficients $(K_F - 0.2)$ or $(K_N - 0.2)$.
10. Add h_F and h_N to the elevation of the downstream (outfall pipe) pressure line to obtain the elevations of the pressure lines of the two laterals at their branch points.

8.4.6f Continued

11. Determine the water surface elevation in the inlet, which is equal to the far lateral pressure line elevation.

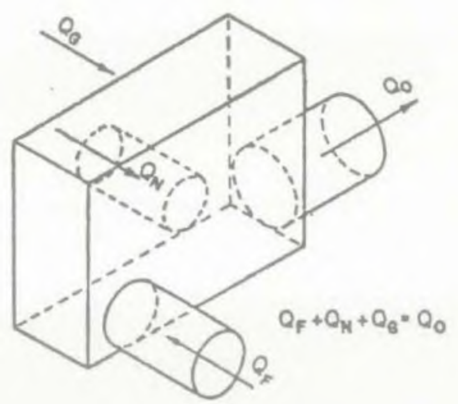
12. Check to be sure the inlet water surface elevation is below the top of the inlet so that inflow may be admitted.





$$h_N = K_N \frac{V_O^2}{2g}$$

$$h_F = K_F \frac{V_O^2}{2g}$$



Elevation Sketch

FIGURE 8-11. RECTANGULAR MANHOLE WITH OFFSET OPPOSED LATERAL PIPES EACH AT 90° TO OUTFALL (WITH OR WITHOUT INLET FLOW) (15)

8.4.6g Figure 8-12

Pressure change coefficients are presented in this Figure for use in determining the elevation of the pressure line of an upstream pipe connected by means of a square manhole to an outfall pipe at a 90° angle. The manhole conditions covered by this Figure do not involve an upstream pipe in-line with the outfall pipe. For this and other manhole figures, the lateral pipe is designated by the subscript L irrespective of its right-hand or left-hand position. The coefficients given by the figure apply directly to manholes having a square-edged entrance to the outfall pipe. Coefficients for a rounded entrance are obtained by reduction of the chart values as stated below. The design of manholes with deflector devices is discussed separately.

To use the figure:

1. Determine the outfall pipe pressure line elevation--Gen. Instr. 1.
2. Calculate the velocity head in the outfall--Gen. Instr. 2.
3. Calculate the ratios D_L/D_O and B/D_O--Gen. Instr. 3.
4. Enter the lower graph of Fig. 8-12 at the pipe size ratio D_L/D_O and read $\overline{K}_L$ at the curve or interpolated curve for the manhole size ratio B/D_O. For all flow from a lateral $K_L = \overline{K}_L$.
5. For a rounded outfall pipe entrance or one formed by a pipe socket reduce the figure value of $\overline{K}_L$ by 0.3 as defined by Gen. Instr. 6.
6. Calculate the change of pressure $h_L = K_L \times \frac{V_O^2}{2g}$ (always positive for 90° deflections.)
7. Add h_L to the elevation of the outfall pressure line at the branch point to obtain the elevation of the lateral pipe pressure line at this point.
8. The water surface elevation in the manhole will be above the lateral pipe pressure line. To determine the water-surface elevation use Fig. 8-13, as instructed in steps (12) through (18) of the instructions for a square manhole at the junction of a 90° lateral with a through main.
9. Check to be sure the water surface elevation is above the pipe crowns to justify using these figures and that it is sufficiently below the top of the manhole to indicate safety from overflow.

8.4.6h Figure 8-12

Pressure change coefficients may also be obtained from this Figure for use in determining the elevation of the pressure line of an upstream pipe connected by means of a round manhole to an outfall pipe at a 90° angle.

To use the figure:

1. Proceed as instructed in steps (1) through (4) for a square manhole at a 90° deflection to obtain a base value of $\overline{K}_L$ for

8.4.6h Continued

the particular values of D_L/D_0 and B/D_0.

2. To provide for the effects of the round manhole cross section, reduce $\overline{K}_L$ in accordance with the following table:

B/D_0	$\frac{D_L}{D_0}$ = 0.8	1.0	1.2
	Reductions for $\overline{K}_L$		
1.75	0.3	0.2	0.0
1.33	0.2	0.1	0.0
1.10	0.1	0.0	0.0

The reduced values apply for a sharp-edged entrance to the outfall pipe.

3. With a well-rounded entrance to the outfall pipe from a round manhole, reduce $\overline{K}_L$ of step (1) by 0.3 with no further reduction for manhole cross section shape.
4. Follow steps (6) through (9) as detailed for square manholes at a 90° deflection.

8.4.6i Figure 8-12

Pressure change coefficients are presented in this Figure for use in determining the elevation of the pressure line of an upstream pipe connected to an outfall pipe at a 90° angle by means of a square or round manhole modified by flow deflectors. Deflectors in a manhole effectively eliminate the effects related to the shape of the manhole. The basic types of deflector walls which may be constructed in square or round manholes to effect a reduction of the pressure loss are detailed and described in Chapter 8 of this Part of the Manual.

The deflectors which are most easily constructed and are as effective as more complex types provide a vertical wall to guide the flow toward the outfall pipe. The wall need not be higher than the outfall pipe diameter and must fill in that part of the manhole opposite the lateral pipe exit so that it is flush with the side of the outfall pipe. Three basic types of such deflector walls are possible and are included in the curves of Figure 8-12. These three are (1) walls parallel to the outfall pipe centerline or 0° walls, (2) inclined walls, limited to an angle of about 15° to the outfall centerline if an upstream in-line pipe is to be used, and (3) walls at 45° to both the lateral and outfall pipes, or walls curved on a radius of about the manhole dimension extending from lateral to outfall, and therefore to be used only when no upstream in-line pipe is involved. Rounding of the corner formed between the deflector wall and the manhole floor is not required, and may be detrimental in some cases.

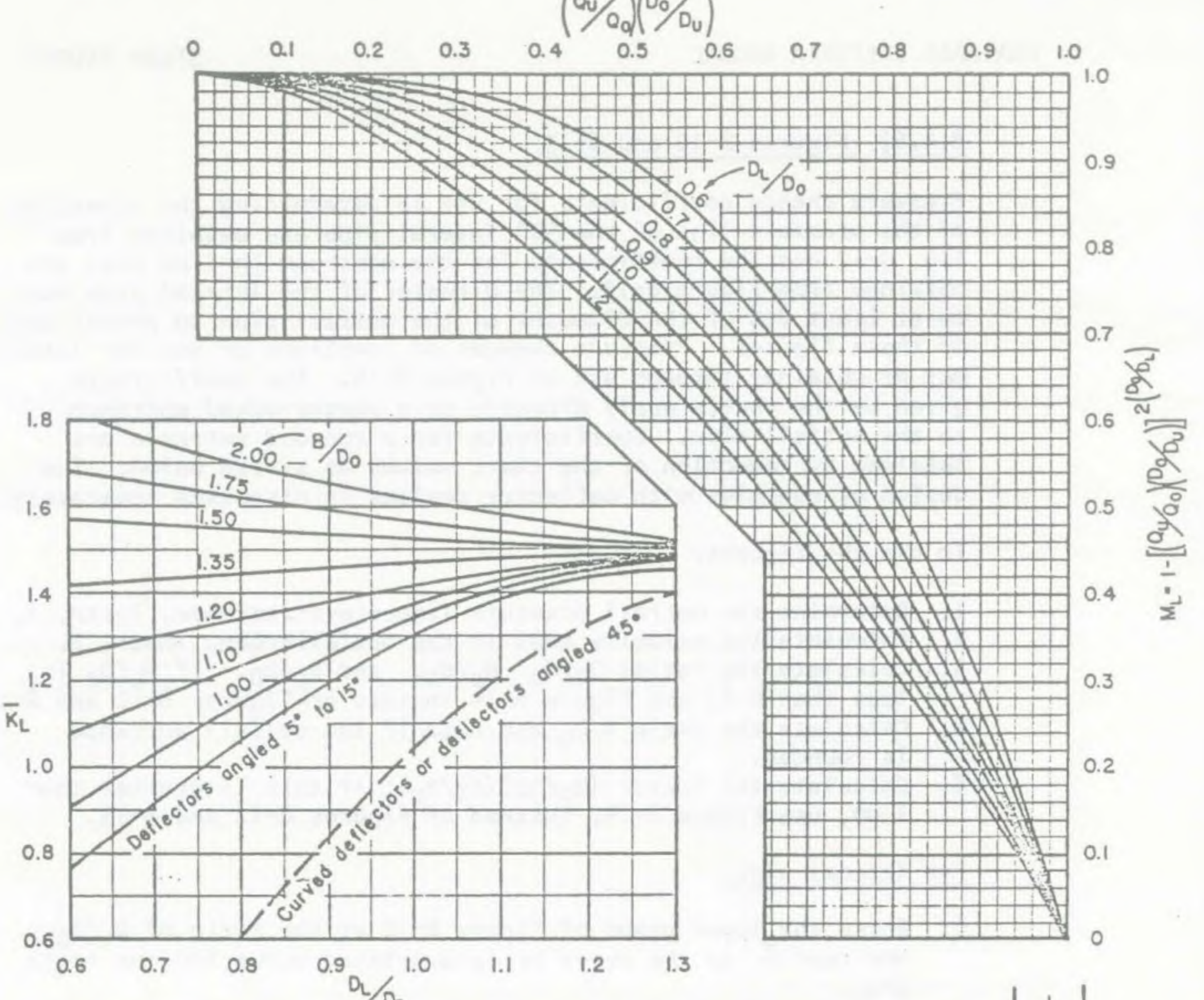
To use the Figure:

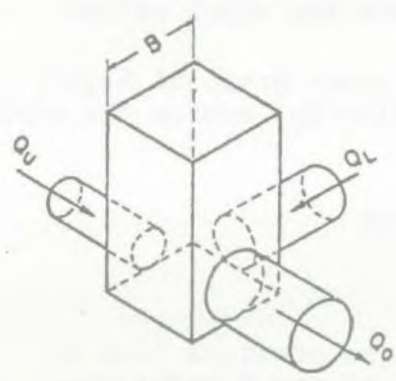
1. Determine the outfall pipe pressure line elevation--Gen. Instr. 1.
2. Calculate the velocity head in the outfall--Gen. Instr. 2.
3. Classify the type of deflector used:
   a. Parallel wall--0°
   b. Inclined wall--5° to 15°
   c. 45° or curved wall.
4. Calculate the ratios D_L/D_O and B/D_O. No distinction between square and round manholes is necessary.
5. If B/D_O is 1.5 or less, enter the lower graph of the chart at the ratio D_L/D_O and read $\overline{K}_L$ at the curve for the appropriate deflector type. In the case of a parallel wall, use the curve for $B/D_O = 1.00$.
6. If B/D_O is more than 1.5 and less than 2.0, use the same dashed curve for 45° or curved deflectors, use the curve for $B/D_O = 1.10$ for 5° to 15° angle deflectors, and use the curve for $B/D_O = 1.20$ for 0° angle deflectors.

8.4.6i Continued

7. A rounded entrance to the outfall pipe or one formed by a pipe socket is less effective in reducing the pressure change with deflectors than when deflectors are not used. A reduction of $\overline{K_L}$ by 0.1 may be justified.
8. Calculate the change of pressure

   $$h_L = K_L \times \frac{V_0^{\,2}}{2g} \quad (\text{for } Q_L = Q_0,\ K_L = \overline{K_L}).$$
9. Add h_L to the elevation of the outfall pressure line at the branch point to obtain the elevation of the lateral pipe pressure line at this point.
10. The water-surface elevation in the manhole will be above the lateral pipe pressure line. To determine the water surface elevation, use Figure 8-13 as instructed in steps (2) through (8) for deflectors in a manhole at the junction of a 90° lateral with a through main.
11. Check to be sure the water surface elevation is above the pipe crowns to justify using these charts and that it is sufficiently below the top of the manhole to indicate safety from overflow.





Elevation Sketch

To find K_L for the lateral pipe, first read $\overline{K}_L$ from the lower graph. Next determine M_L. Then

$$K_L = \overline{K}_L \times M_L$$

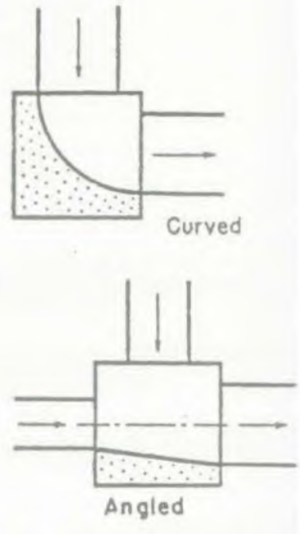
Dashed curve for curved or 45° angle deflectors applies only to manholes without upstream in-line pipe.

Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of $\overline{K}_L$ by 0.2 for combining flow.

For $Q_U/Q_O \times D_O/D_U > 1$ use Figure 8-14

For $D_L/D_O < 0.6$ use Figure 8-14



Plan of Deflectors

$$h_L = K_L \frac{V_O^2}{2g}$$

FIGURE 8-12. MANHOLE AT 90° DEFLECTION OR ON THROUGH PIPELINE AT JUNCTION OF 90° LATERAL PIPE (LATERAL COEFFICIENT). (15)

8.4.6j Figures 8-12 and 8-13

Pressure change coefficients for use in determining the elevation of the pressure line of the 90° lateral pipe are obtained from Fig. 8-12 and the coefficients for the upstream in-line pipe are obtained from Figure 8-13. The diameter of the lateral pipe must be at least 0.6 of the diameter of the outfall pipe to permit use of these figures. Pressure changes at junctions of smaller laterals may be obtained through use of Figure 8-14. The coefficients given by the charts apply directly to a square-edged entrance to the outfall pipe. Coefficients for a rounded entrance are obtained by reduction of the chart values as stated below. The design of manholes with deflector devices is discussed separately.

To use the figures:

1. Determine the outfall pressure line elevation--Gen. Instr. 1.
2. Calculate the velocity head in the outfall--Gen. Instr. 2.
3. Calculate the ratios Q_U/Q_O, D_U/D_O, and D_L/D_O. If D_L/D_O is less than 0.6, use Figure 8-14 instead of Figures 8-12 and 8-13
4. Calculate the ratio B/D_O and note if the outfall entrance is rounded.
5. Calculate the factor $(Q_U/Q_O)(D_O/D_U)$; if this is greater than 1.00, use Figure 8-14, instead of Figures 8-12 and 8-13.

For lateral pipe:

6. Enter the lower graph of Figure 8-12 at the ratio of D_L/D_O and read $\overline{K}_L$ at the curve or interpolated curve for the ratio B/D_O.
7. For a rounded outfall pipe entrance or one formed by a pipe socket as defined by Gen. Instr. 6, reduce the chart values of $\overline{K}_L$ by 0.2.
8. Determine the factor M_L by entering the upper graph of Figure 8-12 at the value of the factor $(Q_U/Q_O) \times (D_O/D_U)$ and at the curve or interpolated curve for D_L/D_O.
9. Calculate $K_L = M_L \times \overline{K}_L$.
10. Calculate the lateral pipe pressure change

$$h_L = K_L \times \frac{V_O^2}{2g}$$

11. Add h_L to the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the lateral pipe pressure line at this point.

For upstream in-line pipe:

12. Enter the lower graph of Figure 8-13 at the ratio of D_L/D_O and read $\overline{K}_U$ at the curve or interpolated curve for B/D_O.

8.4.6j Continued

13. For a rounded entrance to the outfall pipe or one formed by a pipe socket, reduce $\overline{K_U}$ by 0.2.
14. Determine the factor M_U from the upper graph of Figure 8-13.
15. Calculate $K_U = M_U \times \overline{K_U}$.
16. Calculate the upstream in-line pipe pressure change

$$h_U = K_U \times \frac{V_O^2}{2g}$$

17. Add h_U to the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the upstream in-line pipe pressure line at this point.

For water surface:

18. The water-surface elevation in the manhole will correspond to the upstream in-line pipe pressure line at the branch point.
19. Check to be sure that the water surface elevation is above the pipe crowns to justify using these charts and that it is sufficiently below the top of the manhole to indicate safety from overflow.

8.4.6k Figures 8-12 and 8-13

Pressure change coefficients may also be obtained from Figures 8-12 and 8-13 for use in determining the elevations of the pressure lines of the 90° lateral pipe and the upstream in-line pipe connected by a round manhole to an outfall pipe.

To use the Figure:

1. Proceed as instructed by steps (1) through (6) for a square manhole at a similar junction to obtain a base value of $\overline{K}_L$.

For lateral pipe:

2. To provide for the effects of the round manhole cross-section, reduce $\overline{K}_L$ in accoordance with the following table:

B/D_O	$\frac{D_L}{D_O}$ = 0.6	0.8	1.0	1.2
	Reductions for $\overline{K}_L$			
1.75	0.4	0.3	0.2	0.0
1.33	0.3	0.2	0.1	0.0
1.10	0.2	0.1	0.0	0.0

The reduced values apply for a sharp-edged entrance to the outfall pipe.

3. With a well-rounded entrance to the outfall pipe from a round manhole, reduce $\overline{K}_L$ obtained in step (2) by 0.1.
4. Determine the factor M_L from the upper graph of Figure 8-12 and proceed as instructed in steps (8) through (11) for a square manhole to complete the determination of the elevation of the lateral pipe pressure line.

For upstream in-line pipe:

5. Proceed as instructed in steps (12) through (17) for a square manhole at a similar junction to obtain the elevation of the upstream in-line pipe pressure line. Note that no reduction of $\overline{K}_U$ is to be made for effects of the round manhole cross-section.

For water surface:

6. Proceed as instructed by steps (18) and (19) for a square manhole at a similar junction.

1-15-69

8.4.61 Figures 8-12 and 8-13

Pressure change coefficients are also presented in Figures 8-12 and 8-13 for use in determining the elevations of the pressure lines of the lateral and in-line pipes at a junction of this type, with either a square or a round manhole modified by flow deflectors. Deflectors in a manhole effectively eliminate the effects related to the shape of the manhole. Deflector types are described in the instructions for use of Figure 8-12 for a manhole with deflectors at a 90° deflection of a storm drain. The curved and 45° deflectors cannot be used in a manhole on a through pipeline because of the space required for through in-line flow.

To use the Figure:

1. Proceed as instructed in steps (1) through (9) for deflectors in a manhole at a 90° deflection, disregarding references to 45° or curved walls. Through use of Figure 8-12 these steps will give the elevation of the lateral pipe pressure line at the branch point. As noted in the instructions for a manhole of this type without deflectors, Figure 8-14 must be used when $D_L/D_O < 0.6$ or

$$\frac{Q_U}{Q_O} \times \frac{D_O}{D_U} > 1.00$$

For upstream in-line pipe:

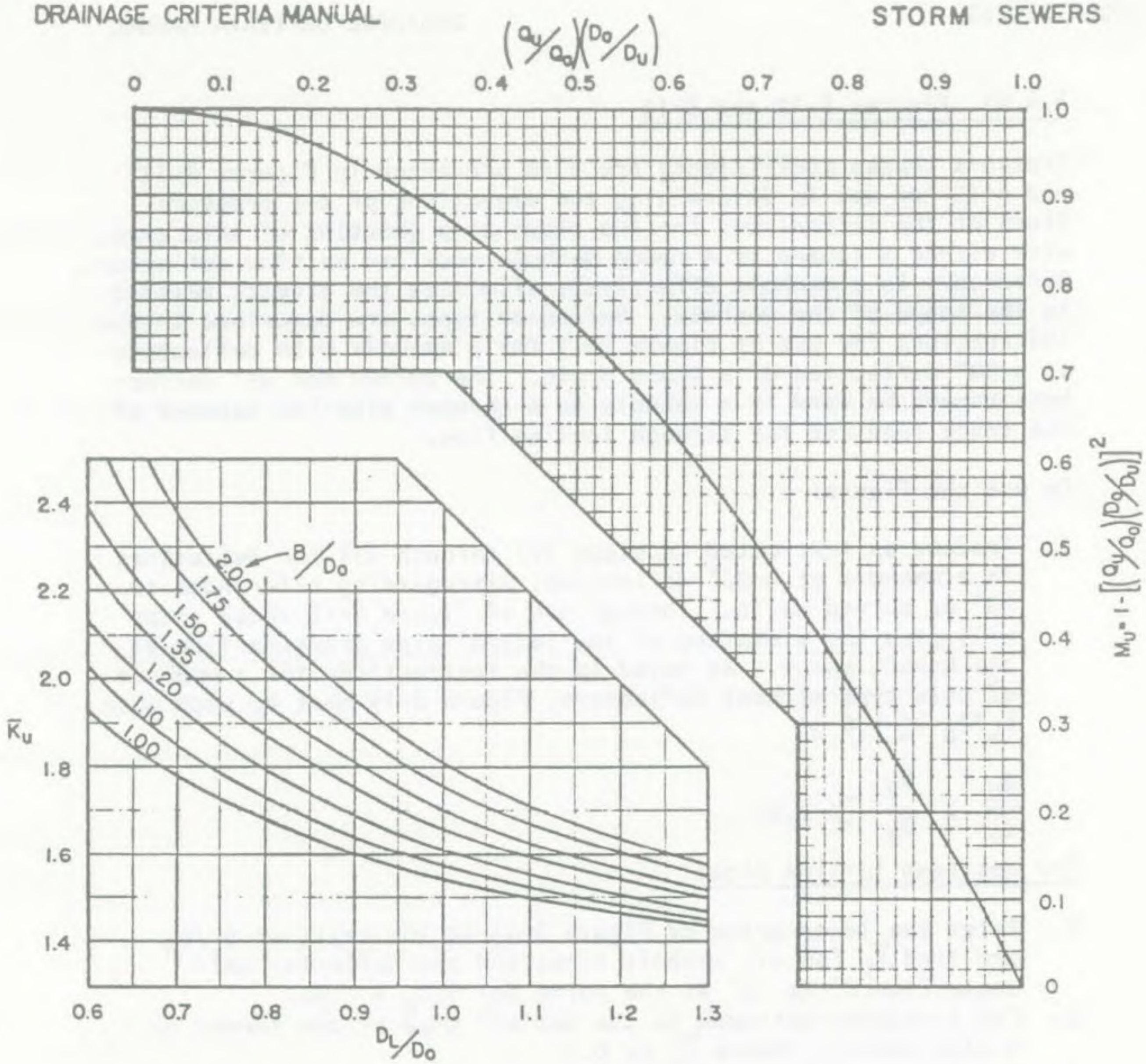
2. Enter the lower graph of Figure 8-13 at the ratio of D_L/D_O and read $\overline{K_U}$ for all manhole sizes and any deflector wall angle from 0° to 15° at the curve for $B/D_O = 1.00$.
3. For a rounded entrance to the outfall pipe or one formed by a pipe socket, reduce $\overline{K_U}$ by 0.1.
4. Determine the factor M_U from the upper graph of Fig. 8-13.
5. Calculate $K_U = M_U \times \overline{K_U}$.
6. Calculate the upstream in-line pipe pressure change

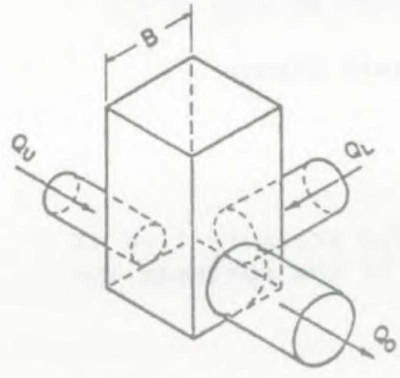
$$h_U = K_U \times \frac{V_O^2}{2g}$$

7. Add h_U to the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the upstream in-line pipe pressure line at this point.

For water surface:

8. The water-surface elevation in the manhole will correspond to the upstream in-line pipe pressure line at the branch point.
9. Check to be sure that the water-surface elevation is above the pipe crowns to justify using these charts and that it is sufficiently below the top of the manhole to indicate safety from overflow.





Elevation Sketch

To find K_U for the upstream main, first read $\overline{K}_U$ from the lower graph. Next determine M_U. Then

$$K_U = \overline{K}_U \times M_U$$

For manholes with deflectors at 0° to 15°, read $\overline{K}_U$ on curve for $B/D_O = 1.0$

Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of $\overline{K}_U$ by 0.2 for combining flow.

For deflectors refer to sketches on Figure 8-12

For $Q_U/Q_O \times D_O/D_U > 1$ use Figure 8-14

For $D_L/D_O < 0.6$ use Figure 8-14

$$h_U = K_U \frac{V_O^2}{2g}$$

FIGURE 8-13 MANHOLE ON THROUGH PIPELINE AT JUNCTION OF A 90° LATERAL PIPE (IN-LINE PIPE COEFFICIENT) (15)

8.4.6m Figure 8-14

Pressure change coefficients are presented in Figure 8-14 for use in determining the common elevation of the pressure lines of the lateral and in-line pipes at a junction of this type for cases of pipe sizes or flow divisions outside the range over which Figures 8-12 and 8-13 may be applied. Figure 8-12 and 8-13 are more reliable within their range and should be used if possible. Neither manhole shape nor size nor relative size of lateral pipe modify the coefficients of Fig. 8-14. The chart may also be used for direct connection of a 90° lateral to a main without use of a manhole. The coefficients of the chart apply directly to a square-edged entrance to the outfall pipe. Coefficients for a rounded entrance are obtained by reduction of the chart values as stated below. Deflectors in the manhole are not effective in the ranges covered by Fig. 8-14 and therefore need not be used.

To use the Figure:

1. Determine the outfall pipe pressure line elevation--Gen. Instr. 1.
2. Calculate the velocity head in the outfall--Gen. Instr. 2.
3. Calculate the ratios D_L/D_O, D_U/D_O, and Q_U/Q_O. Note that use of Figures 8-12 and 8-13 is advisable if the size and flow factors are within their range. Figure 8-14 should not be used for $Q_U/Q_O \leq 0.7$ if other solutions are possible.
4. Note whether the outfall entrance is to be rounded or formed by a pipe socket as defined by Gen. Instr. 6.
5. Enter Figure 8-14 at the ratio D_U/D_O and read K_U (also equal to K_L) at the curve or interpolate curve for Q_U/Q_O.
6. If $\frac{Q_U}{Q_O} \times \frac{D_O}{D_U}$ was found to be greater than 1.00 in an attempt to use Figures 8-12 and 8-13, K_U of step (5) will be negative in sign, thus providing a check on proper use of the figures.
7. For rounded entrance from the manhole to the outfall pipe use the reduced values from the Figure.
8. Calculate the change of pressure $h_U = h_L = K_U \times \frac{V_O^2}{2g}$, h_U and h_L are positive or negative depending on the sign of K_U as read from the figure.
9. Add a positive h_U to or subtract a negative h_U from the elevation of the outfall pipe pressure line at the branch point to obtain the elevation of the upstream in-line pipe pressure line at this point.
10. The elevation of the lateral pipe pressure line at the branch point and the water surface elevation in the manhole will correspond to the upstream in-line pipe pressure line elevation found in step (9)..
11. Check to be sure that the water-surface elevation is above the pipe crowns to justify using these charts and that it is sufficiently below the top of the manhole to indicate safety from overflow.

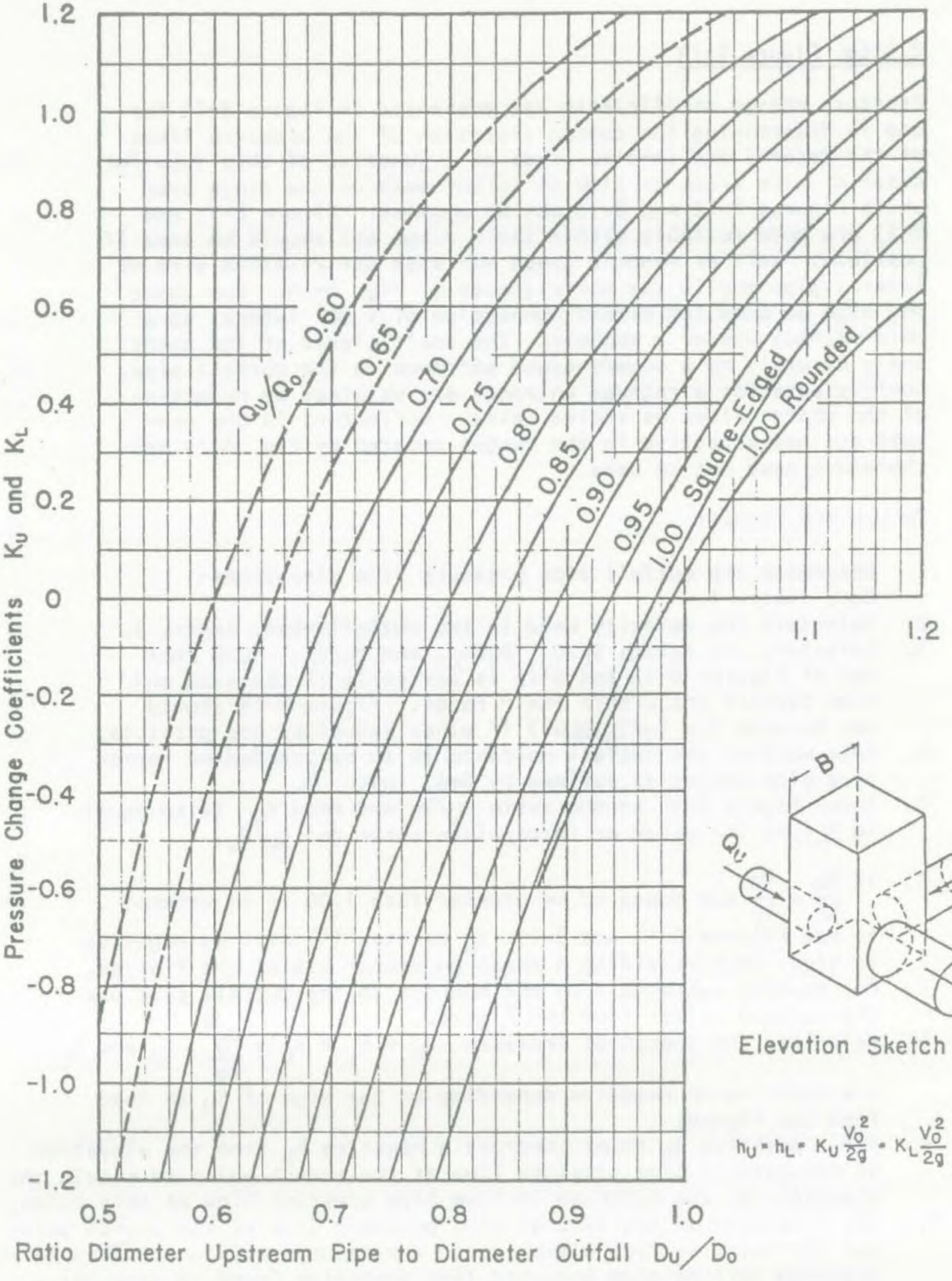


FIGURE 8-14. MANHOLE ON THROUGH PIPELINE AT JUNCTION OF A 90° LATERAL PIPE
(FOR CONDITIONS OUTSIDE RANGE OF FIGURES 8-12 & 8-13)(15)

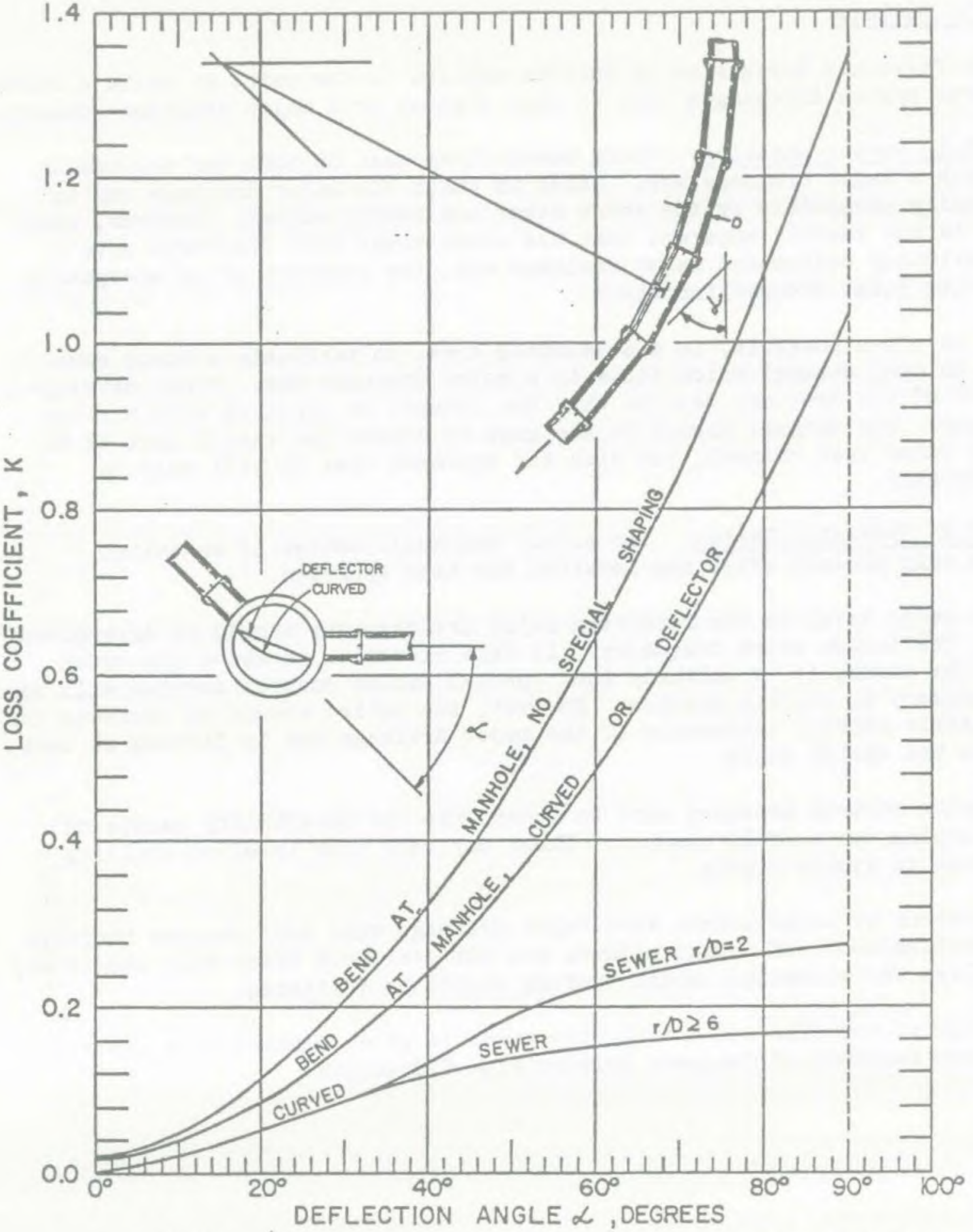


FIGURE 8-15. SEWER BEND LOSS COEFFICIENT (15,16,17)

8.5 Outlets

The following discussion of outlets applies to the point at which a storm sewer system discharges into an open channel or a major drainage conduit.

8.5.1 Outlet Location. Storm sewer flows must in some way eventually reach a major drainage way. Cases in which the major drainage way is readily accessible by the storm sewer are easily solved. However, when it is not readily apparent that the storm sewer will discharge into a previously delineated major drainage way, the question of an acceptable outlet point becomes important.

It is often possible, in a developing area, to terminate a storm sewer in an open channel which flows to a major drainage way. Final development of the area may require that the channel be replaced with a storm sewer. The channel should be designed to convey the runoff just as is any other open channel, but with the approach that it will only be temporary.

8.5.2 Hydraulic Design. The actual hydraulic design of an outlet can only proceed after the location has been approved.

The water level in the receiving major drainage way should be determined for the design storm frequency. If this elevation is above the crown of the sewer, it is unlikely that special outlet control devices will be necessary to prevent erosion. However, the outlet should be reviewed for possible erosion tendencies if the major drainage way is flowing at less than the design depth.

Erosion control measures must be taken when the possibility exists of affecting the outfall channel. These may vary from involved stilling basins to simple riprap.

Junctions of large sewers with major drainage ways must receive thorough investigation. If design methods are not available which will adequately analyze the situation, model testing should be initiated.

Design of endwalls and outlet structures is given considerable attention in the Handbook of Concrete Culvert Pipe Hydraulics.

APPENDIX B

GLOSSARY
OF
ENGINEERING TERMS AND WORDS

The definitions of engineering and related terms are set forth in this glossary to provide a uniform connotation of the terms used in the manual. Definitions of other specialized terms appear in the text where they are first used.

"Antecedent Moisture Condition (AMC)" means the degree of wetness of the soil indicated by the accumulated amount of precipitation occurring in the five days preceding the storm in question.

"Appurtenances to Sewers and Drainage Systems" means structures and devices, other than pipe or conduit, which are an integral part of the drainage system. Manholes, inlets, and storage facilities are appurtenances to the drainage system.

"Backwater" means the increased depth of water upstream from a dam or obstruction in a stream channel due to the existence of such obstruction.

"Backwater Curve" means the term applied to the longitudinal profile of the water surface in a stream or open channel when flow is steady but nonuniform.

"Berm" means a horizontal strip or shelf built into an embankment or cut to break the continuity of an otherwise long slope, usually to reduce erosion or to increase the thickness or width of cross section of an embankment.

"Bypass Channel" means a man-made channel formed to carry excess stormwater runoff through a specific area.

"Carry-Over" means the quantity of water which continues past an inlet.

"Channel" means a natural or artificial waterway which continuously or periodically contains moving water, or which forms a connecting link between two bodies of water. It has a definite bed and banks which confine the water.

"Channelization" means the straightening, dredging, or otherwise modifying of a stream and its overbank areas to permit the rapid passage of flood flows.

"Channel Storage" See "Storage."

"Combined Sewer" means a closed conduit carrying both sanitary and stormwater.

"Composite Hydrograph" means a graph showing the sum of discharge values with respect to time from separate areas or sub-areas for a given point under consideration.

"Conduit" means an artificial or natural duct for conveying liquids.

"Convenience System" See "Initial Drainage System."

"Course" means a natural or artificial channel for passage of water.

"Critical Depth" means that particular depth of flow in a channel or conduit with a given discharge at which the specific energy is at a minimum.

"Critical Flow" means flow at critical depth.

"Critical Storm" means that storm intensity for which it and less intense but more frequent storms will require peak runoff rates be reduced to that of the runoff from a one-year frequency storm under predevelopment conditions. Such reduction is needed to avoid greater peak flows downstream and additional erosion in natural channels. For more intense but less frequent storms the runoff level need only to be controlled to pre-development conditions for the same storm.

"Culvert" means a closed conduit for the passage of surface drainage water under a highway, railroad, embankment, or other impediment.

"Dam" means a barrier constructed across a watercourse for the purpose of (a) creating a reservoir, and (b) diverting water into a conduit or channel.

"Detention" means the temporary delaying of stormwater runoff.

"Deterministic Models" means a system by whose operation the characteristics of another similar system may be predicted. A model is generally a small-scale reproduction of the prototype but may be larger or geometrically distorted.

"Discharge" means the rate of flow, or volume of water flowing in a stream or conduit at a given place and within a given period of time.

"Discharge Control Structure" means a structure designed to control the rate of stormwater discharge. Also called "Flow Control Structure."

"Drainage" means in general, the removal of surface or groundwater from a given area either by gravity or by pumping. The term is applied herein to surface water.

"Drainage Area" means (1) The contributing area to a single drainage basin, expressed in acres, square miles, or other unit of area. Also, called Catchment Area, Watershed, and River Basin. (2) The area served by a drainage system receiving storm and surface water or by a watercourse.

"Drainage System" See "Storm Drainage System."

"Drainageway" means a route or course along which water moves or may move to drain an area.

"Drop Inlet Structure" means a vertical structure in a drainageway for the purpose of dropping water to a lower level.

"Encroachment" means the use of a flood plain for any purpose that would alter the natural flooding process.

"Erosion (Soil Erosion)" means the wearing away of the land surface by running water, wind, ice or other geological agents, including such processes as gravitational creep, or detachment and movement of soil or rock fragments by water, wind, ice, or gravity.

"Excess Stormwater" means that portion of stormwater runoff which exceeds the transportation capacity of natural drainage channels serving a specific watershed.

"Flood" means the temporary inundation of any land not normally covered by water due to heavy rainfall or runoff, or due to temporary rise on the level of rivers, streams, water-courses or lakes.

1. "Average Annual Flood" means a flood equal to the mean of discharges of all the maximum annual floods during the period of record.

2. "Regional Flood" means the name applied to the 100-year flood in flood plain information reports. The 100-year flood has a one percent probability of being equalled or exceeded in a period of 100 years.

3. "Maximum Probable Flood" means the largest flood discharge believed possible considering the meteorologic conditions and snow cover on the watershed.

"Flood Plain" means the area described by the perimeter of the flood under consideration. That portion of a river or stream valley which is covered with water when the stream over-flows its banks at flood stage.

"Flood Plain Management" means control of the use of land subject to flooding.

"Flow Control Device/Structures" means a hydraulic mechanism such as an orifice or weir having known discharge characteristics or for which the discharge characteristics can be determined. A flow control structure is that structure which contains or includes a flow control device.

"Floodway" means the channel of the watercourse and those portions of the adjoining flood plain which are used to convey the regional flood.

"Flood Stage" means an arbitrarily fixed gauge height or elevation above which a rise in the water surface elevation is termed a flood. It is commonly fixed as the stage at which overflow of the normal banks or damage to property would begin.

"Flow Routing" means the derivation of an outflow hydrograph for a given stream reach from known values of upstream inflow.

"Frequency Curve" means a graphical representation of the frequency of occurrence of specific events.

"Gabion" means a cage or wire basket filled with stones and deposited with others to protect against erosion.

"Grade" means (1) The slope of a channel, conduit, or natural ground surface, usually expressed in terms of the ratio or percentage of number of units of vertical rise or fall per unit of horizontal distance. (2) The elevation of the invert of the bottom of a pipeline, culvert, sewer, or similar conduit. (3) The finished surface of a road bed, top of embankment, or bottom of an excavation.

"Gradient" See "Slope."

"Head" means (1) The height of the free surface of a fluid above a point of reference in the system. (2) The energy, either potential or kinetic, possessed by a unit weight of fluid, expressed as height it would have to fall to release the average energy possessed.

"Homogeneous Area" means (1) A drainage area which has relative uniform runoff characteristics; i.e., land use, slope, soil treatment, etc. are relatively uniform. (2) Based on runoff curve numbers, a homogeneous area is one having curve numbers in the range of 65 to 80.

"Hydraulics" means a branch of science that deals with practical applications of the mechanics of water movement.

"Hydraulic Grade Line" means a hydraulic profile of the piezometric level of water at all points along a line.

"Hydrograph" means a graph showing the discharge stage, velocity, or other property of water with respect to time for a given point under consideration.

"Hydrology" means the applied science concerned with the water of the earth in all its states. It deals with the processes governing the depletion and replenishment of the water resources of the land areas of the earth.

"Infiltration" means (1) The entering of water through the pores of a soil or other porous medium. (2) The absorption of liquid by the soil, either as it falls as precipitation or from a stream flowing over the surface.

"Initial Drainage System" means that part of the storm drainage system which is used regularly for collecting, transporting, and disposing of storm runoff, snow melt, and miscellaneous minor flows. The capacity of the initial drainage system should be equal to the maximum rate of runoff to be expected from a design storm which may have a frequency of occurrence of once in two, five, or ten years. The initial system is also termed the "convenience system," "minor system," or the "storm sewer system," and may include many features ranging from curbs and gutters to storm sewer pipes and open drainageways.

"Inlet" means (1) A surface connection to a drain pipe. (2) An opening into a storm sewer system for the entrance of surface or stormwater. (3) A structure at the diversion end of a conduit.

"Inlet Control" means control of the relationship between headwater elevation and discharge by the inlet or upstream end of any structure through which water may flow.

"Inundation" See "Flood."

"Lag" means the increment of time from the center of mass of that portion of rainfall that runs off to the peak rate of runoff from the watershed. The lag of a watershed may be thought of as a weighted time of concentration.

"Lining" means impervious material such as concrete, clay, grass, etc., placed on the bottom and/or sides of a ditch, channel, or reservoir to prevent erosion.

"Major Drainage System" means that storm drainage system which carries the runoff from a storm having a frequency of occurrence of once in 100 years. The major system will function whether or not it has been planned and designed, and whether or not improvements are situated wisely in respect to it.

The major system usually includes many features such as streets, ravines, and major drainage channels. Storm sewer systems may reduce the flow in many parts of the major system by storing and transporting water underground.

"Mean Velocity" means the average velocity of water flowing in a channel or conduit at a given cross section or in a given reach. Also called average velocity. It is equal to the discharge divided by the cross-sectional area of the channel or conduit.

"Non-Homogeneous Area" means (1) A drainage area in which the runoff characteristics are dissimilar. (2) Based on runoff curve numbers, a non-homogeneous area is one having curve numbers outside the range of 65 to 80.

"Orifice" means an opening with closed perimeter and of regular form in a plate, wall, or partition through which water may flow.

"Open-Channel Flow" means flow in any open or closed conduit where the water surface is free; that is, where the water surface is at atmospheric pressure.

"Outfall" means the location where storm runoff discharges from a sewer or conduit. Also applies to the outfall sewer or channel which carries the storm runoff to the point of outfall.

"Outlet Control" means control of the relationship between the headwater elevation and the discharge by the outlet or downstream end of any structure through which water may flow.

"Overflow" means the excess water that overflows the ordinary limits such as the stream banks, the spillway crest, or the ordinary level of a container.

"Precipitation" means the total measurable supply of water received directly from clouds as rainfall, snow, hail or sleet; ususally expressed as depth in a day, month, or year.

"Post-development" means the state of condition of the earth's surface after urbanization occurs. Other terms are developed, future, and after development.

"Pre-development" means the state of condition of the earth's surface prior to urbanization. Other terms are undeveloped, present, and before development.

"Rainfall Duration" means the length of time of the rainfall event from beginning to end, usually expressed in hours.

"Rainfall Event" means a fall of rain or precipitation in the form of water which occurs in a particular period ot time.

"Rainfall Intensity" means amount of rainfall occurring in a unit of time, converted to its equivalent in inches per hours at the same rate.

"Reach" means a hydraulic engineering term to describe a longitudinal segment of a stream or river within which flood heights are primarily controlled by man-made or natural obstructions or constrictions.

"Recurrence Interval" means the average interval of time within which a given event will be equalled or exceeded once.

"Regulatory Area" means that portion of the flood plain subject to inundation by the 100-year flood that has been designated as a portion of the major drainage system.

"Return Period" See "Recurrence Interval."

"Runoff" means the portion of rainfall, melted snow or irrigation water that flows across the ground surface and eventually is returned to streams.

"Accelerated Runoff" means increased runoff due to less permeable surface area primarily caused by urbanization.

"Peak Rate of Runoff" means the maximum rate of runoff for any storm.

"Runoff, Total Direct" means the total volume of flow from a drainage area for a definite period of time such as a day, month or rainfall event which reaches stream channels.

"Runoff Volume" means the total quantity or volume of runoff during a specified time period. It may be expressed in acre-feet, in inches depth of the drainage area, or in other units of vol
"Sediment" means material of soil and rock origin transported, carried, or deposited by water.

"Slope" means the inclination or grade of a channel, conduit, or natural ground surface, usually expressed in terms of the precentage of units of vertical rise or fall per unit of horizontal distance.

"Slope Protection" means soil cover on a slope surface to minimize or eliminate erosion and/or to ensure stability of a soil slope steeper than the normal angle or repose of the soil. Examples are: low maintenance ground cover such as crown vetch and spreading Juniper, dumped rock or riprap, stone filled Gabions, bituminous or concrete paving, and concrete or timber cribbing.

"Spillway" means a waterway in or about a hydraulic structure for the escape of excess stormwater.

"Storage (with respect to channel design)" means the control, retention, or detention of runoff.

> "Detention Storage" means storm runoff collected and stored for a short period of time and then released at a controlled rate (dry pond).
>
> "Retention Storage" means storm runoff collected and stored for a short period of time and which is released at a controlled rate leaving in the facility a minimum pool of water. This facility is often associated with water-related recreational or aesthetic uses (wet pond).
>
> "Upstream On-site Storage" means the storage of storm runoff water near the points of rainfall occurrence, usually applicable to rooftop ponding, parking lot ponding, and small drainage basins.
>
> "Downstream Storage" means the storage of storm runoff water at some distance from the points of rainfall occurrence but before it reaches areas where it may endanger lives or property.
>
> "Offstream Storage" means the temporary storage of storm runoff water away from the main channel of flow.
>
> "Channel Storage" means storm runoff water present in a channel at any given time. Generally considered in the attenuation of the peak of a flood hydrograph moving downstream.
>
> "Onstream Storage" means the temporary storage of storm runoff water behind embankments or dams located on the channel.

"Storage (with respect to runoff analysis)":

> "Detention Storage" means that water that is detained on the surface during a storm and does not become runoff until some time after the storm has ended.
>
> "Depression Storage" means that portion of the rainfall that is collected and held in small depressions and does not become part of the general runoff.

"Storm, Initial Design" means that storm used for design purposes, the runoff from which is used for sizing the initial storm drainage system. It usually is of such magnitude that it can be expected to occur only once each two, five, or ten years.

"Storm, Major Design" means that storm used for design purposes, the runoff from which is used for sizing the major drainage works. It usually is a storm of such magnitude that it can be expected to occur once every 100 years.

"Storm Drainage System" means the surface and sub-surface system for the removal of water from the land, including both the natural elements of streams, gullies, ravines, marshes, swales and ponds whether of an intermittent or continuous nature and man-made elements which include conduits and appurtenant features, culverts, ditches, channels, storage facilities, streets and the storm sewer system.

"Stormwater Management" means the application of various techniques for mitigating the deleterious effects of land use on runoff.

"Street Classifications":

1. "Local Street" means a minor traffic carrier within a neighborhood which usually is characterized by two moving lanes and parking along the curb.

2. "Collector Street" means a street which collects and distributes traffic between arterial and local streets.

3. "Arterial Street" means a traffic carrier which permits rapid and relatively unimpeded vehicular movement throughout the area.

4. "Freeway" means a freeway permits rapid and unimpeded movement of traffic through and around a city. Access to a freeway is completely controlled.

"Street Flow" means the total flow of stormwater runoff in a street, usually the sum of gutter flow on each side of the street. Also, the total flow where there are no curbs and gutters.

"Surface Flow" or "Sheet Flow" means the surface flow from rainfall on ground surfaces, pavements, or other exposed surfaces until such flow reaches a gutter, ditch, swale, inlet, or other point of concentration.

"Swale" means a drainage channel, normally grass-lined, with relatively flat side slopes (5:1 or less).

"Time of Concentration" means the time required for stormwater runoff to travel from the hydrologically most remote section of the drainage area to the point under consideration.

"Time of Flow" means the time required for water to flow in a storm sewer or channel from the point where it enters to a particular point.

"Time of Inlet" means the time of concentration for the inlet, usually includes ground surface flow time and gutter flow time.

"Unit Hydrograph" means a unit hydrograph is the hydrograph of one unit of direct runoff from a drainage area resulting from a uniform intensity of storm of unit length.

"Velocity of Approach" means the mean velocity immediately upstream from a weir, dam, or other hydraulic structure.

"Watercourse" means a channel in which a flow of water occurs either continuously or intermittently in a definite direction. The term applies to either natural or artificially constructed channels.

"Watershed" See "Drainage Area."

"Water Level":

1. "Design High Water Level" means calculated water level in storage facilities associated with the initial design storm and volume requirements criteria. Also, predicted water level in ditches and swales under initial design storm condition.

2. "Maximum Water Level" means the predicted water level in storage facilities or drainage system in overflow condition used in conjunction with flood flow routing of the major design storm.