CITY ENGINEERING DIVISION

CITY OF COLORADO SPRINGS

October 12, 1994

Re: **REVISIONS TO THE CITY OF COLORADO SPRINGS/EL PASO COUNTY DRAINAGE CRITERIA MANUAL -- INITIAL DESIGN STORM AND ALLOWABLE STREET CAPACITIES**

To Whom It May Concern:

Several revisions to the Drainage Criteria Manual were approved by the City Council on October 11, 1994. The revisions relate to the Initial Design Storm Frequency and Allowable Street Capacities. These changes become **effective for use in the City of Colorado Springs (not El Paso County) on October 12, 1994.**

The following information is being distributed at this time:

- City Council Resolution
- October 1994 Summary of Drainage Criteria Changes (Resolution Attachment, 1 page)
- Comparison of Existing and Proposed Criteria (1 page)
- Rainfall Intensity Curve Graph (5, 10 and 100-year storm frequency, 1 page)
- Allowable Street Capacity Graphs (4 pages)

It is anticipated the specific changes to pages of the Drainage Criteria Manual will be distributed around the beginning of 1995. If there are specific questions regarding the criteria revisions, please contact the Stormwater and Subdivision Section staff at 719-578-6212.

Gary R. Haynes

Gary R. Haynes
City Engineer

Attachments

c: Drainage Criteria Manual Holders
City Engineering Division Staff
Alan Morrice, El Paso County Stormwater Supervisor

30 S. Nevada Ave., Suite 403 • TEL 719-578-6606 FAX 719-578-6216
Mailing Address: Post Office Box 1575, Mail Code 435 • Colorado Springs, Colorado 80901-1575

RESOLUTION NO. 170-94

A RESOLUTION ADOPTING CERTAIN AMENDMENTS AND REVISIONS TO THE CITY OF COLORADO SPRINGS/EL PASO COUNTY DRAINAGE CRITERIA MANUAL

WHEREAS, The City of Colorado Springs/El Paso County Drainage Criteria Manual was adopted by City Council on September 8, 1987, and revised and amended on November 1, 1991, as part of the Subdivision Policy Manual; and

WHEREAS, The Cottonwood Creek Task force recommended certain drainage criteria revisions to the Initial Design Storm Frequency and Allowable Street Capacities; and

WHEREAS, On April 12, 1994, City Council in Resolution No. 60-94 authorized a study of technical criteria as part of the interim adoption of the Cottonwood Creek Drainage Basin Planning Study; and

WHEREAS, City Staff and their Study Contractors, Resource Consultants and Engineers, and J.R. Engineering Ltd. upon evaluation of the City's drainage criteria and the drainage criteria of other Front Range communities have proposed revisions to the Initial Design Storm Frequency and Allowable Street capacities; and

WHEREAS, Upon presentation to the Housing and Building Authority Land Use Committee at their September 2, 1994 meeting and to the City/County Drainage Board at their September 15, 1994 meeting, both bodies expressed support for the proposed criteria changes; and

WHEREAS, The criteria changes are prudent with respect to the cost effectiveness of drainage facility construction while providing acceptable levels of service;

NOW THEREFORE BE IT RESOLVED BY THE CITY COUNCIL OF THE CITY OF COLORADO SPRINGS:

Section 1. The City of Colorado Springs/El Paso County Drainage Criteria Manual is hereby revised and amended to change the initial design storm frequency from a ten year storm to a five year storm; and allowable street capacities are revised and amended per the attached "October 1994 Summary of Drainage Criteria Changes."

Section 2. The revisied City of Colorado Springs/El Paso County Drainage Critertia Manual become effective for use in the City of Colorado Springs.

Dated at Colorado Springs, Colorado, on this 11th Day of October, 1994.

Mayor

ATTEST:

City Clerk

OCTOBER 1994 SUMMARY OF DRAINAGE CRITERIA CHANGES

PRESENTATION TO CITY COUNCIL
OCTOBER 11, 1994

By City Council Resolution 170-94, the following changes to the 1987, revised 1991 City of Colorado Springs drainage criteria will be effective for use in the City on October 12, 1994:

1) The storm frequency for the design of the initial drainage system shall be the **FIVE YEAR** storm.

2) Allowable flow depth in Type 1 (8 inch vertical) curb shall be 6 inches.

3) Maximum allowable flow rates, per side, for the initial storm shall be as follows:
   a. Hillside Residential, ramp curb.....15 cubic feet per second
   b. Hillside Residential, vertical curb.....25 cubic feet per second
   c. Residential, ramp curb.....20 cubic feet per second
   d. Residential, vertical curb.....34 cubic feet per second
   e. Collector, vertical curb.....34 cubic feet per second
   f. Arterial, vertical curb.....34 cubic feet per second, and must have one ten foot lane free of water in each direction

4) Allowable 100 year crossflow in Arterial streets at a low point shall be the lesser of:
   a. 12 inches depth at the gutter flowline
   b. 4 inches depth at the crown

SCHEDULE FOR IMPLEMENTATION

September 2, 1994	presentation to HBA Land Use Committee
September 15, 1994	presentation to Drainage Board
October 11, 1994	Resolution by City Council
October 12, 1994	effective date, interim release and notification
January 1, 1995	projected publication of revised DCM chapters with new tables, charts, and graphs
date unknown	technical workshops, if needed

DISCUSSION OF LEVELS OF SERVICE CHANGES

Initial Storm

Allowable flow depths will be experienced more frequently (5 yr. vs. 10 yr. design)

Street flows in residential streets with vertical curb will be one inch deeper and will cross the street crown. This provides 2" of freeboard for blockage.

Street flows on steep grades will have flow limitations to prevent excessive velocities; there is no such limitation in the present criteria.

Street flows in Arterial streets will spread 8 feet farther from the curb, but will always provide 10 feet free of water in each direction.

Major Storm

During 100 year storms, vehicles may drive through 4 inches of flow depth at the street crown as opposed to none presently.

COMPARISON OF EXISTING AND PROPOSED CRITERIA

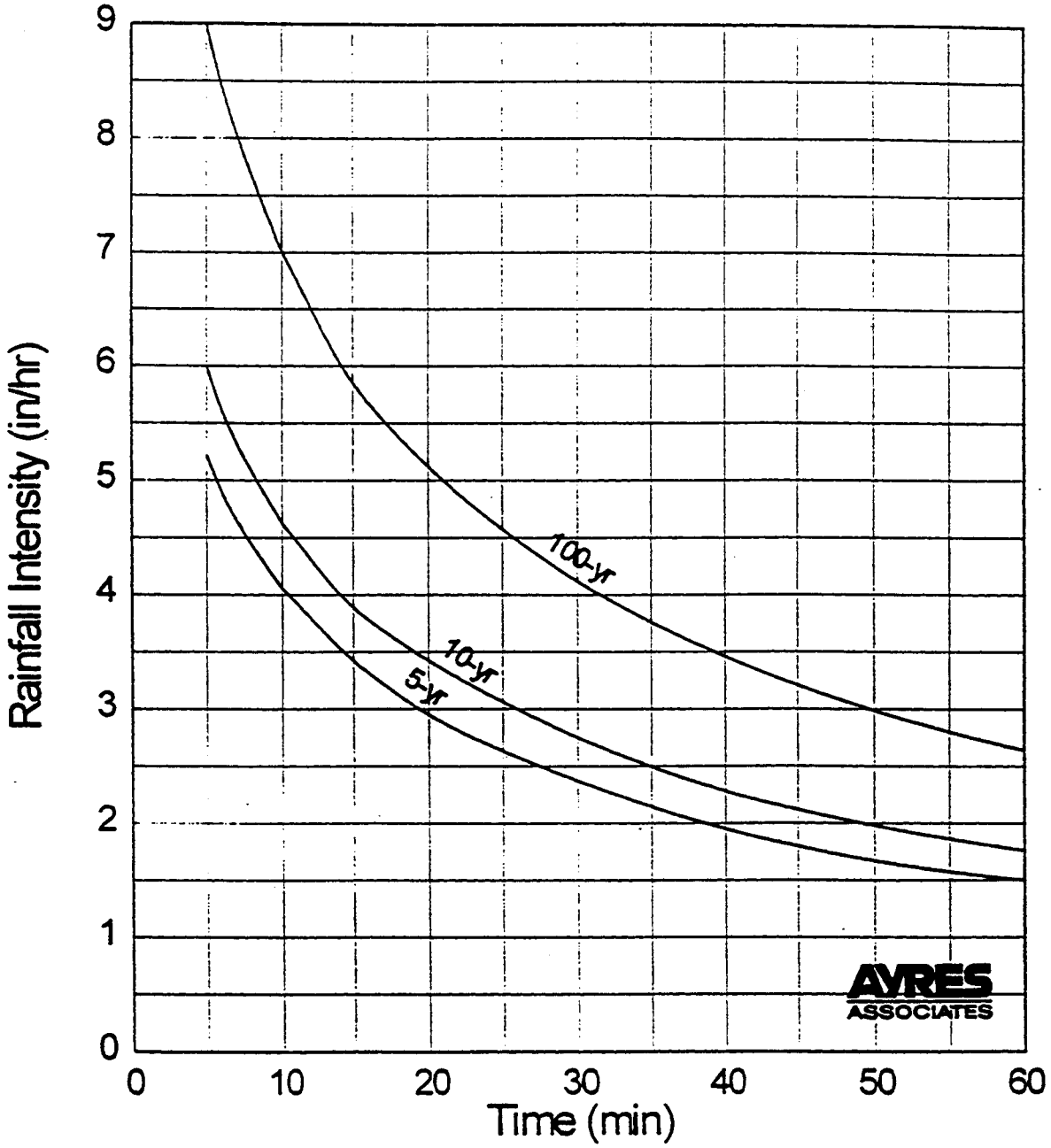
INITIAL STORM:

STREET TYPE	OLD	NEW
Hillside Residential ramp curb	flow spread to crown, maximum 25 cfs. per side, whichever is more restrictive	flow spread to crown max. 15 cfs. per side
Hillside Residential vertical curb	flow spread to crown, maximum 25 cfs. per side, whichever is more restrictive	6" allowable depth @ flowline max. 25 cfs. per side
Residential Street ramp curb	flow spread to crown	flow spread to crown max. 20 cfs. per side
Residential Street vertical curb	flow spread to crown	6" allowable depth @ flowline max. 34 cfs. per side
Collector Street	20 foot flow spread	6" allowable depth @ flowline, max. 34 cfs. per side, no overtopping the crown
Arterial Street	flow may encroach onto one outside lane	6" allowable depth @ flowline, max. 34 cfs. per side, one ten foot lane free of water in each direction

MAJOR STORM:

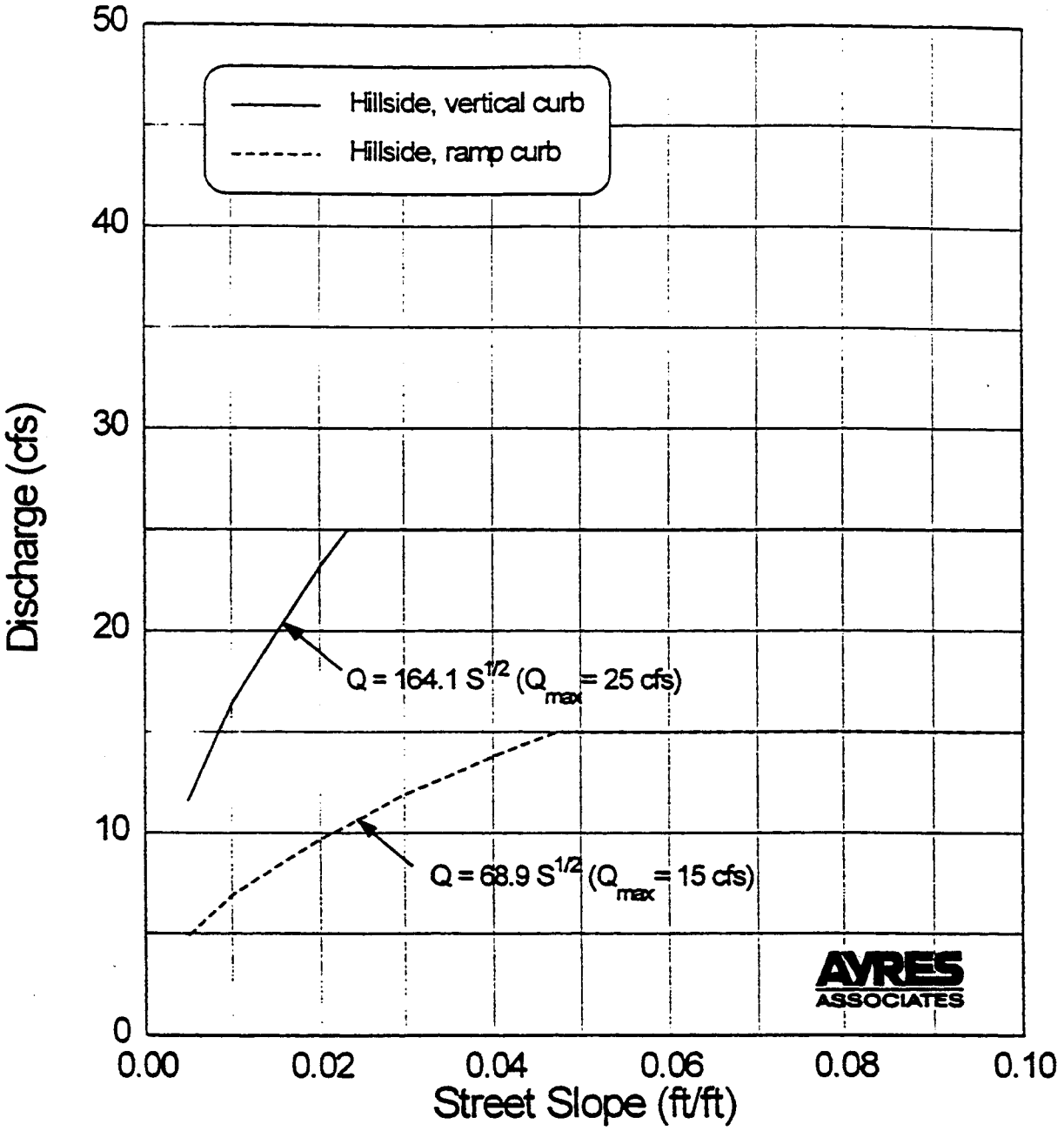
STREET TYPE	OLD	NEW
Hillside Residential Residential Streets Collector Streets	12" max. depth @ flowline no adjacent flooding	NO CHANGE
Arterial Streets	8" max. depth @ flowline (no curb overtopping)	NO CHANGE

CROSS FLOWS: No changes to any street types for the initial storm.
Only change for Major Storm is the Arterial street will now allow 12" max. depth @ flowline and 4" max. depth @ crown whichever is more restrictive. Existing criteria allows no crossflow.



Interim Release October 12, 1994 , Rainfall Intensity Curves
City Of Colorado Springs Drainage Criteria Manual

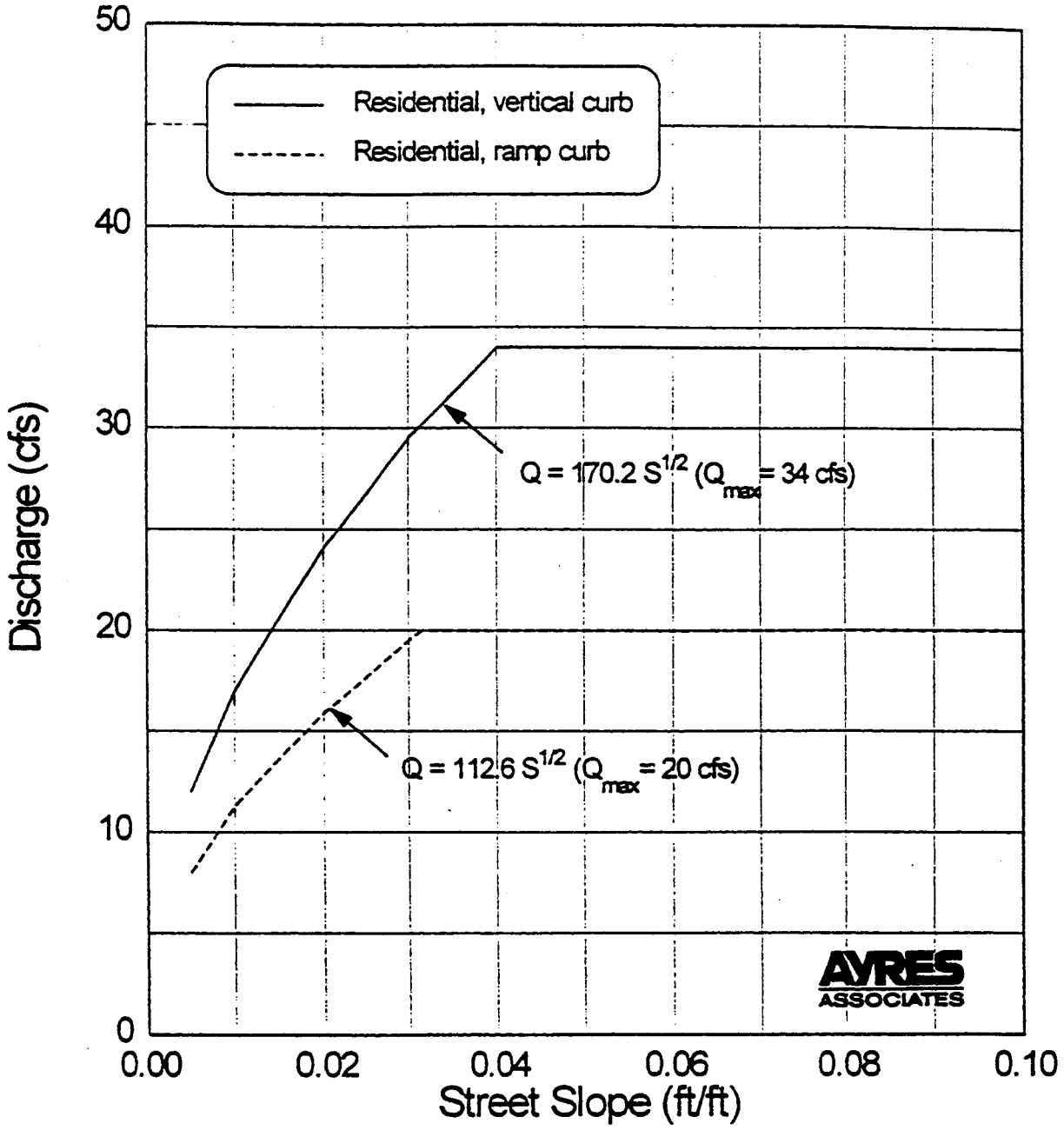
HILLSIDE MINOR RESIDENTIAL



Interim Release October 12, 1994
City of Colorado Springs

Use this graph to determine the allowable street capacity per side, initial storm, for the typical street section using a 2% crown.

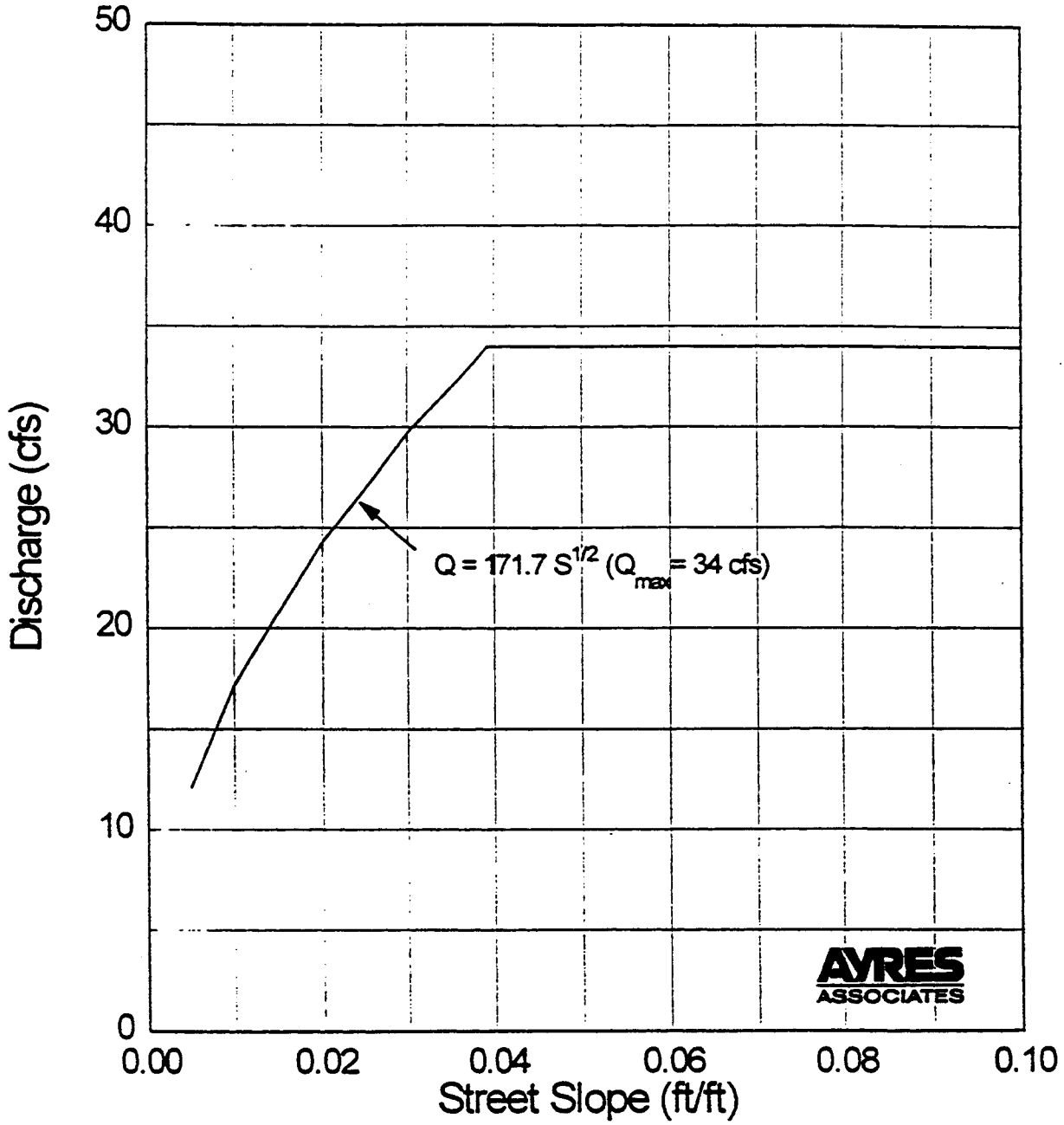
RESIDENTIAL STREET (34′ Flowline to flowline)



Interim Release October 12, 1994
City of Colorado Springs

Use this graph to determine the allowable street capacity per side, initial storm, for the typical street section using a 2% crown.

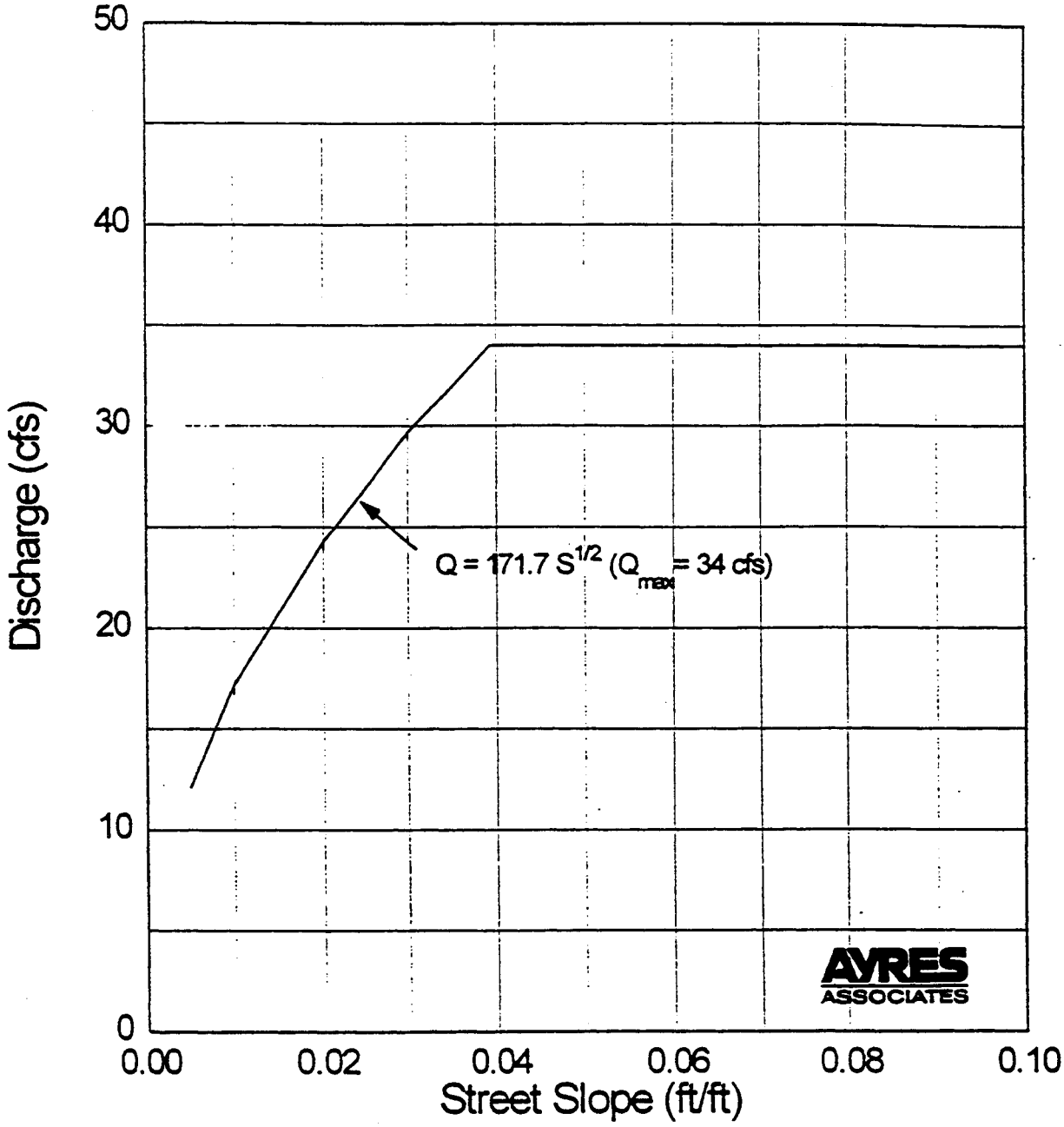
COLLECTOR STREETS (Major and Minor)



Interim Release October 12, 1994
City of Colorado Springs

Use this graph to determine the allowable street capacity per side, initial storm, for the typical street section using a 2% crown. No flow may cross the crown.

ARTERIAL STREETS (Major and Minor)



Interim Release October 12, 1994
City of Colorado Springs

Use this graph to determine the allowable street capacity per side, initial storm, for the typical street section using a 2% crown. The curve corresponds to 6" depth @ flowline, 20 foot flow spread. No flow may cross the crown. Must keep one ten foot lane free of water in each direction.

INTRODUCTION - GENERAL PROVISIONS

A. Short Title

This publication together with all future amendments shall be known as the "The City of Colorado Springs/El Paso County Drainage Criteria Manual" (hereafter called "Manual") BEING PART OF AND SUBJECT TO THE City of Colorado Springs Subdivision Policy Manual and El Paso County Land Development Code and Subdivision Criteria Manual.

B. Jurisdiction

This manual shall apply to all land within the boundaries of El Paso County, including public lands. This manual shall apply to all facilities constructed on City/County ROW, easements dedicated for public use, and to all privately owned and maintained stormwater facilities.

C. Purpose

Presented in this manual is the minimum design and technical criteria for the analysis and design of storm drainage facilities. All subdivisions, resubdivisions, planned unit development, or any other proposed construction submitted for approval under the provisions of the manual shall include adequate storm drainage system analysis and appropriate drainage system design. Such analysis and design shall meet or exceed the criteria set forth herein. Options to the provisions of this criteria may be suggested by the applicant. The burden of proof that the options are equal or better is the responsibility of the applicant.

D. Amendment and Revisions

The policies and criteria may be amended as new technology is developed or if experience gained in the use of this manual indicates a need for revision. All changes will require the review and recommendation of the City/County. Technical criteria changes will be recommended by the Engineering Department. Any revisions will require adoption by resolution of the Board of County Commissioners and/or City Council following a public hearing thereon. The City/County Engineer shall monitor the performance and effectiveness of the criteria contained in the manual and will recommend amendments and revisions after one year, approximately three years, and as needed.

E. Enforcement Responsibility

It shall be the duty of the Board of County Commissioners acting through the Director of Public Works and the City Council acting through the City Engineer to enforce the provisions of this manual. The Floodplain Administrator also shall enforce those floodplain regulations within his jurisdiction.

F. Review and Approval

The City/County will review all drainage submittals for compliance with the manual. An approval by the City/County does not relieve the owner, engineer, or designer from the responsibility of insuring that the calculations, plans, specifications, construction, and record drawings are in compliance with the manual as stated in the owner's engineer's certification, as required in Chapter 4.6.

G. Interpretation

In the interpretation and application of the provisions of the manual the following shall govern:

1. In its interpretation and application, the provisions shall be regarded as the minimum requirements for the protection of the public health, safety, comfort, morals, convenience, prosperity, and welfare of the residents of the City and County.

2. Whenever a provision of this manual or any provision in any law, ordinance, resolution, rule, or regulation, contain any restrictions covering any of the same subject matter, the law, ordinance, resolution, rule or regulation shall govern.

3. This manual shall not abrogate or annul any permits or approved drainage reports and construction plans issued before the effective date of this manual.

H. Use of Manual

This City of Colorado Springs/El Paso County Drainage Criteria Manual (herein referenced as "manual") has been divided into four sections which contain twelve chapters and an appendix. The manual format is listed on page D. Each manual has been numbered and is recorded with the name and address of the owner. Periodic updates will be mailed to those registered owners of the manual.

Section I of the manual contains the drainage policies, criteria, finance and administration text. This section should be referred to by City/County administrators and project owners. Chapter 4 in Section I contains the City/County submittal and review procedures and should be read by everyone.

Section II of the manual contains hydrologic analysis procedures. This will be one of the more heavily used sections of the manual by engineers.

Section III of the manual contains suggested design methods and minimum standards. Procedures outlined in this section may be directly utilized by the engineer to solve drainage problems. The engineer may also use methods demonstrated to the City/County as equal or superior.

I. Acknowledgment

This manual was prepared by HDR Infrastructure, Inc. under contract with the City of Colorado Springs - Department of Public Works and El Paso County Department of Transportation (Public Works). The preparation of this manual involved a considerable effort from the staff of the City and County as well as participation by members of the Technical Advisory Committee.

Technical Advisory Committee Members

Bob Roumph	Corps of Engineers
Anita Culp	Corps of Engineers
Roger J. Sams	Gilbert, Meyer & Sams, Inc.
Richard N. Wray	Simons, Li & Associates
Russ Livingston	US Geological Survey
Glade Wilkes	SCS
John Liou	FEMA
Ollie Watts	Consulting Engineer of Colorado Springs
Jerry Weiss	Consulting Engineer of Colorado Springs
Larry Lang	Colorado Water Conservation Board
Dan Bunting	Flood Plain Administrator

City Staff

DeWitt Miller	Director of Public Works
Gary Haynes	City Engineer
Bruce Thorson	Assistant City Engineer
Bill McCall	Study Coordinator

County Staff

Max Rothchild	Director of Public Works
Don Smith	Manager, Engineering Division
Alan Morrice	Storm Water Management Supervisor

HDR Staff

Bob Looper	Project Manager
Neil Grigg	Senior Consultant
Jim Abbott	Project Principal

J. Drainage Criteria Manual Authority

The following resolutions by the City of Colorado Springs and by El Paso County provide the date the Manual becomes effective and their jurisdictional limits.

Resolution No. 135-87

A RESOLUTION ADOPTING THE CITY OF COLORADO SPRINGS/EL PASO COUNTY DRAINAGE CRITERIA MANUAL AS A PART OF THE CITY OF COLORADO SPRINGS SUBDIVISION POLICY MANUAL

WHEREAS, the City of Colorado Springs' Department of Public Works has jointly with El Paso County Department of Transportation prepared a City/County joint Drainage Criteria Manual; and

WHEREAS, a joint City/County Drainage Criteria Manual will benefit the entire community; and

WHEREAS, the manual contains technical engineering procedures and criteria which updates and adds to current engineering criteria; and

WHEREAS, the City of Colorado Springs will benefit from an improved stormwater management program;

NOW, THEREFORE, BE IT RESOLVED BY THE CITY COUNCIL OF THE CITY OF COLORADO SPRINGS:

Section 1. That the City of Colorado Springs/ El Paso County Drainage Criteria Manual become a part of the City of Colorado Springs' Subdivision Policy Manual.

Section 2. That the City of Colorado Springs/El Paso County Drainage Criteria Manual become effective for use in all subdivisions and construction projects within the public jurisdiction for all plans submitted after October 1, 1987.

Dated at Colorado Springs, Colorado, this 8th day of September, 1987.

Mayor

ATTEST:

City Clerk

Resolution No. 175-91

A RESOLUTION ADOPTING CERTAIN AMENDMENTS
AND REVISIONS TO THE CITY OF COLORADO SPRINGS
EL PASO COUNTY DRAINAGE CRITERIA MANUAL

WHEREAS, The City of Colorado Springs/El Paso County Drainage Criteria Manual (hereinafter called "Manual" was adopted by City Council on September 8, 1987 as part of the Subdivision Policy Manual; and

WHEREAS, Experience gained in the use of the Manual by both the public and private sector since its adoption indicates the need for certain revisions to the Manual; and

WHEREAS, The City of Colorado Springs/El Paso County Drainage Board (Drainage Board) at their September 19, 1991 meeting approved certain amendments and revisions to the Manual; and

WHEREAS, The amendments and revisions approved by the Drainage Board will be a benefit to the City of Colorado Springs by providing for an improved Stormwater Management Program;

NOW THEREFORE, BE IT RESOLVED BY THE CITY COUNCIL OF THE CITY OF COLORADO SPRINGS:

Section 1. That the City of Colorado Springs Subdivision Policy Manual is hereby revised by adopting the amendments and revisions to the City of Colorado Springs/El Paso County Drainage Criteria Manual which were approved by the Drainage Board on September 19, 1991.

Section 2. That the revised Subdivision Policy Manual become effective for use in all subdivisions and construction projects in the City of Colorado Springs on November 1, 1991.

Dated at Colorado Springs, Colorado, this 12th day of November, 1991.

ATTEST:

Deputy City Clerk

Mayor

Commissioner Meier moved adoption of the following Resolution:

BEFORE THE BOARD OF COUNTY COMMISSIONERS
OF THE COUNTY OF EL PASO, STATE OF COLORADO

RESOLUTION NO. 87-178A, Land Use-67A

WHEREAS, the El Paso County Department of Transportation requests approval of the City of Colorado Springs/El Paso County Drainage Criteria Manual which sets forth provisions for drainage policies, criteria, finance and administration as herein described; and

WHEREAS, a public hearing was held by the El Paso County Planning Commission on April 21, 1987, upon which date the Planning Commission did by formal resolution recommend approval of the subject Manual; and

WHEREAS, a public hearing was held by this Board on June 22, 1987; and

WHEREAS, based on the evidence, testimony, exhibits, recommendations of the El Paso County Planning Commission, comments of the El Paso County Land Use Department, comments of public officials and agencies, and comments from all interested parties, this Board finds as follows:

1. That proper publication and public notice was provided as required by law for the hearings before the Planning Commission and the Board of County Commissioners of El Paso County.

2. That the hearings before the Planning Commission and the Board of County Commissioners were extensive and complete, that all pertinent facts, matters and issues were submitted, and that all interested parties were heard at those hearings.

3. That all data, surveys, analyses, and studies, as are required by the State of Colorado and El Paso County have been submitted, reviewed, and found to meet the intent of the General Provisions of the El Paso County Subdivision Regulations.

4. That the proposal shall hereby amend the Subdivision Criteria Manual for El Paso County by deleting present Section DC (Drainage Criteria) in its entirety and incorporating by reference the Drainage Criteria Manual.

5. That for the above-stated and other reasons, the proposed Manual is in the best interest of the health, safety, morals, convenience, order, prosperity, and welfare of the citizens of El Paso County.

NOW, THEREFORE, BE IT RESOLVED that the following City of Colorado Springs/El Paso County Drainage Criteria Manual, as more particularly described in Exhibit A, as recorded at Book 5561 , Page 10 , of the El Paso County Clerk and Recorder records be approved;

BE IT FURTHER RESOLVED that the record and recommendations of the El Paso County Planning Commission be adopted.

DONE THIS 22nd day of June, 1987, at Colorado Springs, Colorado.

BOARD OF COUNTY COMMISSIONERS
OF EL PASO COUNTY, COLORADO

ATTEST:

By Nancy Morrison
Chairman

Doris Hardy
Deputy County Clerk

Commissioner Campbell seconded the adoption of the foregoing Resolution. The roll having been called, the vote was as follows:

Commissioner Morrison	aye
Commsisioner Campbell	aye
Commissioner Whittemore	no
Commissioner Meier	aye

The Resolution was adopted by 3-1 vote of the Board of County Commissioners of the County of El Paso, State of Colorado.

Commissioner Howells moved adoption of the following Resolution:

BEFORE THE BOARD OF COUNTY COMMISSIONERS
OF THE COUNTY OF EL PASO, STATE OF COLORADO

RESOLUTION NO. 91-334, Land Use-68

WHEREAS, the El Paso County Department of Public Works requests approval of Amendments to the City/County Drainage Criteria Manual as herein described by the addition of clarifications to criteria without changing intent, correction of numerous typographical errors and modification of criteria; and

WHEREAS, a public hearing was held by the El Paso County Planning Commission on August 20, 1991, upon which date the Planning Commission did by formal resolution recommend approval of the subject Amendments; and

WHEREAS, a public hearing was held by this Board on October 10, 1991; and

WHEREAS, based on the evidence, testimony, exhibits, recommendations of the El Paso County Planning Commission, comments of the El Paso County Planning Department, comments of public officials and agencies, and comments from all interested parties, this Board finds as follows:

1. That proper publication and public notice was provided as required by law for the hearings before the Planning Commission and the Board of County Commissioners of El Paso County.

2. That the hearings before the Planning Commission and the Board of County Commissioners were extensive and complete, that all pertinent facts, matters and issues were submitted, and that all interested parties were heard at those hearings.

3. That all data, surveys, analyses, and studies, as are required by the State of Colorado and El Paso County have been submitted, reviewed, and found to meet the intent of the General Provisions of the City/County Drainage Criteria Manual.

4. That the proposal shall hereby amend the City/County Drainage Criteria Manual for El Paso County.

5. That for the above-stated and other reasons, the proposed Amendments are in the best interest of the health, safety, morals, convenience, order, prosperity, and welfare of the citizens of El Paso County.

NOW, THEREFORE, BE IT RESOLVED that the Amendments to the City/County Drainage Criteria Manual more particularly described in the attached Exhibit A, including any typographical corrections, be and are hereby approved;

BE IT FURTHER RESOLVED the City/County Drainage Criteria Manual shall be further modified to reflect the required hydrologic analyses will not include the utilization of Antecedent Moisture Condition 3 and instead utilizing only Antecedent Moisture Condition 2;

AND BE IT FURTHER RESOLVED the City/County Drainage Criteria Manual attached hereto supersedes the Manual previously amended by the Board on February 11, 1988;

AND BE IT FURTHER RESOLVED that the Amendments shall be effective as of this date;

AND BE IT FURTHER RESOLVED that the record and recommendations of the El Paso County Planning Commission be adopted.

DONE THIS 10th day of October, 1991, at Colorado Springs, Colorado.

ATTEST:

Deputy County Clerk

BOARD OF COUNTY COMMISSIONERS
OF EL PASO COUNTY, COLORADO

By _____
Chairman

Commissioner Whittemore seconded the adoption of the foregoing Resolution. The roll having been called, the vote was as follows:

Commissioner Shupp	aye
Commissioner Campbell	abstain
Commissioner Whittemore	aye
Commissioner Morrison	no
Commissioner Howells	aye

The Resolution was adopted by a 3-1-1 vote of the Board of County Commissioners of the County of El Paso, State of Colorado.

CITY OF COLORADO SPRINGS/EL PASO COUNTY
DRAINAGE CRITERIA MANUAL
FORMAT

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Chapter 1	Drainage Policy
Chapter 2	Drainage Criteria
Chapter 3	Drainage Basin Fees, Procedures and Layout
Chapter 4	Report Guidelines

SECTION II - HYDROLOGY

Chapter 5	Storm Runoff Methods of Analysis

SECTION III - DESIGN METHODS

Chapter 6	Design Criteria
Chapter 7	Street Drainage/Storm Water Inlets
Chapter 8	Storm Drains and Appurtenances
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Chapter 11	Detention Storage
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- Glossary

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CHAPTER 1 DRAINAGE POLICY

1.1 General Statement

The provision of adequate drainage is necessary to preserve and promote the general health, safety, welfare and economic well-being of the community. Drainage is a regional issue that affects all governmental jurisdictions. The characteristics of drainage make it necessary to formulate a program that balances both public and private interests. Master drainage planning should be integrated on a regional level and coordinated with state and national agencies, where applicable.

When planning drainage facilities, certain underlying principles provide direction. When considered in a comprehensive manner and on a regional level with both public and private involvement, drainage facilities can be provided that will minimize flood losses and disruption, enhance the general health and welfare, and help assure optimum economic and social benefits.

The purpose of this chapter is to provide policy guidelines for storm water management in El Paso County. These policy guidelines are not regulations and do not replace the "standards" which now exist for the City/County. The policy statements form the basis for a comprehensive storm water management program including the planning, financing, design, construction, operation and maintenance of drainage facilities.

The following policy statements are presented in a sequence which reflects the logical progression of storm water management from planning through implementation. The policy statements are supported by detailed drainage criteria and minimum design standards which are set forth in Sections II and III of this manual. The effectiveness of the drainage policy and criteria in this manual will depend largely upon cooperation among the local governments private interests, and other governmental agencies. Their participation in the development of these policies, criteria and minimum design standards is the first step towards a successful storm water management program.

1.2 Drainage Planning Policies

Drainage is a service subsystem for general development in a region. Therefore, the planning of drainage facilities should be in general conformance with a comprehensive land-use plan. The comprehensive land-use plan should consider all major elements necessary for proper development and the resolution of conflicting elements.

Drainage facilities can (may) serve both a conveyance and storage function. When a channel is planned as a conveyance feature, it requires an outlet to a suitable outfall. When considered as a space demand, the provision of adequate drainage becomes a competing use for space along with other land uses. If adequate

provision is not made in a land-use plan for drainage, storm water runoff will conflict with other land uses and could result in flood damage or impair the functioning of other land-use systems. Therefore, designated flood plain areas along drainageways are being incorporated into the City/County Comprehensive Land Use Plans.

The City Code, Chapter 15, Article 6 - Flood Plain Management and the County Land Development Code zoning regulations, Section 32 - Flood Plain District outline development restrictions within flood plain areas.

PROPER PLANNING OF DRAINAGE, IN CONJUNCTION WITH THE ENVIRONMENT AND OTHER NEEDS, RESULTS IN MORE ORDERLY DEVELOPMENT AND LOWER COSTS.

1.2.1 Planning Process

Drainage planning is a complex process. Fundamentals include:

a. Initial Drainage Planning. All local and regional planning must consider the initial drainage system which transports runoff from storms which can be expected to occur on a frequent basis. The initial system must assure a minimum of drainage complaints. The initial drainage system is intended to convey storm runoff from the 10-year event. The initial drainage system must be integrated with the major drainage system to provide overflow capacity for major storms.

b. Major Drainage Planning. All planning must consider the major drainage system necessary to transport the 100-year event. The major drainage system planned is intended to prevent the loss of life and major damage, and will function when the initial drainage is exceeded.

There is no "cut off" point for transitioning design of facilities from the 10-year to the 100-year runoff. The overall conveyance system must meet both the 10-year and 100-year design storm criteria as contained in the manual.

The process of planning initial an major drainage systems starts with a Drainage Basin Planning Study. A Drainage Basin Planning Study should be directed by a broad, general framework of goals. The City/County goals are:

- Economic efficiency
- Regional scope
- Environmental preservation and enhancement
- Social and recreational enhancement
- Responsible funding and implementation policy
- Health, safety and welfare of their citizenry

These goals greatly influence the type of drainage subsystem selected. Planning for drainage facilities should be related to the goals of the region and should be looked upon as a subsystem of the total system.

STORM DRAINAGE IS A PART OF THE TOTAL ENVIRONMENTAL SYSTEM. THEREFORE, STORM DRAINAGE PLANNING AND DESIGN SHALL BE COMPATIBLE WITH COMPREHENSIVE LAND USE PLANNING.

1.2.2 Planning Review Policies

There are four levels of drainage planning. The initial plan is the Drainage Basin Planning Study which is governed by broad general framework goals. Next is a Master Development Drainage Plan which provides greater hydrologic and hydraulic detail as well as interfacent requirements of the Drainage Basin Planning Study. A Master Development Drainage Plan is typically the first drainage submittal required for a comprehensive development plan. A Drainage Basin Planning Study may be required as per the City or County regulations.

In addition to a Master Development Drainage Plan, each new development must provide Preliminary and Final Drainage Reports. Each of these levels of drainage planning is defined as follows:

Drainage Basin Planning Study - A Drainage Basin Planning Study (DBPS) is an engineering and planning study of a drainage basin which is tributary to a major receiving stream. Within El Paso County there are 13 major basins, 23 receiving streams and 139 designated drainage basins. Within the City of Colorado Springs, there are 23 designated drainage basins, these are cross referenced to the County basins in a report prepared in May, 1986 and revised in January 1989 entitled, Drainage Basin Identification and Fee Estimation Report by Muller Engineering. Key tables which summarize the drainage basins have been included in Chapter 3 of this manual.

A DBPS shall show the conduits, channels, natural drainage courses, detention reservoirs, easements, culverts and all other hydraulic facilities required to control surface waters from the 100-year event within said basin and to carry such waters to points of insignificant impact. Subbasins shall be delineated. The study shall include an estimate of the cost of said drainage facilities, the computation of such cost may include the expense of the study. The drainage fee is intended to represent the equitable share of the cost of drainage facilities to be constructed and is expressed in dollars per acre. The estimated cost per acre of providing said facilities shall generally be determined within each drainage basin by dividing the number of developable acres within the basin into the total cost of improvements. City/County participation in funding shall be determined by the City Council and County Commissioners and shall consider recommendations by the Storm Drainage Board. This per

acre cost shall be known as the unit drainage fee. This unit drainage fee concept is described in the Land Development Code for the County and the Subdivision Policy Manual for the City of Colorado Springs. The current Drainage Board Policy should be consulted when establishing the drainage fee. The unit drainage fee is discussed in detail in Chapter 3.

Master Development Drainage Plan - A Master Development Drainage Plan (MDDP) will be prepared for developments within any drainage basin. The purpose of the MDDP is to complete drainage planning for the proposed development before embarking on individual phases. Site requirements including public improvements for the development will be identified in the MDDP. The MDDP must identify hydrology and hydraulics of existing/proposed drainageways and appurtenant structures that are within or effect a proposed development. This hydrology should be compatible with the DBPS. Changes proposed by the MDDP, must be analyzed within the context of the DBPS to determine impacts on downstream users. The MDDP must describe the general treatment of drainageways, including safety and maintenance. The plan outlines the protection of public facilities, such as bridge crossings and utilities, and the protection of private property adjacent to the banks of the drainageways. Right-of-ways requirements must be delineated on the plans for all drainageways, storage facilities and other drainage structures.

A Master Development Drainage Plan shall be required for any phased development greater than 10 acres prior to approval for land annexation, or preliminary plat/plan.

Where a development is not phased, the report requirements for a Master Development Drainage Plan and the preliminary drainage report may be combined.

Preliminary Drainage Report - The purpose of a Preliminary Drainage Report (PDR) is to identify and propose specific solutions to drainage problems that would occur as a result of that portion of the development considered for plating. Detailed analysis of drainage basin hydrology and hydraulics is required. Alternative solutions to drainage problems shall be noted and the capacity of drainage facilities on and off-site shall be evaluated. Specific improvements, including open channels, storm sewers, grading, site stabilization, catch basins, culverts and other improvements will be located and sized to meet requirements of the initial and major drainage system. (See submittal requirements in Chapter 4).

Final Drainage Report - The purpose of a Final Drainage Report (FDR) is to analyze all drainage features of the proposed platted development in the detail necessary to complete construction plans for all project drainage structures, including grading, streets, and off-site facilities, as applicable.

THE CITY OF COLORADO SPRINGS AND EL PASO COUNTY (CITY/ COUNTY) DRAINAGE PLANNING PROCESS REQUIRE DESIGNERS OF STORM WATER DRAINAGE FACILITIES WITHIN THE RESPECTIVE JURISDICTIONAL BOUNDARIES TO FOLLOW THE PLANNING AND DESIGN GUIDELINES PRESENTED HEREIN. THE RESPECTIVE AND RELEVANT DETAILS OF DRAINAGE INVESTIGATION(S) WILL BE DEPENDENT UPON THE LEVEL OF STUDY REQUIRED i.e., DRAINAGE BASIN PLANNING STUDY, MASTER DEVELOPMENT DRAINAGE PLAN, PRELIMINARY DRAINAGE REPORT OR FINAL DRAINAGE REPORT.

1.2.3 Drainage Systems

Every area has two separate and distinct drainage systems. One is the initial system and the other is the major system. To provide for orderly growth, reduce future costs, and prevent loss of life and major property damage, both systems must be planned and properly designed.

Initial Storm Provisions - The initial storm drainage system (10 year storm) is necessary to reduce maintenance costs, to provide protection against frequently reoccurring storms, to implement an orderly system, and to provide convenience to the residents.

Major Storm Provisions - In addition to providing the storm drainage facilities for the initial storm runoff, provisions shall be taken to prevent major property damage and loss of life from the 100-year storm runoff. These provisions shall be known as the major drainage system. The impact of the 100-year storm shall be investigated and adequate 100- year facilities shall be provided.

THE CITY/COUNTY DRAINAGE POLICY REQUIRES ALL DEVELOPMENT TO INCLUDE PLANNING, DESIGN, AND CONSTRUCTION OF INITIAL AND MAJOR STORM DRAINAGE SYSTEMS IN ACCORDANCE WITH THE CRITERIA PRESENTED HEREIN.

1.2.4 Major Drainageways

The City/County considers the following streams as Major Drainageways, and these have 100-year flood plain designations along selected reaches:

Bear Creek	Beaver Creek
Becker's Lake Trib.	Big Springs Creek
Black Squirrel Creek	Black Forest
Brackett Creek	Camp Creek
Cheyenne Run (southwest area)	Chico Creek
Cheyenne Creek (southwest area)	Crystal Creek
Cottonwood Creek	Crews Gulch (Big Johnson)
Dry Creek	Douglas Creek
Hay Creek	Fisher's Canyon
Jackson Creek	Fountain Creek
Jimmy Camp Creek	Horse, N. Fk. Horse, Little Horse Creek

Kiowa Creek	Kettle Creek
Little Fountain Creek	Mesa Basin Creek
Monument Branch	Monument Creek
Mustang Creek	Pine Creek, N. Pine Creek
Peterson Field Basin	Pond Creek
Raxton Creek	Rock Creek
North Rockrimmon Basin	South Rockrimmon Basin
Security Creek (Little Johnson)	Sand Creek, E. Fork Sand Ck.
Sand Creek, E. Fork Spring	Sand Creek, (Main Branch)
Sand Creek, Center Tributary	Shook's Run
Spring Run	Smith Creek
Steel Fork Creek	Sutherland Creek
Templeton Gap & Tributaries	Turkey Canyon
L. Turkey Canyon	Unnamed Tributaries
Williams Canyon Creek	Windmill Gulch
Others Easterly	

These streams serve as outfall channels for basin storm water drainage systems.

THE CITY/COUNTY DRAINAGE POLICY REQUIRES MANAGEMENT AND REGULATION OF FLOOD PLAINS ALONG ALL REACHES OF ALL DRAINAGEWAYS. IMPROVEMENTS TO MAJOR DRAINAGEWAYS WITHIN DEVELOPMENT AREAS WILL BE PLANNED AND DESIGNED IN ACCORDANCE WITH THE CRITERIA PRESENTED HEREIN. DEVELOPMENT ALONG THE MAJOR DRAINAGEWAYS WILL BE IN CONFORMANCE WITH FEDERAL AND STATE LAW, CURRENT FLOOD PLAIN ORDINANCES, RESOLUTIONS AND REGULATIONS.

1.2.5 Multipurpose Use

The many competing demands placed upon water within a region suggest that a strategy for managing drainage be multi-purpose. In addition, facilities not designed primarily for drainage frequently can be designed to provide drainage benefits. Another aspect of a drainage strategy is that it may consider multiple means of accomplishing its objective. In general, there is not one single, all-encompassing method of handling storm water runoff. What is required is a combination of conveyance and storage, as well as system types. When a multipurpose strategy is adopted it may involve participation by both public and private interests.

A DRAINAGE STRATEGY SHOULD BE A MULTIPURPOSE, MULTIPLE MEANS EFFORT.

1.2.6 Basin Transfer

Colorado drainage law recognizes the inequity of transferring the burden on managing storm drainage from one location or property to another. Liability questions also arise when the historic drainage continuum is altered. The diversion of storm runoff from one basin to another should be avoided unless specific and prudent reasons justify and dictate such a transfer.

The subdivision development process can significantly alter historic or natural drainage paths. When these alterations result in a subdivision outfall system that discharges back into the natural drainageway at or near the historic location, then the alterations (intra-basin transfer) are generally acceptable. However, when the subdivision outfall system does not return to the historic drainageway, then inter-basin transfer results. Inter-basin transfer must be prevented since it violates a basic civil drainage law principle by discharging water onto a property in a manner or quantity which may do more harm than formerly. The right to engage in these practices will be subject to the legal requirements as stated in state statutes and case law.

THE POLICY OF THE CITY/COUNTY SHALL BE TO AVOID INTER-BASIN TRANSFER OF STORM DRAINAGE RUNOFF AND TO MAINTAIN HISTORIC DRAINAGE PATHS. HOWEVER, THE TRANSFER OF DRAINAGE FROM BASIN TO BASIN IS A VIABLE ALTERNATIVE IN CERTAIN INSTANCES AND WILL BE REVIEWED ON A CASE-BY-CASE BASIS.

1.3 Financial Responsibility

A developer is generally required to pay for the drainage facilities serving his subdivision.

The City/County have established a drainage board to monitor the Storm Drainage Basin Fund. The Storm Drainage Basin Fund, in part, pays for drainage improvements based on collection of a basin fee from developers within a given basin. (See Chapter 3)

THE CITY/COUNTY DRAINAGE POLICY REQUIRES THAT DEVELOPERS PAY FOR THEIR STORM WATER DRAINAGE IMPROVEMENTS. WHERE PUBLIC IMPROVEMENTS ARE NECESSARY, FUNDING MECHANISMS WILL BE DEVELOPED ON A CASE-BY-CASE BASIS AND IN GENERAL CONFORMANCE WITH FEDERAL, STATE AND LOCAL LAW, LOCAL POLICY, ORDINANCES, RESOLUTIONS AND/OR REGULATIONS.

1.4 Development Near Channels, Irrigation Ditches and Drainageways

1.4.1 Drainage and Development Near Irrigation Ditches

Irrigation canals often capture storm water from upstream areas. Development tends to increase the volume and the flow rate of storm runoff. In addition, the concentration patterns of runoff may be changed. The net effect of development is generally an increase in peak flow rate received downstream.

Changing historical routing patterns from irrigation canals could create severe liability problems. However, the use of irrigation canals for storm water conveyance is not acceptable since the capacity of these canals generally does not increase downstream, and the slopes, which tend to be almost flat, lessen stormwater transport capabilities.

Since ditch owners can be held liable in case of ditch failure during intense storm events, acceptance of increased runoff may aggravate this problem.

Whenever development will alter patterns of drainage into irrigation ditches by increasing flow rates or volumes, or will change the historic concentration points of runoff the developer shall obtain written consent from canal owners to do so.

Runoff from development shall be directed into historic and natural drainageways, avoiding discharge into irrigation canals, except as required by water rights or where no reasonable alternative exists.

Irrigation canals often intercept natural drainageways that are part of the drainage system. Irrigation facilities may suffer substantial damage in the event of a major storm.

Wherever irrigation canals intercept major drainage channels the developer shall separate developed peak storm runoff flows from historic and canal flows whenever such separation is required by the canal owner and cannot be considered historical. The right to engage in these practices will be subject to the legal requirements as stated in state statutes and case law.

WHEN DEVELOPMENT ENCROACHES UPON EXISTING IRRIGATION FACILITIES, THE CITY/COUNTY SHALL REQUIRE DEVELOPERS TO MINIMIZE THE ADVERSE EFFECT ON THE IRRIGATION FACILITIES, AND COORDINATE WITH THE IRRIGATION FACILITY OWNERS.

1.4.2 Development Along Natural Channel Systems/Major Drainageways

Natural channel systems, primarily the designated Major Drainageways and Primary outfalls, serve to store flood waters, enhance water quality, provide for ground water recharge and preserve riparian corridors. The use of historical channels to convey storm water runoff from developed and developing areas is acceptable. However, if historical storm water flows are increased, or if historical channels are unstable in their natural condition, these channels must be adequately stabilized to prevent excessive erosion. Making necessary improvements may be subject to Section 404 of Public Law 92-500, administered by the U.S. Army Corps of Engineers. The right to engage in these practices will be subject to the legal requirements as stated in state statues and case law.

Within the City of Colorado Springs, drainageways are part of a Comprehensive Plan which preserves and enhances drainageways as amenities to the City. Refer to Section 9-1 of the Comprehensive Plan.

THE CITY/COUNTY DRAINAGE POLICY REQUIRES DEVELOPMENT TO BE A MINIMUM OF 1-FOOT ABOVE THE ESTIMATED 100-YEAR FLOOD WATER SURFACE ELEVATION OF MAJOR DRAINAGEWAYS, CHANNEL OUTFALLS, UPSTREAM SITES OF IRRIGATION DITCHES, DAM AND RESERVOIR SITES. ADDITIONALLY, DEVELOPERS SHALL PROVIDE SUFFICIENT RIGHT-OF-WAY FOR MAINTENANCE OF DRAINAGEWAYS OR RESERVOIRS AS REQUIRED.

DEVELOPERS IN AND ALONG A DRAINAGEWAY ARE REQUIRED TO IMPLEMENT THE PROPER MEASURES TO MAINTAIN OR CREATE STABLE CHARACTERISTICS OF THE DRAINAGEWAY. THE PRINCIPAL OBJECTIVE IS TO LIMIT EXCESSIVE EROSION IN AND ALONG THE CHANNEL. HISTORICAL CHANNEL RELOCATIONS/REALIGNMENTS SHALL NOT BE ALLOWED UNLESS ENGINEERING DESIGNS FOR STABLE SYSTEMS UNDER FLOOD FLOW CONDITIONS ARE ACHIEVED AND APPROVED.

1.5 Storm Water Runoff Detention

Detention storage of storm water runoff may be necessary in certain drainage basins to attenuate peak flood flows. Regional detention ponds are required in place of numerous smaller detention ponds. Temporary, smaller detention ponds will only be allowed on a case by case basis. Drainage Basin Planning Studies should identify the feasibility of regional detention facilities. These regional structures identified in the DBPS will be subject to reimbursement from the Drainage Basin Fund.

THE CITY/COUNTY DRAINAGE POLICY PERMITS THE USE OF DETENTION STORAGE OF STORM WATER RUNOFF WHEN COMPATIBLE WITH DRAINAGE BASIN STUDIES AND/OR OTHER APPROVED STUDIES. REGIONAL DETENTION STORAGE FACILITIES SHALL BE UTILIZED WHERE NECESSARY AND APPROVED TO AFFORD PUBLIC SAFETY, PROVIDE FOR ECONOMIC DEVELOPMENT OF BASIN DRAINAGE SYSTEMS OR TO PROTECT DOWNSTREAM DEVELOPMENTS FROM FLOOD DAMAGE.

1.6 Quality of Storm Water Runoff

Water pollution from developed areas commonly involves concentrations of trace metals, organic materials, nutrients, salts, and sediment. Sediment production from developed areas, however is usually not a major problem; the greatest sources of sediment from developed areas include erosion from new construction and erosion of unstable channels. Sediment production in unstable channel reaches can increase in areas where development causes local increases in storm runoff rates. Detention storage ponds can decrease the sediment load and debris in storm water runoff.

The Environmental Protection Agency (EPA) controls water pollution by the administration of provisions set forth in Section 208 of the Federal Water Pollution Control Act Amendments of 1972 (PL 92-500). Basin-wide "Section 208" plans prepared for the City/County areas in accordance with EPA guidelines, help to:

- establish policies, programs and appropriate regulatory guidelines to limit non-point pollution to ground and surface waters resulting from development, and

- abate existing pollutant loadings to the best extent possible, and

- integrate water quality considerations in the management of storm water runoff and control.

THE CITY/COUNTY DRAINAGE POLICY SHALL REQUIRE PLANNING FOR WATER QUALITY MANAGEMENT OF STORM WATER RUNOFF FROM DEVELOPMENTS. THE MANAGEMENT OF OVERALL WATER QUALITY OF STORM DRAINAGE WILL REQUIRE DEVELOPERS TO SUBMIT AN EROSION CONTROL PLAN FOR ALL DEVELOPMENT IN ACCORDANCE WITH CRITERIA SET FORTH IN THIS MANUAL.

1.7 Operations and Maintenance

Operation and maintenance of storm water drainage facilities is required to ensure that they will perform as designed. Channel bed and bank erosion, drop structure, pipe inlet and outlet, and overall deterioration to he facilities must be routinely inspected and repaired to avoid reduced conveyance capacity, displeasing aesthetics and ultimate failure.

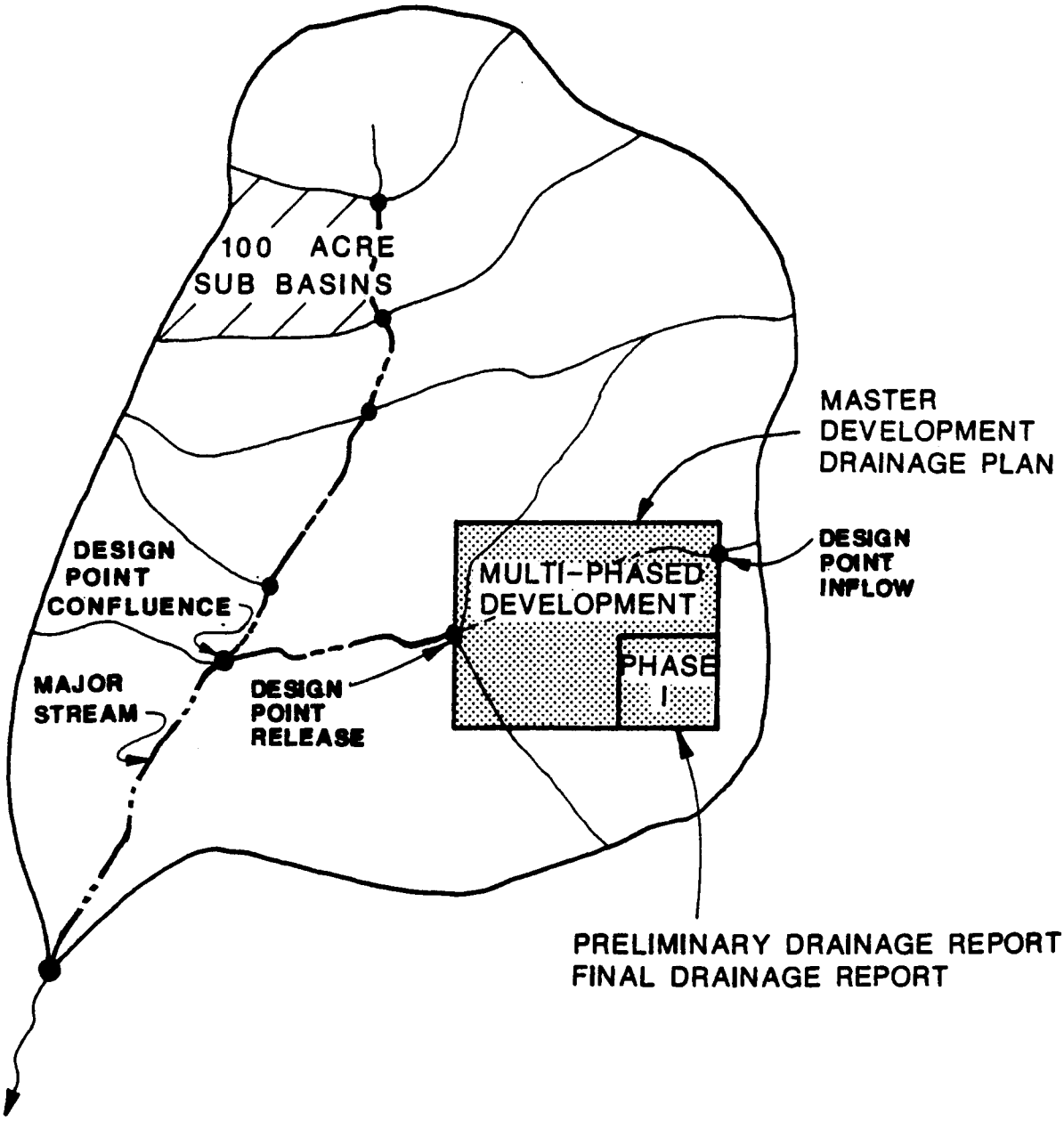
Sediment and debris must also be periodically removed from channels, storm sewers and detention basins. Trashracks and inlets must also be routinely cleared of debris to maintain system capacity.

THE CITY/COUNTY DRAINAGE POLICY REQUIRES THAT ACCESS BE PROVIDED TO ALL STORM WATER DRAINAGE FACILITIES FOR MAINTENANCE AND INSPECTION. DEVELOPERS SHALL MAINTAIN DRAINAGE FACILITIES DURING THE WARRANTY PERIOD.

THE DEVELOPER SHALL PROVIDE FOR PERPETUAL MAINTENANCE OF PRIVATE DRAINAGE FACILITIES THROUGH MECHANISMS ACCEPTABLE TO THE CITY/ COUNTY.

THE CITY/COUNTY WILL PROVIDE FUNDING OF PERPETUAL MAINTENANCE OF PUBLIC DRAINAGE FACILITIES AFTER THE WARRANTY PERIOD PROVIDED THAT WRITTEN FINAL ACCEPTANCE OF THE FACILITIES HAS BEEN GRANTED.

DRAINAGE BASIN PLANNING STUDY
NO-NAME BASIN NO. FO-FO-08-00



HDR Infrastructure, Inc.
A Centerra Company

The City of Colorado Springs / El Paso County
Drainage Criteria Manual

Drainage Report

Date
OCT. 1987

Figure
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SECTION I

Chapter 2 Drainage Criteria

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Chapter 2 DRAINAGE CRITERIA

2.1 Design Storm Water Runoff Determination

The City/County drainage policy permits the Rational Method and the Soil Conservation Service (SCS) Hydrograph Method as models to be used by designers in estimating storm water runoff for project purposes. The Rational Method is required for drainage basins of 100 acres or less. The SCS Hydrograph Method is required for drainage basins greater than 100 acres. For large complex Drainage Basin Planning Studies, computer models may be utilized following approval by City/County. Necessary data requirements, assumptions and detailed procedures for using models are discussed Section II. All drainage systems must be planned, designed and constructed to handle runoff from both the initial and major design storms. The initial design storm shall be the 10 year event. The major design storm shall be the 100-year event.

Normally in a roadway section, the initial drainage system should convey a substantial portion of the minor storm flows with the more intense or major storm flows conveyed by the available street capacity. Out of roadway sections, storm drain systems combined with overflow swales or drainageway or channel sections must be of adequate capacity to convey both design storms and protect adjacent properties.

2.2 Open Channels and Roadside Ditches

2.2.1 Channelization

Historic routing of drainageways is required where possible. Major consideration shall be given to flood plain and open space requirements in the area.

A stable natural channel reaches "equilibrium" over may years. Therefore, channel modifications should be minimal. However, in areas of the City/County jurisdictions many historical channels are in a dynamic unstable condition and will require stabilization. When a channel improvement is necessary, the historic route should be maintained if possible. Good channel design includes consideration of:

- Carrying capacity, including freeboard
- Alignment
- Erosion resistance
- Aesthetics

The hydraulic properties of natural channels are generally irregular. A comprehensive study of flow in natural channels requires consideration of sediment transportation, river morphology, etc. The field constitutes a subject known as river

mechanics. The hydraulic properties of artificial channels can be controlled to the extent desired or designed to meet given requirements. The choice of and design of open channels requires good hydraulic practice, environmental and sociological consideration, and project requirements.

When historic drainageway receives increased flows the erosion process may accelerate causing slope stability problems. Channel and bank erosion may encroach upon and undermine existing structures. Historic drainageways must be adequately stabilized to prevent bank and channel erosion.

The investigations necessary to insure that a natural channel will be adequate are different for every waterway. Generally, the designer should prepare cross sections of the channel for the major design runoff, investigate the bed and bank material to determine the particle size classification, and study slope stability of the channel under future conditions of flow.

The extent of stabilization will depend on channel slope, soil conditions and hydraulic conveyance requirements. When possible, soft channel linings should be utilized in conjunction with grade stabilization structures to retain a natural channel. Hard channel linings may be required in steep areas or in erosive soils. Historic drainageways should be utilized wherever possible. Chapter 10 contains greater detail on channel design criteria and specifications.

2.2.2 Channel Storage

Drainageways can provide significant storage capacity. This storage may be beneficial to the downstream community by reducing downstream peak runoff rates and providing the opportunity for ground water recharge. Also, wide, natural channels can provide urban open space.

2.3 Street Flows

Streets can be an integral part of the drainage system and can be used for transporting storm runoff within reasonable limits. However, the engineer should recognize that the primary purpose of streets is for traffic, and therefore reasonable limits for the use of streets for storm runoff must be followed. The City/County policy is to permit the use of streets for the initial drainage system within the limitations noted in Chapter 6.

2.3.1 Bridges

A bridge over a drainageway shall be defined as a Roadway Structure having a passageway for carrying traffic or other moving loads and having a drainageway clear opening measured along the centerline of the Roadway of more than 20 feet between abutments.

Bridge "width" refers to the dimension of the bridge measured along the drainageway. Bridge "length" is the measurement along the roadway.

2.4 Culvert Systems

The function of a drainage culvert is to pass the design storm flow under a roadway, railroad or other embankment section without causing excessive backwater and creating excessive downstream flow velocities. Flow velocities are considered excessive when erosion or safety of downstream structures and embankments are endangered. Consideration must be given to the effect of high velocities, local eddy currents, or other turbulence on the channel, surrounding property and roadway embankment. Flow velocities greater than 2.5 ft/sec in culverts must be maintained in all drainage structures to limit siltation. Where local silting or scour is determined to be a threat, protection shall be installed to ensue that damage to or failure of the structure will not occur.

The hydraulic design of any culvert may be affected by its cross-sectional area, shape, entrance geometry, length, slope, construction material, and depth of ponding at the inlet (headwater) and outlet (tailwater). Culvert flows are classified as having either inlet or outlet control. Inlet control means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater and the entrance geometry, including the barrel shape and cross-sectional area, and the type of inlet edge. In inlet control, the roughness and length of the culvert barrel and outlet conditions, including depth of tailwater, are not factors in determining culvert capacity. Outlet control means that the discharge capacity of a culvert is controlled at the culvert exit. This usually results from a downstream flow constriction causing the tailwater to back up into the barrel, thus reducing the flow.

The general criteria for culvert design shall be:

1. Culvert inlets shall be designed for entrance and friction losses.

2. Outlets shall be designed to avoid potential sedimentation and erosion of the downstream channel. Outlet erosion control devices and energy dissipators may be required where excessively high discharge velocities occur.

3. The minimum size <u>culvert</u> shall be 18 inches in diameter or its hydraulic equivalent. The use of culverts less than 18" in diameter for driveways must have the approval of the governing jurisdiction.

4. All culverts shall be designed for H-20 loading with the appropriate embankment considerations.

5. Culverts, will be designed under allowable headwater and tailwater conditions as noted in 6.4.1.

6. Culverts designed on roadways with State jurisdiction shall conform to the Colorado Department of Highway criteria.

In areas where a Federal Emergency Management Agency (FEMA) designated floodway exists, increases in the 100-year water-surface elevation are not allowed above the established 100-year maximum water surface elevation. Once the FEMA 100-year floodway limits have been established, no further infringement will be allowed.

2.5 Detention Storage of Storm Water Runoff

2.5.1 On-site and Off-site Regional Detention Basins

Regional detention basins or other structures can be used to maintain historical runoff flows. Care must be taken to design and construct all such facilities in accordance with applicable federal, state and local regulations. Detention basins may be required to reduce peak flow resulting from a proposed development. The drainage master planning process shall identify the alternatives of improvement to downstream facilities or providing for detention. The initial burden of cost for either alternative shall be born by the developer. The costs of constructing a storm water detention basin, including the safety features, will be eligible for reimbursement in accordance with the provisions for reimbursement or credits contained in 15-3-907 of the Code of Colorado Springs, 1980, as amended or Section 49.3D of the Land Development Code of El Paso County, 1985, as amended.

2.5.2 Open-channel Storage

Open-channel storage is addressed in Section 2.2.2.

2.5.3 Volume and Release Requirements

Storm water detention ponds shall be sized by the Flood-Routing Procedure as outlined in Chapter 11. The design storms to be considered for computation of storage volume requirements for detention ponds shall be the 2-hour and 24-hour, 10-year and 100-year design storms as shown in Chapter 5.

The storm water detention pond emergency spillway shall be designed to pass the maximum peak discharge for a 100-year storm without damage to the structure, assuming the outlet structure is totally inoperative and the upstream watershed is fully developed. No buildings or structures shall be constructed within the limits of the 100-year flood plain. All storm water detention basins which fall under the jurisdiction of the State, shall be designed using the State's criteria.

Detention storage areas shall have designed volume requirements compatible with release rates that will not exceed the historic runoff rate or capacity of downstream facilities whichever would govern or as per the Drainage Basin Planning Study. The design of all impoundments which meet the specifications of a jurisdictional dam, whether temporary or permanent, must be submitted to the State Engineer for review and approval.

2.5.4 Maintenance Needs

Public detention basins, designated as such on the Drainage Basin Planning Study and the Subdivision Drainage Studies, shall be dedicated to the City/County for maintenance following the warranty period. Maintenance of private detention ponds shall remain the responsibility of the property owner(s).

The City/County requires the right and authority to enter upon private property and make necessary corrections or perform necessary maintenance to every private detention facility which is a part of the drainage plan should the facility constitute a hazard to persons or downstream properties. Detention basins shall be constructed off-stream of the major drainageways unless otherwise depicted in the Drainage Basin Planning Study. The cost of such maintenance on private detention basins shall be the responsibility of the property owner(s). A perpetual private maintenance agreement between the City/County and the property owner(s) shall be recorded at the County Recorder's Office for all private detention facilities.

2.6 Erosion and Sediment Control

The need for erosion and sediment control facilities; either permanent or temporary, shall be determined according to the standards for sediment and erosion control (referenced in Chapter 4 of this manual) in developing areas, as required by the City/County.

An Erosion Control Plan is required of property owners/developers prior to commencing any grading and where platting occurs. A plan is required at the time of final drainage report submittal. The plan will be approved concurrently with the report unless otherwise waived by the City/County. Temporary erosion and sedimentation control facilities shall be constructed prior to any grading or extensive land clearing, in accordance with the erosion control plan. These facilities must be satisfactorily maintained until construction and landscaping are completed and the potential for on-site erosion has been significantly reduced.

2.7 Flood Plain Standards

In Colorado, cities and counties have been given plenary grants of power by the Legislature to regulate land use in the flood plain.

The governing body of each municipality has the authority:

> "... to establish, regulate, restrict and limit such uses on or along any storm or floodwater runoff channel or basin, as such storm or floodwater runoff channel or basin has been designated and approved by the Colorado Water Conservation Board, in order to lessen or avoid the hazards to persons and damage to property resulting from the accumulation of storm or floodwaters." C.R.S. 31-23-301(1).

Counties in Colorado are directly authorized by statute to adopt zoning plans concerned with regulating use in a flood plain area through the provisions of C.R.S. 30-28-111(1):

> "... the county planning commission may include in said zoning plan or plans provisions establishing, regulating, and limiting such uses upon or along any storm or water runoff channel or basin as such storm or runoff channel or basin has been designated and approved by the Colorado Water Conservation Board in order to lessen or avoid the hazards to persons and damage to property resulting from the accumulation of storm or flood waters."

The City/County has adopted flood plain regulations as part of the "Zoning Regulations" set forth in Section 32, Part IV of the "El Paso County Land Development Code" and Chapter 15, Article 6 of the Colorado Springs City Code. The City/County flood plain policy is noted in Section 1.4.2. The adopted City/County flood plain mapping standards include those formulated by FEMA and identified as part of the zoning regulations for the Flood Plain District. The Flood Insurance Studies completed by FEMA for the City of Colorado Springs and for El Paso County were designated and approved by the Colorado Water Conservation Board on November 18, 1986.

The position of a Flood Plain Administrator has been established by the City/County with the following responsibilities:

- Permit review (Drainage Plan review)
- Use of base flood data other than that included in published FEMA maps
- Interpretations of base flood elevation boundaries
- Maintain file of all FEMA maps and all lowest floor elevations for new or substantially improved structures
- Monitor alteration of watercourses
- Administer FEMA and local City/County guidelines or regulations

The developer shall consult with the City/County and Flood Plain Administrator concerning current FEMA mapping, map revisions and amendments in conjunction with the planning process.

2.8 Flood Proofing Evaluation

Flood proofing is defined by FEMA as any combination of structural and nonstructrual additions, changes or adjustments to structures which would reduce or eliminate flood damage to real estate or improved real property, sanitary facilities, structures, and their contents. FEMA has published several references to provide detailed criteria and design procedures for flood proofing structures. These references are listed in the bibliography and should be consulted for detailed design information. The more commonly used flood proofing measures are discussed in Section III.

Any proposed structures located within the 100-year flood fringe shall either have the lowest floor elevated one (1) foot above the 100-year floodway water surface elevation or be flood proofed (non-residential) to one foot above the floodway water surface elevation. Exceptions shall be reviewed by the City/County Flood Plain Administrator. The 100-year flood plain and corresponding elevations of the 100-year floodway can be determined by reviewing the Flood Insurance Rate Maps (FIRM) published by FEMA and available for review in the City/County offices as well as the flood plain administrator's office.

2.9 Maintenance

The City/County may assume the maintenance of public surface and subsurface drainage systems and erosion control structures after a probationary two-year maintenance period by the developer if the following are met:

a. All of the requirements of The Drainage Criteria Manual have been met.

b. The City/County have completed a probationary inspection of the facilities. Probationary acceptance of the facilities is required before the developer may apply for reimbursement or credit of costs for drainage facilities.

c. Probationary acceptance by the City/County shall initiate a two year maintenance period during which the developer is totally responsible for maintenance of all drainage facilities. The developer shall post the two year maintenance financial assurance with the City/County.

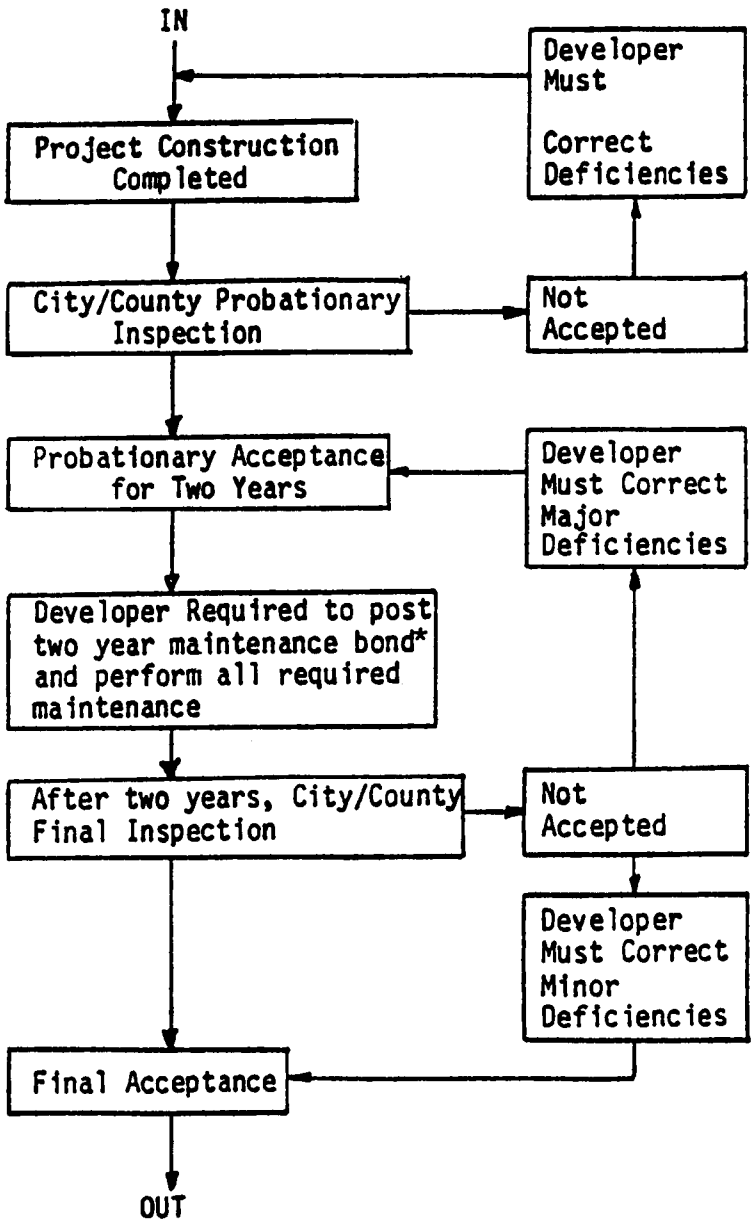
d. A final inspection at the end of the two year probationary maintenance period shall be conducted by the City/County. Following final acceptance by the City/County, maintenance responsibilities will transfer to the City/County.

e. All necessary easements and/or ROW, entitling the City/County to properly maintain the facility, have been conveyed to the City/County.

f. The developer has supplied to the City/County an accounting of capital construction for the drainage facilities through the end of the two-year maintenance period.

In the event that the City/County elects not to assume the operation and maintenance responsibility for the facilities, it will be the responsibility of the developer to make arrangements with the occupants or owners of the subject property for assumption of operation and maintenance, subject to the approval of the City/County. Such arrangements shall be completed and approved prior to the end of the two-year period of developer responsibility. The City/County may inspect the facilities in order to ensure proper functioning of the facilities for the purposes for which they were built. A flow chart of the maintenance acceptance process is shown in Figure 2.1. A private maintenance agreement may be required by the City/County and the agreement shall be recorded against the land.

MAINTENANCE ACCEPTANCE PROCEDURE



* In lieu of a bond, other financial assurances will also be acceptable by the City/County

SECTION I

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CHAPTER 3 DRAINAGE BASIN FEES, PROCEDURES AND LAYOUT

3.1 Drainage Basin Fee Assessment

El Paso County, inclusive of the Colorado Springs Metropolitan area, has been divided into 13 major basins. These basins have been further divided into 139 smaller drainage basins. The drainage basin divides constitute jurisdictional boundaries for the purpose of assessing drainage improvement fees. These fees are assessed on a per acre basis. The City of Colorado Springs and El Paso County entered into an intergovernmental agreement, dated November 22, 1983 to establish a joint storm drainage board. The Subdivision Storm Drainage Board consists of seven members who are appointed by City Council and the Board of County Commissioners. The County Director of Public Works and City Director of Public Works are non-voting secretaries of the Board. The Board recommends, to the City Council and the Board of County Commissioners, regarding drainage basin fees, the investment of surplus funds that may temporarily accumulate, and the repayment or credit for facilities installed from the drainage basin funds.

The Drainage Basin Fees represent the equitable share of the cost of drainage improvements to be constructed and is expressed in dollars per acre of unplatted land in the drainage basin. The drainage basin fee is the total drainage improvement cost divided by the unplatted acreage. The current Drainage Board Policy should be consulted when establishing the drainage fee.

The Drainage Basin Fees are a one time charge to the developer based on the number of acres platted and is collected at the time of the final plat recording. The developer may provide drainage improvements which are creditable toward drainage basin fees. The required fee is that in effect at the time of the final plat approval and/or bid opening.

The Drainage and Bridge Fees are subject to annual revision based on the current construction costs index. The Drainage and Bridge Fee Schedule is available for reference at the appropriate City/County office.

Drainage basin fees for residential subdivisions with one dwelling unit per lot having lot sizes of 1.0 acre or greater shall be assessed only for the first acre of each lot (County only). More detailed discussion of drainage funding requirements is contained in Section 49.3(D) of the El Paso County Land Development Code and Chapter 15, Article 3 of the Colorado Springs zone regulations code.

3.2 Procedures for Submittals to the Drainage Board

1. The deadline for submittals to the Drainage Board shall be 4:00 P.M. on the third Thursday of the month prior to the <u>next</u> month's Drainage Board Meeting.

2. All petitioners for Drainage Board items will be notified by the staff prior to the meeting. They will be informed of the staff's recommendation concerning their item(s) and whether their item is included on that particular agenda.

3. All petitioners will be provided with the Board's decision concerning their agenda item and advised of their appeal right following each Board meeting.

4. All administrative items shall be directed to the staff prior to scheduling for the Board.

3.3 Procedures for Drainage Improvement Credits and Reimbursements

In order for a developer to obtain reimbursement or credit for drainage costs, certain procedures must be followed. Prior to construction, the developer must:

1. Obtain through the subdivision process, an approved drainage report showing the facilities required and the estimated cost of those facilities, and indicate the drainage basin and fee for the acreage shown on the drainage plan. (Copies of the current per-acre fee can be obtained from the Public Works Department.) All drainage basins are identified and fees established each year. Letters of credit are to be posted with the City/County in accordance with the Subdivision Ordinance or Regulations.

2. Obtain three (3) sealed bids for constructing the facilities. If the lowest bid exceeds the Engineer's estimated costs as indicated in the drainage report, the City Engineer must review and give approval (or disapproval) before proceeding.

3. If it is not possible to receive three bids, the developer must obtain the City/County Engineer's approval before proceeding with construction, and the cost and credits shall be determined at that time.

4. The developer proceeds with construction according to the approved plans and any changes during construction shall be discussed with the City/County Engineer.

5. Upon completion of the construction of facilities, the developer shall obtain a certification from a Colorado Registered Professional Engineer that the facilities inspected are constructed in accordance with the approved plans. A written request for the City/County inspection of facilities must be submitted to the City/County Engineer. The assurance for facilities may be released after inspection by the City/County.

6. The developer then submits this certification along with records of the cost of construction, i.e., a copy of the three bids (including the low bid), as-paid bills, a copy of the City/County's inspection and cover letter of request for cash reimbursement or credits. This request must be submitted by the third Thursday of the month to be considered at the next month's Drainage Board Meeting.

7. The City/County Engineer will review the request, and after verification of the costs, will place the request on the Drainage Board Agenda for formal review.

8. The Drainage Board meets on the third Thursday of each month and reviews the requests for reimbursement after hearing from both the City/County and the Developer.

9. If the request is approved, the item is then forwarded to City Council/County Commissioners for final approval of reimbursement after hearing from the City/County and the developer.

10. If the request is not approved, the Drainage Board will set the amount of reimbursement and the developer has the right to appeal to City Council or the County Commissioners.

11. Reimbursements are made twice a year: on the first of June and the first of December. Items to be heard for June disbursement must be approved by the Drainage Board by April 1. Items to be heard for December disbursement must be approved by the Drainage Board no later than October 1.

12. If the Drainage Basin account has insufficient funds for reimbursement, the developer is placed on a priority list and is paid when revenue within the fund is sufficient. Partial reimbursements are made until the developer is fully reimbursed.

13. If the proposed estimated facilities are less than the fees, the difference will be a cash payment by the developer if he has no available credits in the basin. The assurances will not be released until the final paid bills have been submitted to Land Development to verify costs. If any additional fees are due, they will be payable prior to release of assurances.

14. If an overpayment of fees has occurred, the request for cash reimbursement must be submitted to the City/County Engineering Division for approval. Upon approval, the request shall be placed on first priority for immediate refund dependent upon funds available in the Basin Account.

15. If the proposed estimated facilities <u>are equal to the fees</u>, assurances will not be released until the final paid bills have been submitted to verify costs. If any fees are due, they will be payable prior to release of assurances.

16. Basin funds are not transferable among basins.

17. Any dispute over bidding or fees may be heard by the Drainage Board and City Council or County Commissioners.

18. The developer should be aware that all drainage basin fees are finally computed to the time that bids are taken and not at the time of platting.

19. Fees on all basins are adjusted by City/County each year at their first meeting of the year and at other intervals as needed.

For the purpose of arterial bridge reimbursement from the City Bridge Fund (City Code 15-3-1003), the bridge width shall be measured perpendicular to the ROW and the bridge width will be limited to the ROW dimension. (See definition Section 2.3.1) Box culverts, arch pipe culverts, and similar structures will be considered drainage facilities and included in the drainage basin fee computation.

3.4 Drainage Basin Layout

<u>County Basin Code</u>

A basin numbering system has been implemented to provide an orderly system of identifying and indexing County basins. Each basin number is an eight digit alpha-numeric identifier composed of four designators as shown below:

A	B	C	D
00	-00	-00	-00

<u>Designator A</u> - The first two digits signify the major basin designation. El Paso County has thirteen major basins, each tributary to either the South Platte (on the north) or the Arkansas (on the south) Rivers in the State of Colorado. Figure No. 3-1 shows the major basins and the major streams in the County. Thirteen major basins and their two digit codes are listed below:

1. Beaver Creek (BE)
2. Big Sandy Creek (BG)
3. Bijou Creek (BI)
4. Chico Creek (CH)

5. Cherry Creek (CY)
6. Fountain Creek (FO)
7. Haynes Creek (HA)
8. Horse Creek (HO)
9. Kiowa Creek (KI)
10. Kramer Creek (KR)
11. Plum Creek (PL)
12. Trout Creek (TR)
13. Turkey Creek (TU)

Designator B - The second two digits signify the major receiving streams within the major basins. El Paso County has designated twenty three receiving streams. They are shown in Figure No. 3-1 and listed below with their two digit codes:

1. Beaver Creek (BE)
2. Bijou Creek (BI)
3. Brackett Creek (BR)
4. Chico Creek (CH)
5. East Chico Creek (EC)
6. West Fork Black Squirrel Creek (WS)
7. Fountain Creek (FO)
8. William Creek (WI)
9. Horse Creek (HO)
10. Kiowa Creek (KI)
11. Plum Creek (TR)
12. Turkey Creek (TU)
13. Big Sandy Creek (BG)
14. Big Springs Creek (BG)
15. Black Squirrel Creek (BS)
16. Dry Squirrel Creek (DS)
17. Middle Fork Black Squirrel Creek (MS)
18. Cherry Creek (CY)
19. Monument Creek (MO)
20. Haynes Creek (HA)
21. Pond Creek (PO)
22. Kramer Creek (KR)
23. Trout Creek (TR)

Designator C - The third two digits are numbers unique to each basin. These numbers are even, starting with 02 at the downstream end of each major basin, and increasing consecutively upstream. The number is an indication of the location of the basin within the major basin.

Designator D - The last two digits are presently 00 for all basins, allowing further basin subdivision in the future. Therefore, up to 99 subbasins may be incorporated into each of the drainage basins in the County.

Example - A basin with the number CHBR0400 indicates:

CH - The basin is in the Chico Creek major basin.

BR - Brackett Creek is the receiving stream.

04 - The basin is the second basin upstream along Brackett Creek in the Chico Creek major basin.

00 - No further subdivision of the basin has occurred.

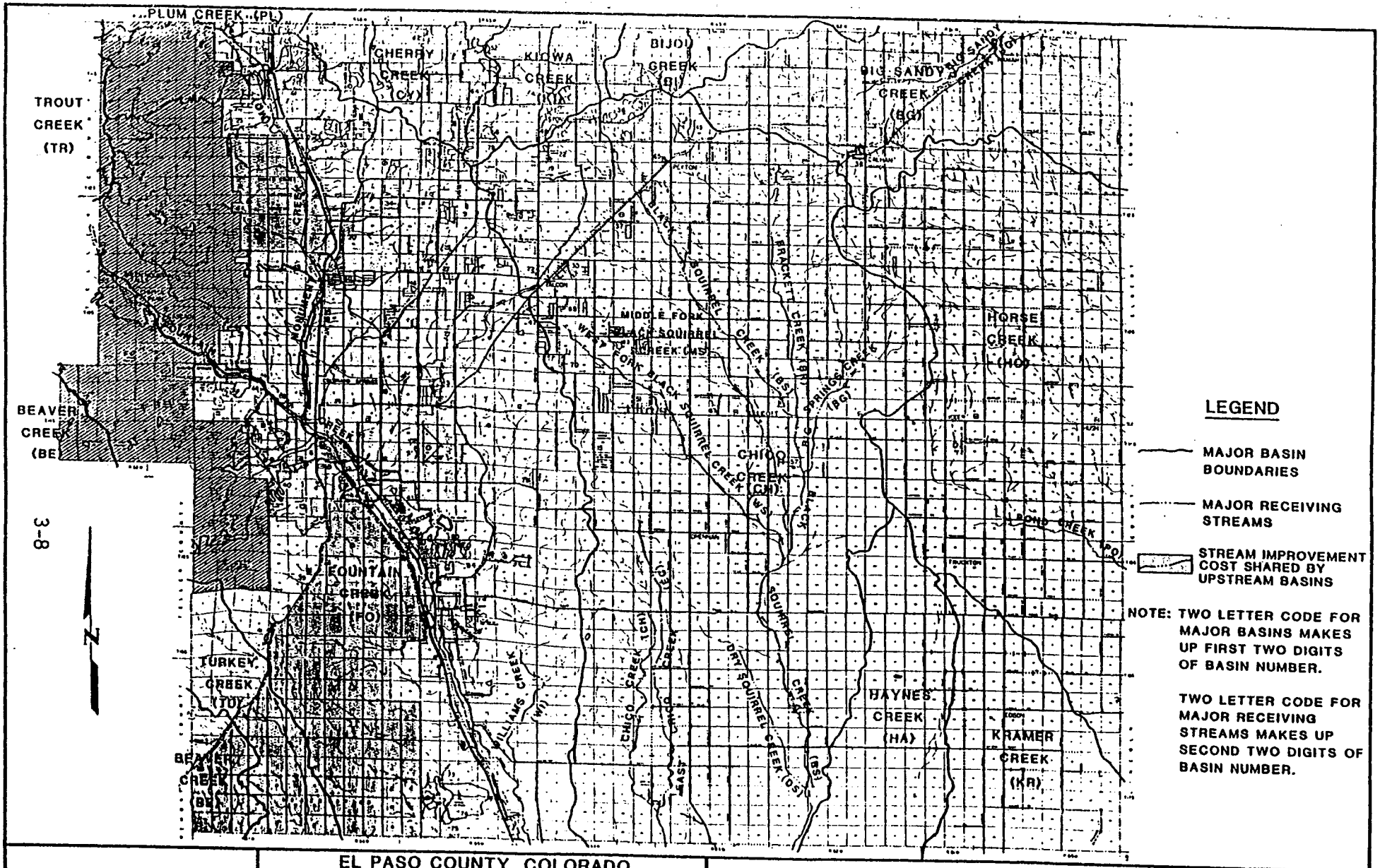
A set of drawings that depict the drainage basins in more detail can be purchased from the City/County. The County maps were prepared from 7.5 Minute United States Geological Survey (USGS) Quadrangle maps. The maps are 24-inch by 36-inch in size, with a final scale of one inch equals 4000 feet.

City Basin Code

The City of Colorado Springs has a code number for each basin in which unit drainage fees are presently being collected. These City basin names and codes are listed below:

01 - Sand Creek
02 - Spring Creek
03 - Templeton Gap
04 - Douglas Creek
05 - 19th Street
06 - Popes Bluff
07 - Camp Creek
08 - Peterson Field
09 - South Rockrimmon
10 - Pulpit Rock
11 - Dry Creek
12 - North Rockrimmon
13 - Cottonwood Creek
14 - Miscellaneous*
15 - Mesa
16 - 21st Street
17 - Bear Creek
18 - Southwest Area
19 - Windmill Gulch
20 - Black Squirrel Creek
21 - Monument Branch
22 - Middle Tributary
23 - Little Johnson
24 - Black Forest
25 - Big Johnson (Crews Gulch)
26 - Fisher's Canyon

*Unstudied Basins



EL PASO COUNTY, COLORADO
Department of Transportation
3170 CENTURY STREET, COLORADO SPRINGS, COLORADO 80907

MAJOR BASINS AND RECEIVING STREAMS

Figure 3 - 1

SECTION I

Chapter 4 Report Guidelines

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CHAPTER 4 REPORT GUIDELINES

4.1 Drainage and Erosion Control Reports and Plans

The four step drainage planning/design process as defined in Chapter One provides the framework for which submittal requirements are established. Discussion of Drainage Basin Planning Studies (DBPS) is limited in this manual to submittal requirements and technical criteria. Each DBPS is unique and the engineer must work with the City/County in preparing the format and technical approach. A general outline as shown in Figure 4-1 may be used as a guideline for development of a DBPS.

The remaining steps in the drainage planning/design process includes the Master Development Drainage Plan and the Preliminary and Final Drainage Reports.

A Master Development Drainage Plan shall be submitted (minimum 7 copies) by developers that addresses both local and regional land development plans. A preliminary drainage report shall be submitted (minimum 7 copies) at the time of preliminary plat (plan) submittal. Following approval of the preliminary drainage report, a final drainage report shall be submitted (minimum 7 copies) with the final plat. The flow chart in Figure 4-2 depicts the City/County Drainage review process. When specific improvements are required, construction drawings and specifications must be submitted for review with the final plat (County only). All reports shall be typed on 8-1/2" x 11" paper and properly bound. Drawings, figures, plates, and tables shall be bound with the report or included in a folder/pocket attached to the report. All reports shall include a cover letter and shall be prepared by a registered professional engineer licensed in Colorado. The report shall be properly certified and signed by the engineer and the owner/developer.

All drainage reports shall be reviewed for compliance with the minimum requirements set forth herein. The Engineer should be aware that whenever unusual or serious drainage problems are anticipated in conjunction with a proposed development, additional information and analysis beyond the minimum requirements of these specifications and criteria will be required. Failure to comply with the provisions of this section may result in the report being rejected for review.

Any proposed development which alters an existing FEMA National Flood Insurance Program (NFIP) may must identify this change in the Preliminary Drainage Report and shall apply for a Letter of Map Revision (LOMR) or Letter of Map Amendment (LOMA). The September, 1985 FEMA publication entitled "Appeals, Revisions, and Amendments to Flood Insurance Maps, A Guide for Community Officers" should be consulted to determine requirements for changing NFIP maps. The City/County may not approve a Final Drainage Report unless a LOMR or LOMA has been submitted to FEMA.

Letter of Transmittal, Professional Engineers Certification and Executive Summary

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- B. Nature and Purpose of Study
- C. Scope of Work
- D. Past Studies - Related Investigations
- E. Agency Jurisdictions (governmental)
- F. Drainage Criteria and Special Requirements Unique to Basin

II Project Description, Location and Drainage
- A. Basin Description and Location
- B. Major Drainageways and Facilities
- C. Existing and Proposed Land Use
- D. Existing Irrigation Facilities
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- F. Existing/Proposed Utilities (If Applicable)
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The City of Colorado Springs / El Paso County Drainage Criteria Manual	Date 9-30-90
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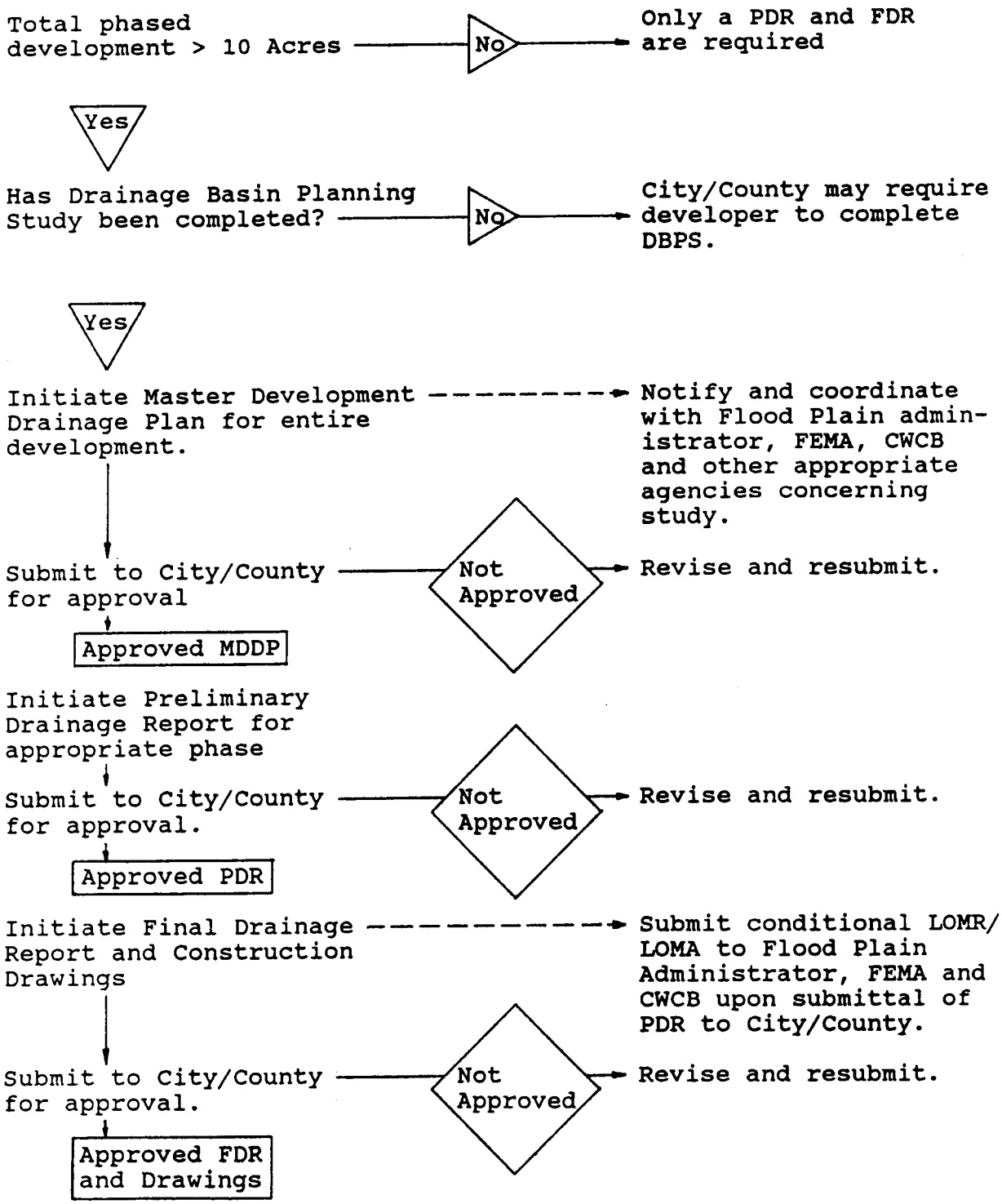

The City of Colorado Springs / El Paso County
Drainage Criteria Manual

Flow Chart for Drainage Studies

Date: OCT. 1987

Figure: 4-2

Type of Map To Be Changed	Status of Map	Type of Change	Process and Authority	Method of Change
FIRM	In 90-Day Appeal Period	Change to BFE	Appeal (44 CFR, Part 67)	Map and Report Revised Before Printing
FHBM FIRM FBFM	Effective	Changes to Any Flood or Non-Flood Information	Map Revision (44 CFR, Part 65)	Physical Map Revision or Letter of Map Revision (LOMR)
		Exclusion From SFHA of Structures and Legally Described Parcels of Undeveloped Land Elevated By Fill Placed After the Effective Date of the First NFIP Map That Showed the Structure or Parcel To Be Within the SFHA		Letter of Map Revision (LOMR)
		Conditional Changes and Conditional Exclusion of Structures and Legally Described Parcels of Undeveloped Land Proposed To Be Elevated By Fill		Conditional LOMR
FHBM FIRM	Effective	Exclusion From SFHA of Structures and Legally Described Parcels of Undeveloped Land	Map Amendment (44 CFR, Part 70)	Letter of Map Amendment (LOMA)
		Conditional Exclusion of Proposed Structures		Conditional LOMA

Reference: Appeals, Revisions and Amendments to Flood Insurance Maps . A Guide to Community Offices FEMA–FIA

HDR Infrastructure, Inc.
A Centerra Company

The City of Colorado Springs / El Paso County
Drainage Criteria Manual

Summary of NFIP Map Change Processes

Date
OCT. 1987

Figure
4 – 3

Since the City/County criteria is based on future conditions, modification to the floodplain and flooding will also be based on future conditions. FEMA does not recognize future conditions as part of the NFIP map development process. Therefore, conditional LOMR/LOMA must be filed with FEMA. The submittal and payment procedures for conditional LOMR/LOMA is outlined in the "National Flood Insurance Program and Related Regulations - October 1985 Revision", 44CFR-Part 72.4. A summary of this process is shown in Figure 4-3.

4.2 Master Development Drainage Plan (MDDP)

The purpose of the Master Development Drainage Plan is to identify major drainageways, ponding/detention areas, locations of culverts, bridges, open channels and drainage areas which are tributary to the proposed development. Phased developments greater than 10 acres must submit a MDDP. The MDDP should present alternate solutions to drainage problems which may have been identified by the Drainage Basin Planning Study. The ability of downstream drainage facilities to pass developed runoff from the proposed development must be thoroughly analyzed in the MDDP.

The report shall include but not be limited to the following information and calculations:

1. Location and description of the proposed development stating the proposed land use acreage and adjacent features to the site.

2. Calculations for design peak flows from all off-site tributary drainage areas.

3. Calculations for design peak flows within the proposed development for all drainage areas.

4. Discussion and analysis of existing and proposed downstream facilities.

5. Discussion of drainage problems anticipated within the development and their solutions.

The drawings shall include but not be limited to the following information:

1. Any and all flood plains must be identified.

2. Existing topography.

3. Location and size of open channels, bridges, culverts, storm sewers and ponding areas.

4. Identification of all drainage areas tributary to the development.

5. Identification of drainage areas within the development.

6. Location of all streets classified higher than residential.

7. The following scales may be used to show the entire development and all off-site drainage areas, 1"=50' to 1"=400'.

4.3 Preliminary Drainage Report (PDR)

The purpose of the Preliminary Drainage Report is to identify specific solutions to problems on site and off-site resulting from the development of the subdivision to be platted. In addition, those problems that exist prior to development must be addressed in the preliminary report.

The PDR shall be in accordance with the following outline and contain the applicable information listed. Drainage reports must utilize the following format and major headings as noted below.

1) **GENERAL LOCATION AND DESCRIPTION**

a) **Location**

1. City and County, and local streets within and adjacent to the subdivision.

2. Township, range, section, 1/4 section.

3. Major drainageways and existing facilities.

4. Names of surrounding platted developments.

b) **Description of Property**

1. Area in acres.

2. Ground cover (type of trees, shrubs, vegetation).

3. General topography.

4. General soil conditions.

5. Major drainageways.

6. Irrigation facilities.

7. Utilities and other encumbrances.

2) **DRAINAGE BASINS AND SUB-BASINS**

a) **Major Basin Descriptions**

1. Reference should be made to major drainage-way planning studies; such as drainage basin planning studies, flood hazard delineation reports, and flood insurance studies or maps, if available.

   A flood plain statement shall be provided indicating whether any portion of the development is in a designated floodplain as delineated on the current FEMA mapping.

2. Major basin drainage characteristics.

3. Identification of all nearby irrigation facilities and other obstructions which could influence or be influenced by the local drainage.

b) **Sub-basin Description**

1. Discussion of historic drainage patterns of the property in question.

2. Discussion of off-site drainage flow patterns and their impact on the development.

3) **DRAINAGE DESIGN CRITERIA**

a) **Development Criteria Reference**

1. Reference all criteria, master plans, and technical information used for report preparation and design; any deviation from such material must be discussed and justified.

2. Discussion of previous drainage studies (i.e., PDR, drainage basin planning studies, master plans, flood insurance studies) for the site in question that influence or are influenced by the drainage design and how the studies affect drainage design for the site.

b) **Hydrologic Criteria**

1. Identify design rainfall.

2. Identify runoff calculation method.

3. Identify design storm recurrence intervals.

4. Identify detention discharge and storage calculation method.

4) **DRAINAGE FACILITY DESIGN**

a) **General Concept**

1. Discussion of compliance with off-site runoff considerations.

2. Discussion of anticipated and proposed drainage patterns.

3. Discussion of the content of tables, chart, figures, plates or drawings presented in the report.

b) **Specific Details**

1. Presentation of existing and proposed hydrologic conditions including approximate flow rates entering and exiting the sub-division with all necessary calculations.

2. Presentation of approach to accommodate drainage impacts on existing or proposed improvements and facilities.

3. Presentation of proposed facilities with respect to alignment, material and structure type.

4. Discussion of the drainage impact of site constraints such as streets, utilities, existing and proposed structures.

5. Environmental features and issues shall be presented if applicable.

6. Discussion of maintenance access and aspects of the preliminary design.

5) **DRAWING CONTENTS**

a) General Location Map: A map shall be provided in sufficient detail to identify drainage flows entering and leaving the development and general drainage patterns. The map should be at a scale of 1" = 50′ to 1" = 2000′. The map shall identify any major construction (i.e., development , irrigation ditches, existing detention facilities, culverts, storm sewers, etc.) that shall influence or be influenced by the subdivision.

b) Drainage Plan: Map(s) of the proposed development at a scale of 1" = 20′ to 1" = 200′ shall be included to identify existing and proposed conditions on or adjacent to the site in question.

The Drainage plan shall delineate all subbasins and proposed initial and major facilities as well as provide a summary of all initial and major flow rates at design points. All floodplains effecting the site shall be shown.

4.4 Final Drainage Report (FDR)

The purpose of the Final Drainage Report is to finalize the concepts and to present the design details for the drainage facilities presented in the PDR. Also, any change to the preliminary design concepts which were incorporated due to review comments by the City/County must be presented.

A. Report Contents

The FDR shall contain all components of the PDR plus additional necessary information relating to the design of specific facilities associated with the development. Such additional information shall include the following:

1) Drainage Facility Design

a) General Concept

1. Discussion of proposed drainage patterns.

2. Discussion of compliance with off-site runoff considerations.

3. Discussion of the content of tables, charts, figures, plates, or drawings presented in the report.

b) Specific Details

1. Presentation of detention storage and outlet design (including reservoir routings) when applicable.

2. Presentation of all hydrologic and hydraulic calculations including hydraulic grade line computations as appropriate.

3. Presentation of an accurate, complete, current estimate of cost of proposed facilities.

4. Presentation of all drainage fees and bridge fees for the property in question, if applicable.

c) Other Government Agency Requirements

1. Federal Emergency Management Agency (FEMA)

2. Army Corps of Engineers (COE)

3. Colorado State Engineer

4. Colorado Water Conservation Board (CWCB)

5. Others

2) Drawing Contents

a) General Location Map: See section pertaining to PDR

b) Drainage Plan: Map(s) of the proposed development at a scale of 1" = 20′ to 1" = 200′ shall be included. The plan shall show the following:

1. Existing and proposed contours at 2 feet maximum intervals. For subdivisions involving rural lots greater than 1.0 acre, the maximum interval may be 5 feet where approved. In terrain greater than 10% - 10 feet.

2. Property lines and existing or proposed easements with purposes noted.

3. All streets

4. Existing drainage facilities and structures, including irrigation ditches, roadside ditches, drainageways, gutters and culverts, all indicating flow direction. All pertinent information such as material, size, shape, slope, and locations shall also be included.

5. Overall drainage area boundary and drainage sub-area boundaries relating to the subdivision.

6. Proposed type of street sections (i.e., vertical or ramp curb and gutters, roadside ditch, gutter flow and/or cross pans).

7. Proposed storm sewers and open drainageways, including inlets, manholes, culverts, and other appurtenances.

8. Proposed outfall point for runoff from the developed area and facilities to convey flows to the final outfall point without damage to downstream properties.

9. Routing and summary of initial and major flow rates at various design points for all storm runoff associated with the property.

10. Path(s) chosen for computation of time concentration.

11. Details of and design computations for detention storage facilities including outlet.

12. Location and elevations of all defined 100-year flood plains affecting the property.

13. Location of all existing and proposed utilities affected by or affecting the drainage design.

4.5 Small Subdivision Drainage Report Format

This format is designed for the "Letter Type" drainage report which is required for a resubdivision or replat of property for which a complete drainage report has previously been approved by the City/County Engineer and significant changes from such report is not proposed.

The "Letter Type" drainage report will include the following:

1. Cover sheet or statement stating the name and purpose of the report. This shall include the date of preparation and the name of the previous subdivision.

2. Engineer's Statement (Standard for City/County, see 4.6).

3. Developer's Statement (Standard for City/County, see 4.6).

4. Body of the report shall include:

   a. General property description with acreage.
   b. General existing drainage characteristics (on and off site).
   c. General proposed drainage characteristics (on and off site).
   d. Hydrologic calculations with tabulations of: areas, runoff, coefficients, time of concentrations intensity, or "Q", "q_p", time to peak, etc. (Required if existing conditions have channels)

5. A site map showing location with regard to the area.

6. A drainage plan indicating site and adjacent property as platted with name and filing. Indicate storm runoff routing and rates if applicable.

7. The drainage fees (cash or letter of credit) shall be determined in accordance with the latest drainage ordinances/resolutions and applicable basin fees.

4.6 Drainage Statements

4.6.1 City/County Drainage Report Statements and Certifications

The following statements must be included with drainage reports and detailed drainage plans and specifications which are submitted to the City/County.

<u>Engineer's Statement:</u>

The attached drainage plan and report were prepared under my direction and supervision and are correct to the best of my knowledge and belief. Said drainage report has been prepared according to the criteria established by the City/County for drainage reports and said report is in conformity

with the master plan of the drainage basin. I accept responsibility for any liability caused by any negligent acts, errors or omissions on my part in preparing this report.

_____ Seal

Name

Developer's Statement:

I, the developer have read and will comply with all of the requirements specified in this drainage report and plan.

_____

Business Name

By: _____

Title: _____

Address: _____

_____

EL PASO COUNTY ONLY:

Filed in accordance with Section 51.1 of the El Paso Land Development Code, as amended.

_____ _____

Director of Public Works Date

Conditions:

CITY OF COLORADO SPRINGS ONLY:

Filed in accordance with Section 15-3-906 of the Code of the City of Colorado Springs, 1980, as amended.

_____ _____

City Engineer Date

Conditions:

4.6.2 Construction Plan Drainage Statements and Notes

a. Detailed Drainage Construction Plans and Specifications Engineer's Statement:

These detailed plans and specifications were prepared under my direction and supervision. Said detailed plans and specifications have been prepared according to the criteria established by the City/County for detailed drainage plans and specifications, and said detailed plans and specifications are in conformity with the master plan of the drainage basin. Said detailed drainage plans and specifications meet the purposes for which the particular drainage facility(s) is designed. I accept responsibility for any liability caused by any negligent acts, errors or omissions on my part in preparation of the detailed drainage plans and specifications.

b. Required Notes

The following shall be placed on all drainage plan drawings:

"City/County plan review is provided only for general conformance with City/County Design Criteria. The City/County is not responsible for the accuracy and adequacy of the design, dimensions, and/or elevations which shall be confirmed at the job site. The City/County through the approval of this document assumes no responsibility for completeness and/or accuracy of this document."

4.7 Submittal and Review Procedures

The preliminary drainage report must be submitted to the County as part of the preliminary plat/plan or P.U.D. application.

The City Engineer will require a preliminary drainage analysis at the time of Development Plan or Concept Plan submittal in order that drainage issues can be addressed to adequately evaluate the proposed development.

The final drainage report must be submitted to the City/County as part of the final plat or P.U.D. application. For additional information, see the City of Colorado Springs Subdivision Policy Manual. The construction plans must be submitted concurrently with the final drainage report for the <u>County Only</u>.

4.8 Erosion Control Plan

4.8.1 General Principals

The required erosion control plan is a plan for controlling erosion during construction in compliance with the laws, ordinances, regulations, resolutions, standards and specifications. This plan shall be a part of the total site development plans and prescribe all the steps necessary including scheduling to assure

erosion control during all phases of construction including final stabilization. The erosion control shall be sufficient to control the runoff from the design storm.

Planning for erosion control shall begin with Preliminary Drainage Report preparation, and shall include first-hand knowledge of the site by the engineer. The plan shall be based on a sufficiently accurate topographic map. Adjacent off-site areas affecting the site or affected by the site as developed shall be shown on the plans in sufficient detail to determine plan adequacy.

An Erosion Control Plan is required of property owners/developers prior to commencing any grading and where platting occurs; a plan is required at the time of final drainage report submittal. The plan will be approved concurrently with the report unless otherwise waived by the City/County. Temporary erosion and sedimentation control facilities shall be constructed prior to any grading or extensive land clearing, in accordance with the Erosion Control plan. These facilities must be satisfactorily maintained until construction and landscaping are completed and the potential for on-site erosion has been significantly reduced.

The Erosion Control Plan shall consist of the best selection of erosion control practices and sediment trapping facilities in conjunction with an appropriate schedule in order to accomplish adequate control. Adequate erosion control measures shall be constructed prior to land disturbing activities such that no adverse affect of site alteration will impact surrounding properties. Particular attention shall be given to concentrated flows of water, either to prevent their occurrence or to provide appropriate conveyance devices to prevent erosion. Sediment trapping devices shall be required at all points where sediment laden water might leave the site. The plan must include permanent structures for conveying storm runoff, site grading, final site stabilization, temporary sediment control features including sediment basins and finally, stabilization of the site where temporary features have been removed. Plans showing improvements or construction to be constructed outside the property line of the site will not be approved unless the plan is accompanied by an appropriate legal easement for the area in which such work is required.

Unless otherwise approved on of the following scales shall be used for the erosion control plan; 1"=20′ to 1"=100′. The contour interval for these plans shall be two feet unless otherwise allowed by the City/County.

4.8.2 Information to be included in the Erosion Control Plan

The plan shall describe the following information for the entire tract of land to be disturbed, whether or not the tract will be developed in stages:

a. A vicinity map indicating the proposed development in relationship to roadways, jurisdictional boundaries, and streams.

b. A site plan showing soil types, existing and proposed contours, existing vegetation, existing and proposed water courses, critical erosion areas, adjacent existing and proposed development affected, and the proposed features of the site.

c. A plan for temporary and permanent vegetative and structural practices which specify the conservation measures to be used during all phases of clearing, grading, filling, construction, and permanent development.

d. A schedule of anticipated starting and completion dates for each sequence and stage of land-disturbing activities depicting conservation measures anticipated, including the expected date on which the final stabilization will be completed.

e. A detailed description of the maintenance program for sediment control facilities, including inspection programs, vegetative establishment on exposed soils, method and frequency of removal and disposal of waste materials from control facilities, and disposition of temporary structural measures.

f. Soil borings and tests, including groundwater analysis and plan for safe discharge if required by the City/County.

g. The Erosion Control Plan is to be certified by a Colorado Registered Architect or Engineer and to be signed by the Owner with a statement that "The Owner will comply with the requirements of the Erosion Control Plan".

4.8.3 Additional Information Requirements

Additional data shall be required for all projects where the City/County deems that soil erosion and sedimentation control problems may exist. Such data may include, but not be limited to other engineering studies, computations, schedules, and supportive data deemed necessary by the City/County.

It shall be understood that additional protection may be required should construction site observation indicate the need.

4.8.4 Scope and Exclusions

These criteria shall apply to any land-disturbing activity undertaken by any person on any lands, except on minor land-disturbing activities, such as individual home landscaping and gardening, maintenance and repair work, and those land-disturbing activities which, in the judgement of the City/County, as set forth in writing, will not result in significant soil erosion or the movement of significant sediment into waters or onto lands off the project site. Factors to be considered when making such determinations shall be:

a. Any project involving land-disturbing activity of 2.0 acres, or less.

b. Agriculture and related activities within the jurisdiction of the City/County.

c. Construction of single-family residences when they are constructed by, or under contract with, the owner for the owner's occupancy.

d. A project which disturbs less than 500 CY of material (cut and/or fill).

4.8.5 Plan Implementation

No clearing, grading, excavation, filling, or other land-disturbing activities shall be permitted until approval of the Erosion Control Plan is received from the City/County Department of Public Works. Once approval is received, approved conservation measures must be installed before land-disturbing activities are initiated such that no adverse effect of site alteration will impact surrounding property. These conservation measures shall apply to all features of the construction site including, but not limited to, street and utility installations, as well as to the protection of individual lots. During all phases of construction, it shall be the responsibility of those initiating such land disturbing activities to maintain all erosion control features in a functional manner.

4.8.6 Correction of Deficiencies

Should the approved erosion control measures as initially installed be observed ineffectual, modifications to correct deficiencies shall be made immediately by the owner/developer to restore and maintain these measures for the intended purpose until stabilization is achieved.

4.8.7 Guarantee

Financial assurances for all temporary and permanent measures to prevent and control anticipated erosion shall be provided by the owner/developer at the time of plan approval and not released until such time that the disturbed areas are stabilized or established to the satisfaction of the City/County.

SECTION II

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CHAPTER 5 STORM RUNOFF METHODS OF ANALYSIS

5.1 General Statement

This section presents three methodologies for computing storm water runoff in the City/County. These include the Rational Method, the Soil Conservation Service (SCS) Hydrograph procedure and statistical methods. The statistical methods include the Log-pearson Type III frequency distribution and multiple regression analyses. The Rational Method will be used for the determination of runoff from areas of approximately 100 acres or less, and is especially useful for the design of storm sewer systems. The SCS procedure is used to quantify the effects of urbanization to determine peak flows for large drainage areas, and to design storm water storage facilities.

The various runoff models selected for use in evaluating drainage in the City/County are broken down by basin area restrictions as listed below:

Drainage Basin Size	Runoff Methodology
100 acres or less	Rational Method
Greater than 100 acres	SCS Hydrograph Procedure
Major Basins (greater than 10 sq. miles)	Statistical or Computer Models

The engineer must secure approval from the appropriate design review agencies before utilizing different methods.

Acceptable computer simulation models used in hydrologic evaluation are listed in the following Table. These computer models, designed for general use, should be calibrated for local use, and based on locally available data. Updated versions and variations of the models are available, but any model used for storm water design in the City/County should be carefully calibrated with local data and methods to ensure consistent results for all basin studies in the City/County. Several of these models listed are available in IBM PC compatible forms. Many of the storm sewer routing programs which utilize the Rational Method of analysis to develop basin hydrology are excellent analytical tools. Use of a computer modeling tool must be approved by the City/County engineering staff.

It should be recognized that the engineer must compute storm water runoff for at least the 10-year and 100-year storms at all design points to conform to the City/County design criteria. The design of initial and major storm drainage systems will require checking both runoff events to ensure all drainage criteria are met. The fully developed, ultimate condition storm water runoff for the Basins is to be analyzed unless otherwise determined by the City/County.

STORM WATER DRAINAGE COMPUTER SIMULATION MODELS
(after Nieber et al., 1976; Viessman et al., 1977)

Model Code	Model Name	Agency	Original Development Date
TR-20	Computer Program for Project Hydrology	SCS, USDA	1965
HEC 1	Flood Hydrograph Package	Corps of Engineers, U.S. Army	1973
STORM	Quantity & Quality of Urban Runoff	Corps of Engineers, U.S. Army	1974
MITCAT	M.I.T. Catchment Model	MIT (Univ.)	1970
SWMM	Storm Water Management Model	EPA	1971
USDAHL	Watershed Hydrology	ARS, USDA	1970
SWM-IV	Stanford Watershed Model IV	Stanford Univ.	1959
CUHP	Colorado Urban Hydrograph Procedure	UDFCD	1974

5.2 Rational Method

The Rational Method is an empirical runoff formula which has gained wide acceptance because of its simple, intuitive treatment of estimating peak storm runoff. This method relates peak runoff to drainage area, rainfall intensity, and basin surface characteristics by the formula:

$$Q = CiA \qquad (5\text{-}1)$$

where:

Q = peak runoff rate, in cubic feet per second (cfs).

C = Runoff coefficient representing a ratio of peak runoff rate to average rainfall intensity for a duration equal to the runoff time of concentration;

i = average rainfall intensity in inches per hour; and

A = drainage area in acres; and

The Rational Method is based on the following assumptions:

1. The peak rate of runoff at any point is a direct function of the average uniform rainfall intensity for the estimated runoff time of concentration to the point.

2. The frequency of the peak discharge is the same as the frequency of the average rainfall intensity.

3. The runoff time of concentration is the time required for the runoff to become established and flow from the most hydraulically remote part of the drainage area to the design point under consideration. This assumption applies to the runoff most remote in time, not necessarily in distance.

4. The peak discharge per unit area generally decreases as the drainage area increases, and the intensity of rainfall decreases as its duration increases.

Although the basic principles of the Rational Method generally apply to drainage areas less than 200 acres, common design practice generally limits its use to some lesser area. The Rational Method shall be used for basins up to 100 acres for the Colorado Springs/El Paso County Area. For larger areas, basin storage and subsurface drainage flow cause an attenuation of the runoff so that the rates of flow tend to be overestimated by the Rational Method. In addition, the assumption of uniform rainfall distribution and intensity becomes less appropriate as drainage area increases. Because of the trend for overestimation of flows

and the additional cost in drainage facilities associated with this overestimation, the application of a more sophisticated runoff computation technique is usually warranted on larger drainage areas. The designer must obtain permission from the appropriate design review agency before applying the Rational Method to areas larger than 100 acres.

5.2.1 Runoff Coefficient, C

The runoff coefficient, C, is a variable of the Rational Method which is least susceptible to precise determination and provides the designer with a degree of latitude to exercise his independent judgment based upon known effecting parameters. The following discussion is intended to provide a guide to promote the uniform selection and application of runoff coefficients.

The runoff coefficient accounts for abstractions for losses between rainfall and runoff which may vary with time for a given drainage area. These losses are caused by intercepting vegetation, infiltration into soils, retention in surface depressions, evaporation and transpiration. In determining this coefficient, different climatological and seasonal conditions, antecedent moisture conditions, and the intensity and frequency characteristics of the design storm should be considered.

Table 5-1 provides runoff coefficients that vary with recurrence frequency. The coefficients were developed using the available rainfall and runoff information in the Denver region and have been adjusted to work in conjunction with the Colorado Springs/ El Paso County area. Use of these coefficients and procedures in other than semi-arid climates may not be valid. However, because the coefficients vary with frequency, no further adjustments are needed for large storms. Adjustments should be made for level of development, surface type, soil type, and surface slope. It is often desirable to develop a composite runoff coefficient based in part on the percentage of different types of surfaces in the drainage area. This procedure can be applied to typical "sample" areas as a guide to the selection of usual values of the coefficient for the entire area.

A composite runoff coefficient is calculated using the relationship:

$$C = \frac{\sum_{i=1}^{n} C_i A_i}{A_t} \qquad (5\text{-}2)$$

where:

C_i = individual runoff coefficient corresponding to surface type;

A_i = area of surface type corresponding to C_i;

A_t = total drainage area for which composite runoff coefficient is applicable;

n = total number of surface types in drainage areas; and

C = the composite runoff coefficient.

The coefficients presented in Table 5-1 are applicable for the initial 10-year storm and for the major 100-year storm. These coefficients, however, are based on the assumption that the design storm does not occur when the ground surface is frozen or covered by melting snow. The designer must exercise good judgement in estimating runoff coefficients for these conditions, if necessary.

5.2.2 Rainfall Intensity, i

Rainfall intensity, i, is the average rate of rainfall, in inches per hour, for a storm of a given duration equal to the estimated runoff time of concentration. Intensity is selected on the basis of design frequency of exceedence, a statistical parameter established by design criteria, and the storm rainfall duration. For the Rational Method, the critical rainfall intensity is the rainfall having a duration equal to the estimated runoff time of concentration for the drainage basin. For the City/County, rainfall intensity for T_c values ranging from 0 to 60 minutes can be determined for the 10-year and 100-year return periods from Figure 5-1.

5.2.3 Runoff Time of Concentration, T_c

One of the basic assumptions underlying the Rational Method analysis is that runoff from the most hydraulically remote point of the drainage basin to the design point under consideration. Runoff time of concentration is usually estimated by calculating travel time through the basin. Overland flow, storm sewer and/or road gutter flow, and channel flow are typical phases of direct flow commonly used in calculating travel time.

Overland Flow

The travel time for overland flow is the estimate in time required for flow to travel from the uppermost part of a drainage basin to a defined channel or inlet of a local storm sewer system. Overland flow can be significant in small basins because a significant portion of time of concentration is due to overland flow. The velocity of overland flow can vary greatly with the surface cover and tillage characteristics. If the slope and land use of the overland flow reach are known, the travel time can be read from figure 5-2 or calculated using the following equation:

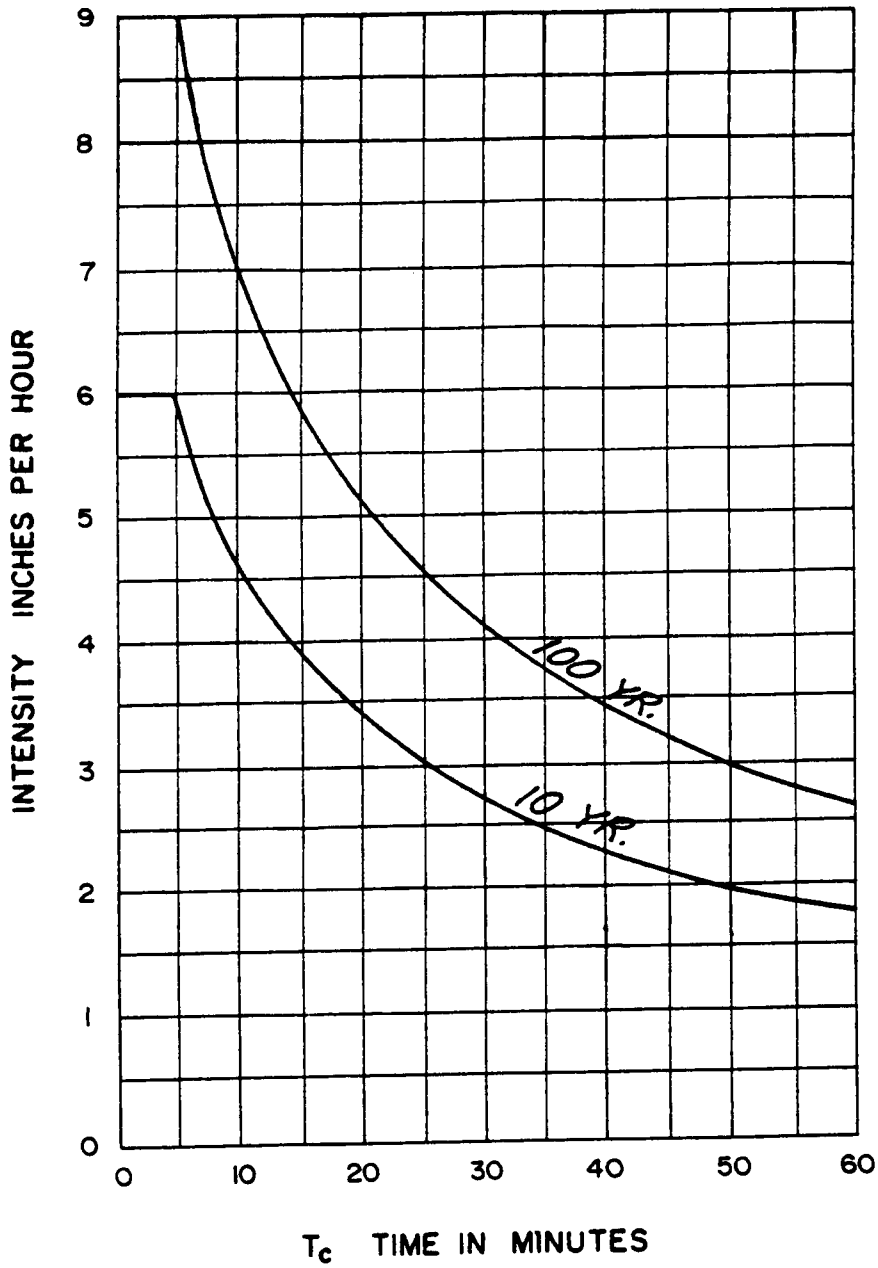
TABLE 5-1

RECOMMENDED AVERAGE RUNOFF COEFFICIENTS AND PERCENT IMPERVIOUS

LAND USE OR SURFACE CHARACTERISTICS	PERCENT IMPERVIOUS	"C" FREQUENCY 10 A&B*	10 C&D*	100 A&B*	100 C&D*
Business					
Commercial Areas	95	0.90	0.90	0.90	0.90
Neighborhood Areas	70	0.75	0.75	0.80	0.80
Residential					
1/8 Acre or less	65	0.60	0.70	0.70	0.80
1/4 Acre	40	0.50	0.60	0.60	0.70
1/3 Acre	30	0.40	0.50	0.55	0.60
1/2 Acre	25	0.35	0.45	0.45	0.55
1 Acre	20	0.30	0.40	0.40	0.50
Industrial					
Light Areas	80	0.70	0.70	0.80	0.80
Heavy Areas	90	0.80	0.80	0.90	0.90
Parks and Cemeteries	7	0.30	0.35	0.55	0.60
Playgrounds	13	0.30	0.35	0.60	0.65
Railroad Yard Areas	40	0.50	0.55	0.60	0.65
Undeveloped Areas					
Historic Flow Analysis- Greenbelts, Agricultural	2	0.15	0.25	0.20	0.30
Pasture/Meadow	0	0.25	0.30	0.35	0.45
Forest	0	0.10	0.15	0.15	0.20
Exposed Rock	100	0.90	0.90	0.95	0.95
Offsite Flow Analysis (when land use not defined)	45	0.55	0.60	0.65	0.70
Streets					
Paved	100	0.90	0.90	0.95	0.95
Gravel	80	0.80	0.80	0.85	0.85
Drive and Walks	100	0.90	0.90	0.95	0.95
Roofs	90	0.90	0.90	0.95	0.95
Lawns	0	0.25	0.30	0.35	0.45

* Hydrologic Soil Group

9/30/90



RE: Based upon Pikes Peak area council of governments/ areawide urban runoff control manual.

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Storm Rainfall
Time Intensity-Frequency Curves

Date
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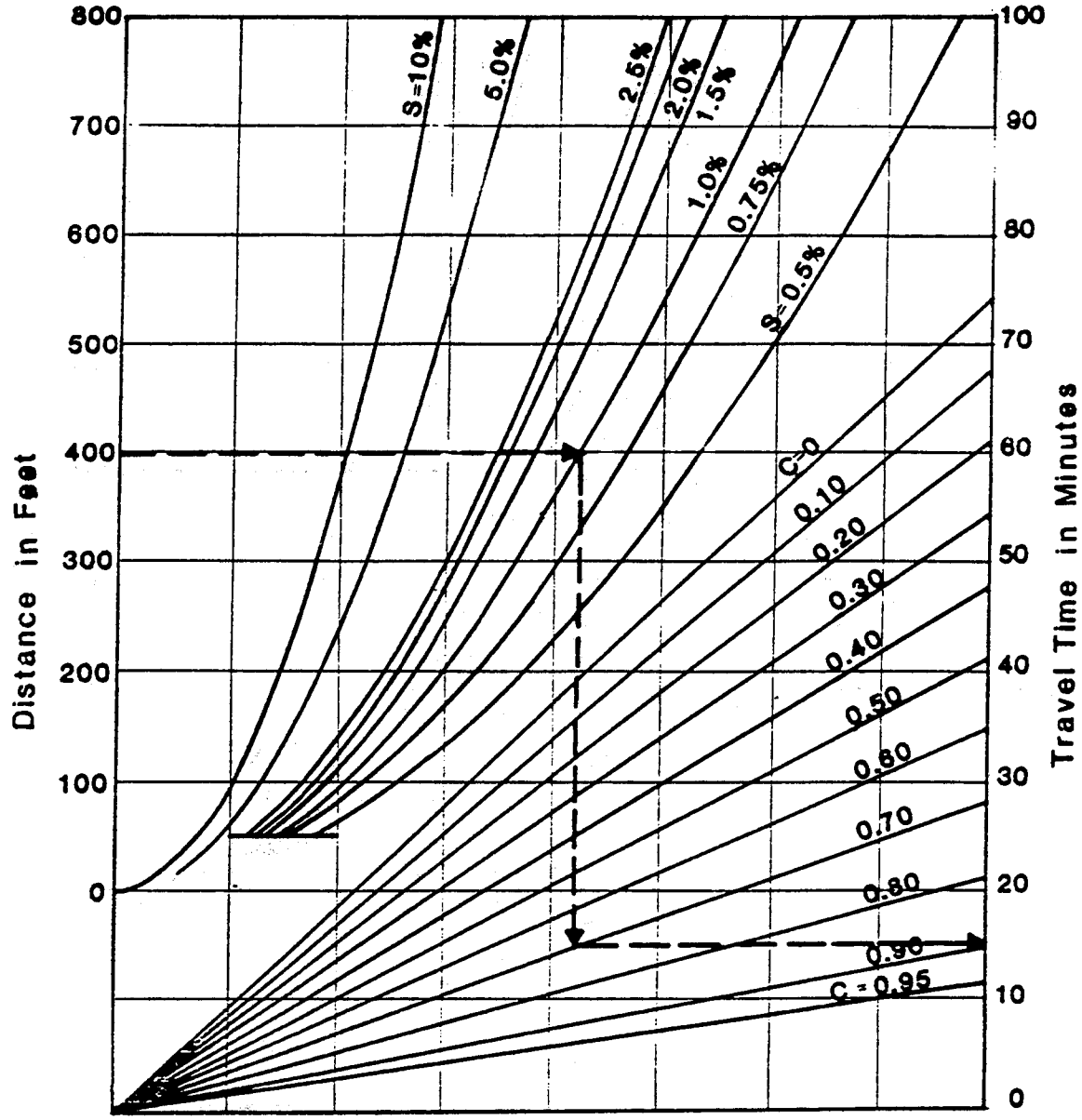
Figure
5 - 1



REFERENCE : Wright - McLaughlin Engineers, Urban Storm Drainage Criteria Manual, Vol. 1, Denver Regional Council of Governments, Denver, Co. 1977

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Figure
5-2

Overland Flow Curves

$$T_c = 1.87\ (1.1 - C_{10})L^{0.5}\ S^{-0.33}$$

where:

C_{10} = adjusted runoff coefficient for ten-year flow;

L = length of overland flow in feet;

S = slope of flow path in percent; and

T_c = travel time in minutes

Times of concentration calculated for fully developed land use should not be less than five (5) minutes to avoid the oversizing of inlets, storm drains and open channels. Overland flow lengths should not exceed 300 feet for developed areas or 1,000 feet for undeveloped areas before being intercepted by a channel or storm sewer inlet. Beyond these distances, gutter flow or channel flow velocities calculated by Manning's Formula must be used. Refer to Section III for discussion on Manning's Formula.

Storm Drain or Road Gutter Flow

Travel time through a storm drain or road gutter system to an open channel is the sum of travel times in each individual component of the drainage system between the uppermost inlet and the outlet. In most cases, average velocities can be used without a significant loss of accuracy. During major storm events, the sewer system may be full and additional channel flow may occur, generally at a significantly lower velocity than the flow in the storm drains. By using the average conduit size and the average slope (excluding any vertical drops in the system), the average velocity can be estimated using Manning's Formula.

Since the hydraulic radius of a pipe flowing half full is the same as that for a pipe flowing full, the respective velocities are equal. Therefore, travel time may be based on the pipe flowing full or half full. The travel time through a storm sewer is computed by dividing the length of pipe by the average velocity of flow.

Channel Flow

The travel time for flow in an open channel can be determined by using Manning's formula to compute average velocities. Bankfull velocities should be used to compute these averages. Channels may be either natural or in an improved condition.

Example 1

A model urbanized drainage basin is shown on Figure 5-3. Three types of flow exists from the furthermost point of the basin to the outlet. Compute time of concentration (t_c) based on the following data:

Reach	Description of Flow	Slope Percent	Length Feet
A to B	Overland (park, C = 0.2)	7	500
B to C	Street Flow (n=.016)	2	900
C to D	Storm drain (n = 0.013; diameter = 3 feet)	1.5	2,000
D to E	Open channel, trapezoidal (b = 5 feet; d = 3 feet; r = 1.78 feet; z = 1.5; n = 0.015)	0.5	3,000

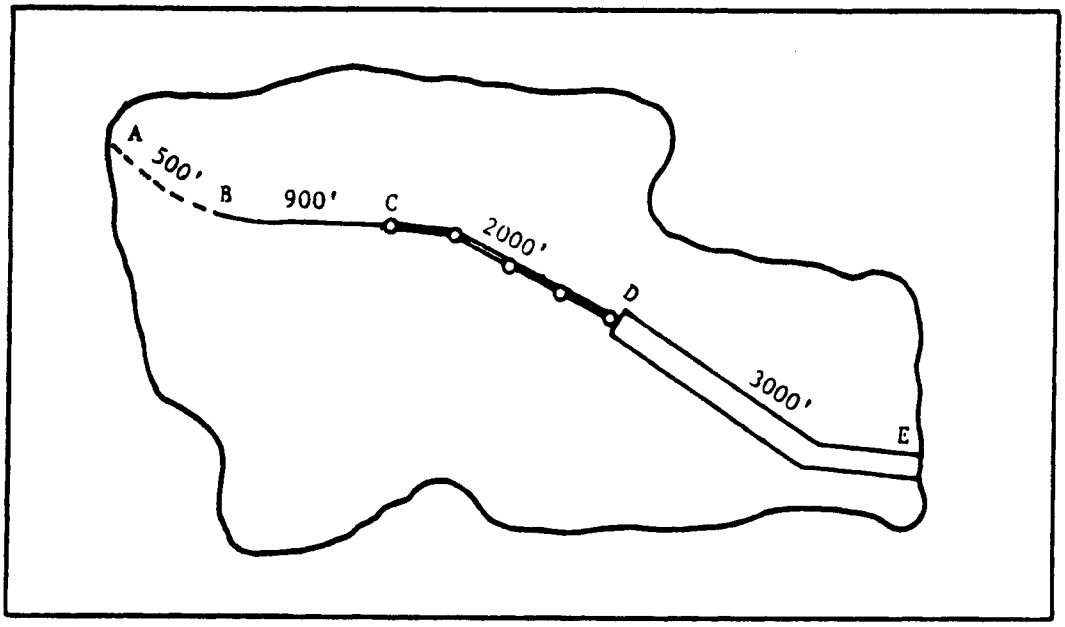


Figure 5-3 Urbanized Drainage Basin

1. Compute the overland flow travel time.
   Reach A to B (park). From Figure 5-2 for a slope of 7 percent, length of 500 feet, and C = 0.2, read T = 19.7 min = 1180 sec.

2. Reach B to C (street gutter). From Mannings Equation for a slope of 2 percent, length of 900 feet, and n = .016, find T = 3.9 min = 237 sec.

$$v = \frac{1.49}{n} R^{2/3} s^{1/2}$$

n = .016 for gutter flow
s = .02

$$R = \frac{S \times D}{S+D}$$ (when S, spread is much greater than D); for D=6", R=0.5

$$v = \frac{1.49}{.016} (0.5)^{2/3} (.02)^{1/2}$$

v = 8.3 ft/sec

3. Compute the storm drain flow travel time.
Reach C to D. Use Manning's equation to compute full-pipe velocity.

$$v = \frac{1.49}{n} \frac{d}{4}^{2/3} s^{1/2}$$

$$v = \frac{1.49}{0.013} \frac{d}{4}^{2/3} (0.015)^{1/2} = 11.6 \text{ ft/sec}$$

$$T = \frac{\text{length}}{\text{velocity}} = \frac{2{,}000 \text{ ft}}{11.6 \text{ ft/sec}} = 172 \text{ sec}$$

4. Compute the open-channel flow travel time.
Reach D to E. Use Manning's formula to compute bankfull velocity.

$$v = \frac{1.49}{n} r^{2/3} s^{1/2}$$

$n = 0.015$ for concrete channel

$s = 0.005$

$$v = \frac{1.49}{0.015} (1.78)^{2/3} (0.005)^{1/2} = 10.3 \text{ ft/sec}$$

$$T = \frac{\text{length}}{\text{velocity}} = \frac{3{,}000 \text{ ft}}{10.3 \text{ ft/sec}} = 292 \text{ sec}$$

5. Summary

Reach	Description of flow	Length (ft)	Velocity (ft/sec)	Travel Time (sec)
A to B	Overland	500	0.42	1180
B to C	Street Flow	900	8.3	108
C to D	Storm Sewer	2,000	11.6	172
D to E	Open Channel	3,000	10.3	292
Total		6,400	3.65 (avg)	1752

$$t_c = \frac{1{,}752 \text{ sec}}{60 \text{ sec/min}} = 29.2 \text{ min}$$

5.3 Soil Conservation Service (SCS) Hydrograph Procedure

The Soil Conservation Service (SCS) Hydrograph procedure is to be used for computing flood peaks and associated runoff volumes for drainage basins large than 100 acres. This method considers determination of probable storm rainfall patterns, associated rainfall losses and unit hydrograph development for estimating potential flooding conditions in basins. Site-specific information is required, as discussed in the following paragraphs.

5.3.1 Definition of Terms

The following definitions of terms are included to provide a better understanding of the SCS unit hydrograph method.

Actual retention, F: Total of initial abstraction (Ia) and infiltration, in inches.

Duration, D: Period of unit rainfall excess, in hours.

Concentration Time, T_c: Time for runoff to travel from the hydraulically most distant part of the watershed to the design point, in hours.

Design Storm: Selected storm duration and recurrence interval.

Dimensionless Unit Hydrograph: A hydrograph made to represent many unit hydrographs by using the time to peak and the peak discharge as basic units and plotting the hydrographs in ratios of these units.

Direct Runoff, Q: Volume of runoff resulting from precipitation (P) minus actual retention, in inches.

Drainage Area, A: The total area that contributes directly to surface runoff, in square miles.

Hydraulic Length (e): Distance from the hydraulically most distant part of the watershed to the design point, in miles.

Impervious Area, I: The area within a basin through which water infiltrates with great difficulty. Typical impervious surfaces include parking lots, roads, paved driveways, sidewalks, and roofs.

Initial Abstraction, I_a: Volume of rainfall at the beginning of the storm which does not appear as runoff, in inches.

Lag Time, L: Time from the midpoint of rainfall excess to unit hydrograph peak, in hours.

Peak Discharge, q_p: The maximum rate of runoff created by Q inches of direct runoff, in cfs.

Point Rainfall, P: The total precipitation, or rainfall, at a given point for a storm of given duration and return period.

Potential Maximum Retention, S: Maximum volume of rainfall not contributing to runoff due to depression storage and infiltration, in inches.

Runoff Hydrograph: The product of the unit hydrograph ordinates and excess rainfall increments.

Slope, s: The average slope of the channel, in feet per mile.

Y: The slope in percent of the land of the watershed.

Storm Distribution: Rainfall vs. time for the selected storm duration.

Storm Duration: Actual duration of excess rainfall.

Time to Peak, T_p: Time form the beginning of rainfall excess to the peak rate of runoff, in hours.

Unit hydrograph: The hydrograph having a volume of 1 inch of runoff from the corresponding unit hydrograph rainfall duration.

5.3.2 Drainage Basin Delineation

Drainage basins are delineated based on available detailed topographic maps, field reconnaissance and survey data. Road embankments and urban developments that have altered normal drainage patterns must be considered in the drainage basin delineation. Major drainage basins have been identified in Chapter 3 and are shown on Figure 3-1.

5.3.3 Storm Rainfall

In order to estimate the maximum rate of runoff from a given drainage basin, the design storm duration should be at least equal to the estimated runoff time of concentration. Also, for establishing a critical precipitation time distribution, the total design storm precipitation must be divided into short duration rainfall events, which may be sequenced to represent a critical storm pattern.

Storm rainfall isopluvial maps reproduced from the NOAA Atlas 2, Volume III, are included as Figures 5-4a, b, c, d, and e. The 24-hour point rainfall (precipitation) is read <u>directly</u> from Figure 5-4d for a 10-year storm, and Figure 5-4e for a 100-year storm. The 2-hour point rainfall is derived from the isopluvial maps in a less direct manner. The procedure involves first computing the 1-hour rainfall for the 10-year and 100-year events, to which the percentages given in Figure 5-5a are applied to arrive at the 2-hour storm distribution and total rainfall.

The 1-hour rainfall is derived by the following method as described in the NOAA Atlas and read from Figures 5-4a thru 5-4e:

1. Select and label the following values from the isopluvials.

   2-year 6-hour precipitation = X_1 inches
   2-year 24-hour precipitation = X_2 inches
   100-year 6-hour precipitation = X_3 inches
   100-year 24-hour precipitation = X_4 inches
   Elevation = Z in hundreds of feet

2. Compute the 2-year 1-hour rainfall from the formula

$$Y2 = 0.218 + 0.709\ [(X_1)(X_1/X_2)]$$

3. Compute the 100-year 1-hour rainfall from the formula

$$Y100 = 1.897 + 0.439\ [(X_3)(X_3/X_4)] - 0.008Z$$

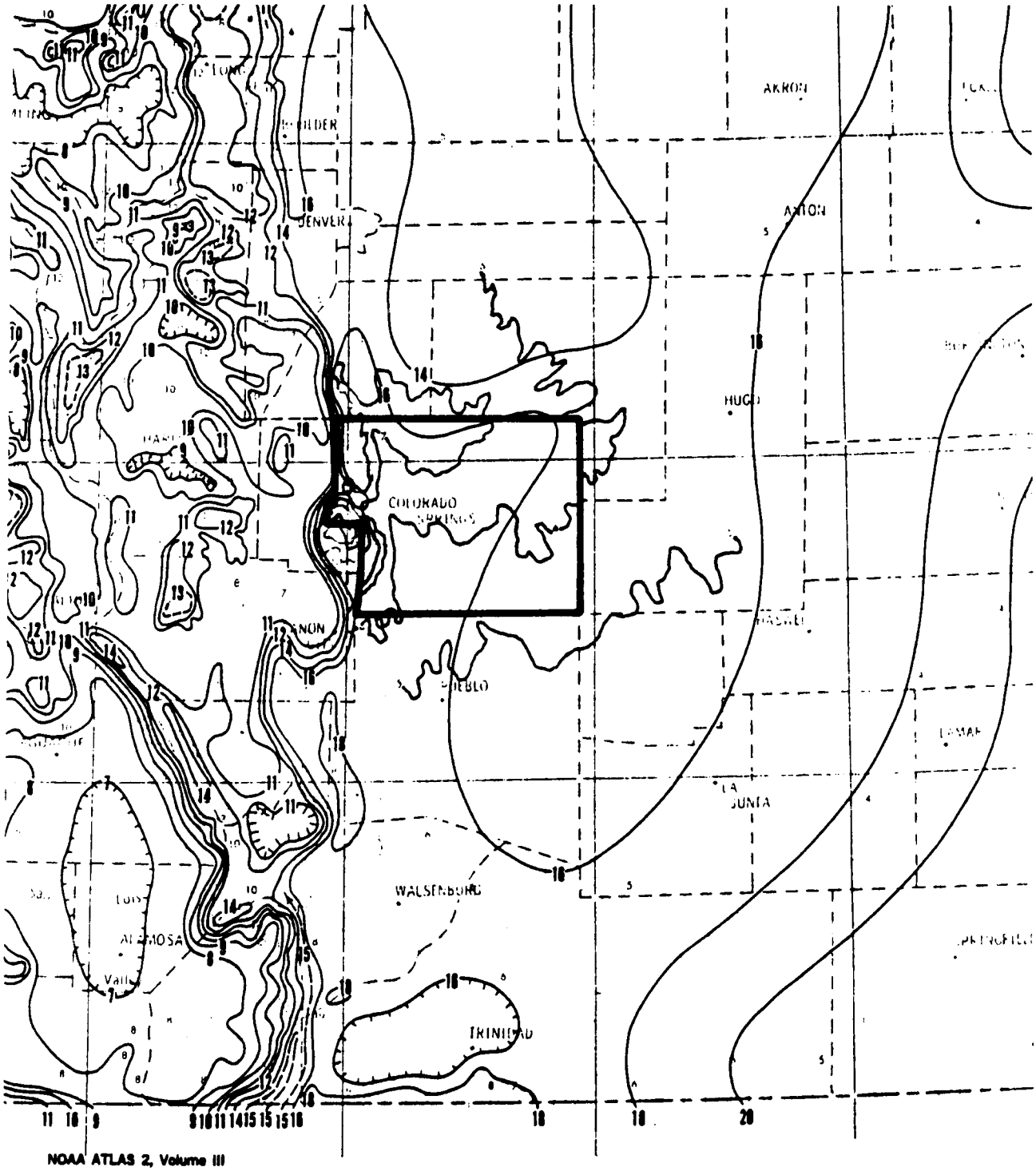
4. Plot the 1-hour values on the diagram in Figure 5-6, connect with a straight line, and note the value at the intersection of the plotted line and the 10-year return period line, which is the 10-year 1-hour rainfall.

5. The 2-hour rainfall for the 10-year or 100-year event, P, is the 1-hour rainfall times the appropriate cumulative percentage value listed in Figure 5-5a.

There are two storm distributions that will be utilized to determine the critical design storm. The critical design storm shall be defined as the storm distribution producing the greatest level of protection for flood conveyance and storage facilities. The two storm distributions are of two (2) and twenty-four (24) duration.

The recommended 24-hour time distribution of storm rainfall is the SCS Type IIa for the majority of El Paso County. The recommended 2-hour storm is similar to the Colorado Urban Hydrograph Procedure 2-hour storm distribution which encompasses the one-hour rainfall intensity. These distributions are shown on Figure 5-5a and 5-5b.

Selection of the design storm will depend on a number of factors including: 1) the type of structure (small dam, road culvert, storm drainage, etc.) 2) the particular component of a structure being designed (principal spillway, emergency spillway, etc.), and 3) the population and hazard potential. The design storm selection procedure contained in this manual represents a general procedure applicable to the various hydraulic elements of the stormwater management system. Final design of structures may require adherence to special design procedures and standards. The engineer shall pick the storm duration which is most critical to the application in terms of volume and peak flow. The selection must be approved by the City/County Engineer.



NOAA ATLAS 2, Volume III

Prepared by U.S. Department of Commerce
National Oceanic and Atmospheric Administration
National Weather Service, Office of Hydrology

Prepared for U.S. Department of Agriculture,
Soil Conservation Service, Engineering Division

ISOPLUVIALS OF 2-YR 6-HR PRECIPITATION IN TENTHS OF AN INCH

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Drainage Criteria Manual

NOTE :

Use only for derivation of 2 HR rainfall.
See procedure on page 5-14

Date
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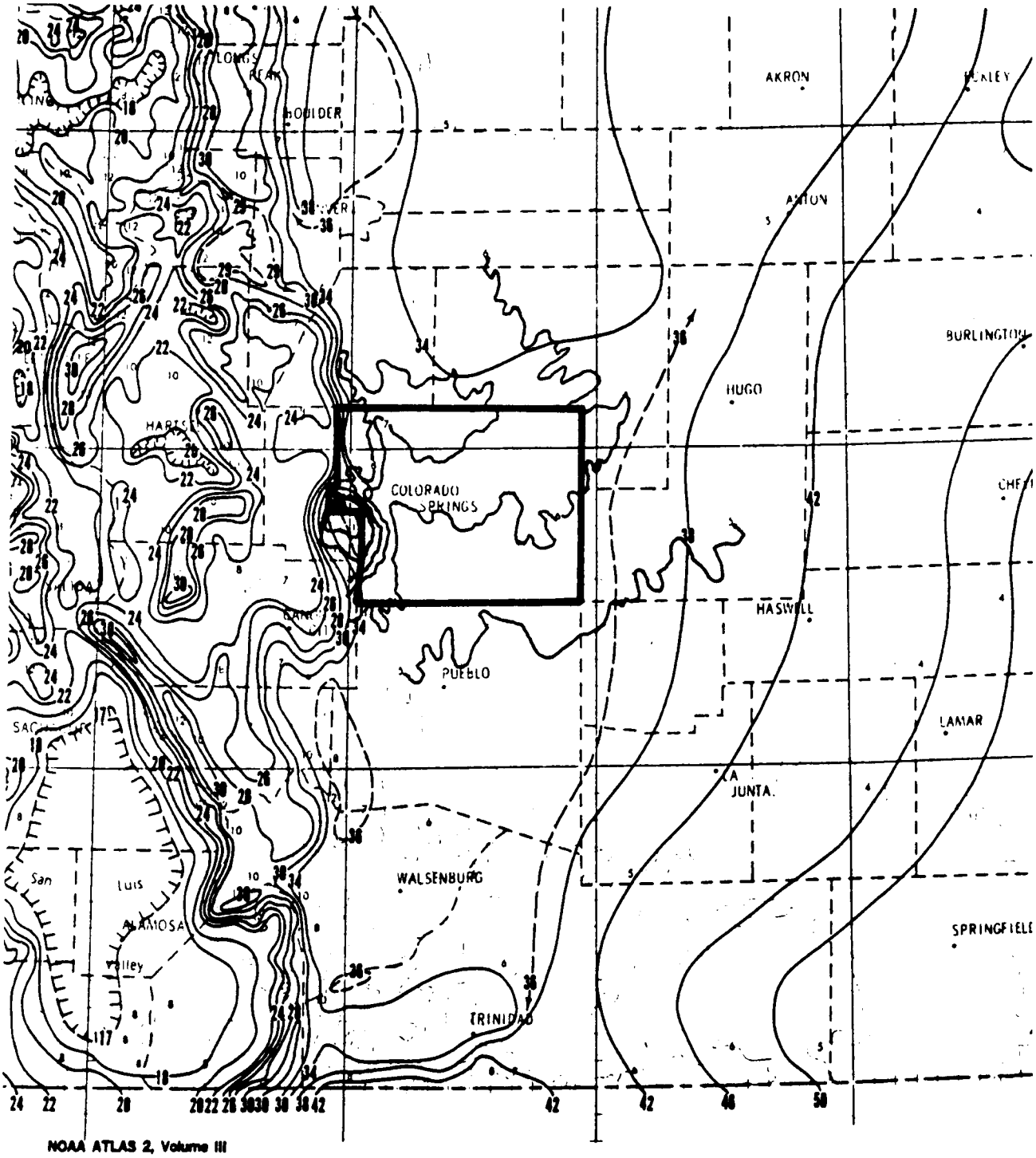
Figure
5-4a



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ISOPLUVIALS OF 100-YR 6-HR PRECIPITATION IN TENTHS OF AN INCH

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NOTE :

Use only for derivation of 2 HR rainfall.
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Date
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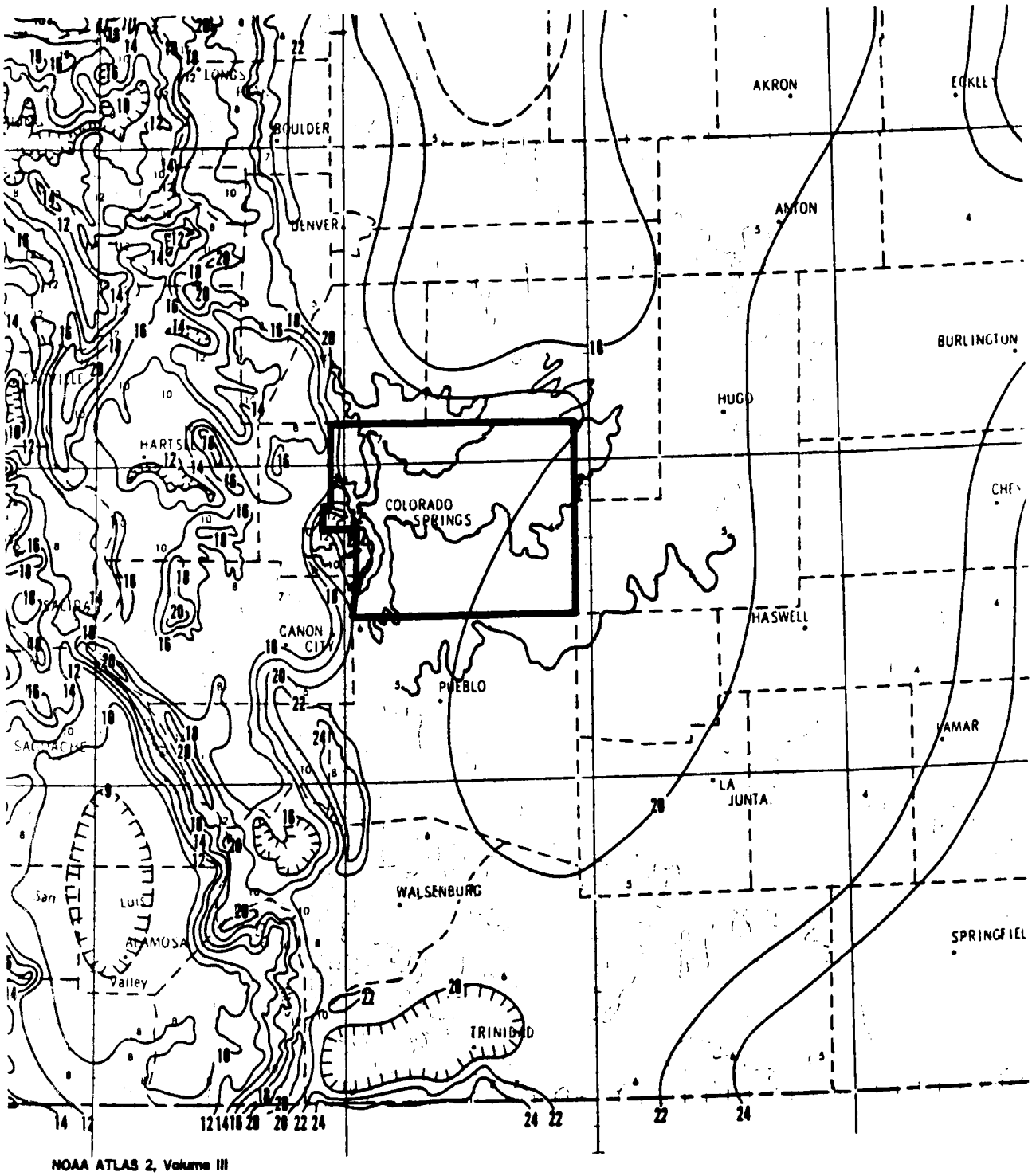
Figure
5-4b



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ISOPLUVIALS OF 2-YR 24-HR PRECIPITATION IN TENTHS OF AN INCH

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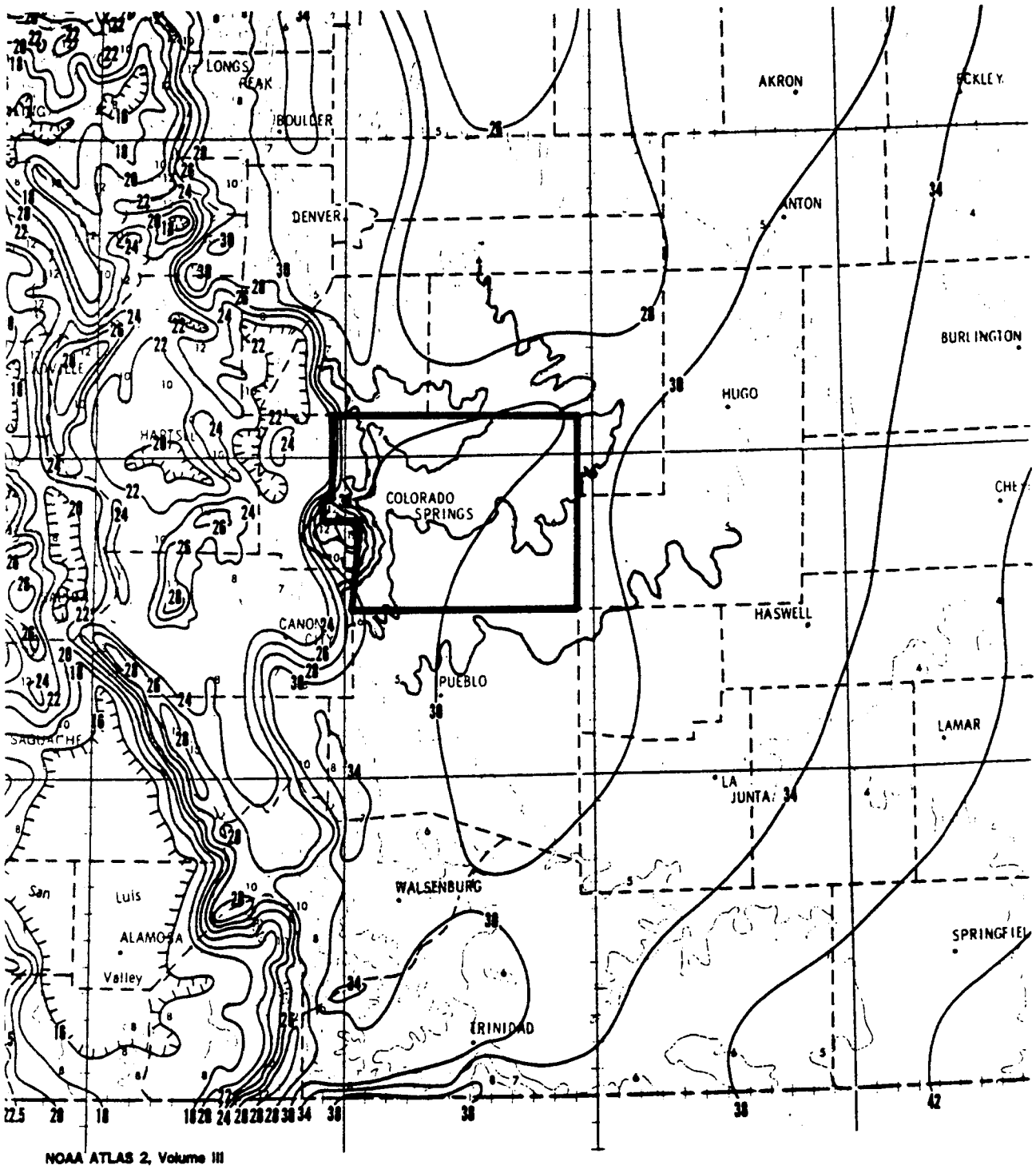
NOTE :

Use only for derivation of 2 HR rainfall.
See procedure on page 5-14

Date
OCT. 1987

Figure
5-4c



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ISOPLUVIALS OF 10-YR 24-HR PRECIPITATION IN TENTHS OF AN INCH

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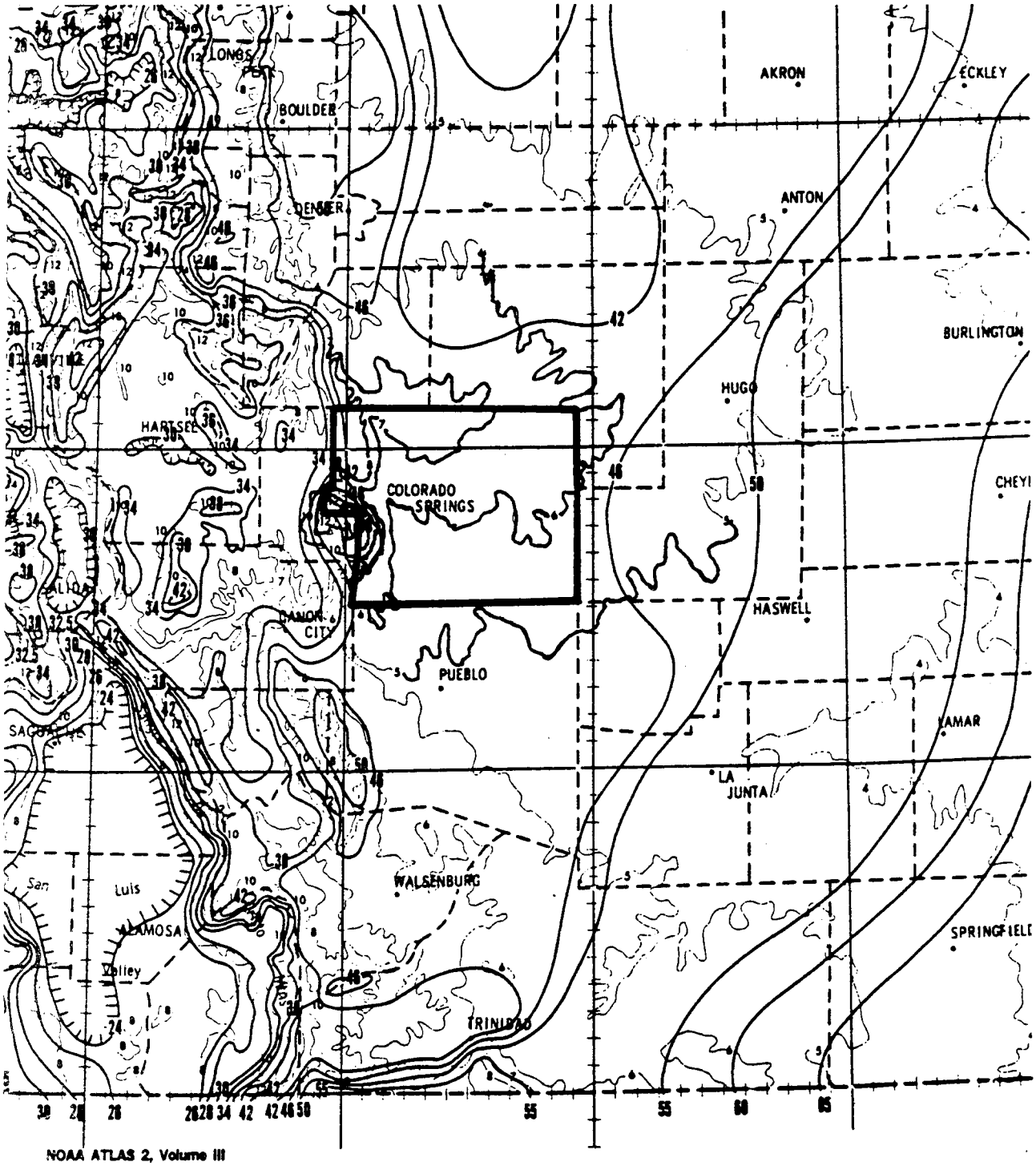
Figure
5-4d



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ISOPLUVIALS OF 100-YR 24-HR PRECIPITATION IN TENTHS OF AN INCH

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Figure
5-4e

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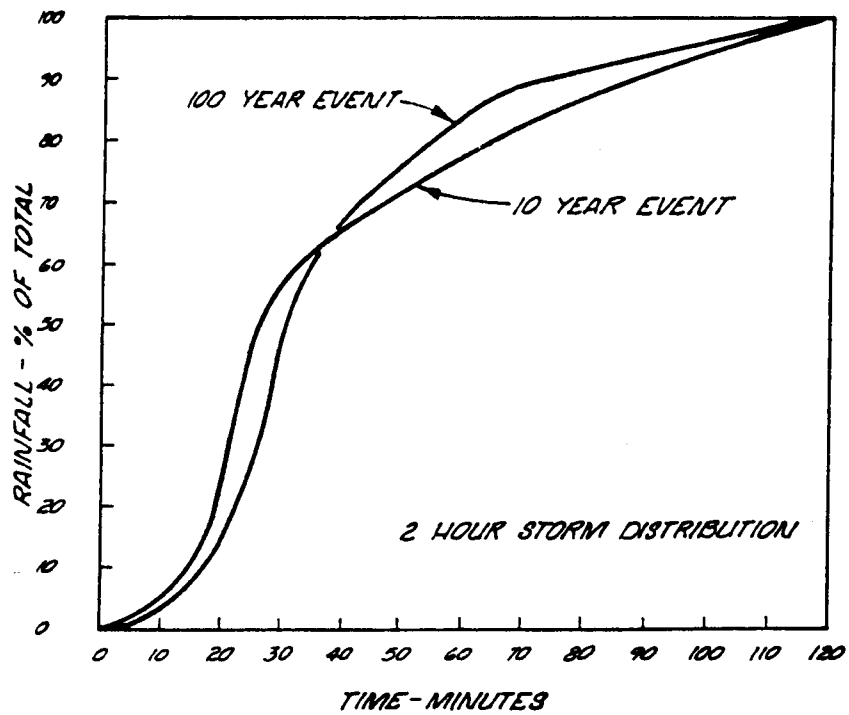

2-HR DESIGN STORM DISTRIBUTION

Time (Minutes)	Percent of 1-Hr Rainfall 10 Year	100 Year
5	2.0	1.0
10	3.7	3.0
15	8.2	4.6
20	15.0	8.0
25	25.0	14.0
30	12.0	25.0
35	5.6	14.0
40	4.3	8.0
45	3.8	6.2
50	3.2	5.0
55	3.2	4.0
60	3.2	4.0
65	3.2	4.0
70	3.2	2.0
75	3.2	2.0
80	2.5	1.2
85	1.9	1.2
90	1.9	1.2
95	1.9	1.2
100	1.9	1.2
105	1.9	1.2
110	1.9	1.2
115	1.7	1.2
120	1.3	1.2
	115.7	115.6

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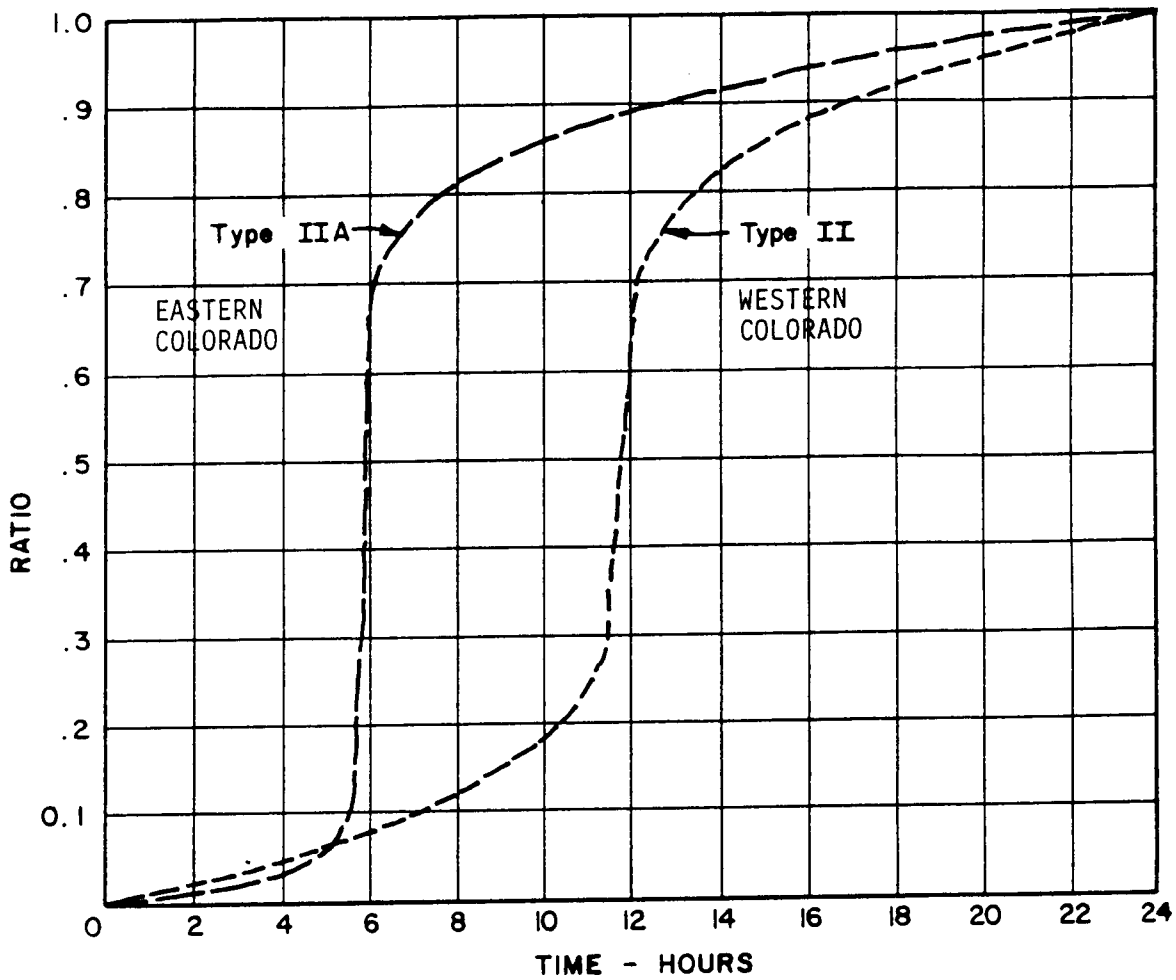
Figure
5-5a

Critical Storm Rainfall Distributions



RE: SCS

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24 HOUR RAINFALL DISTRIBUTIONS

Date
OCT. 1987

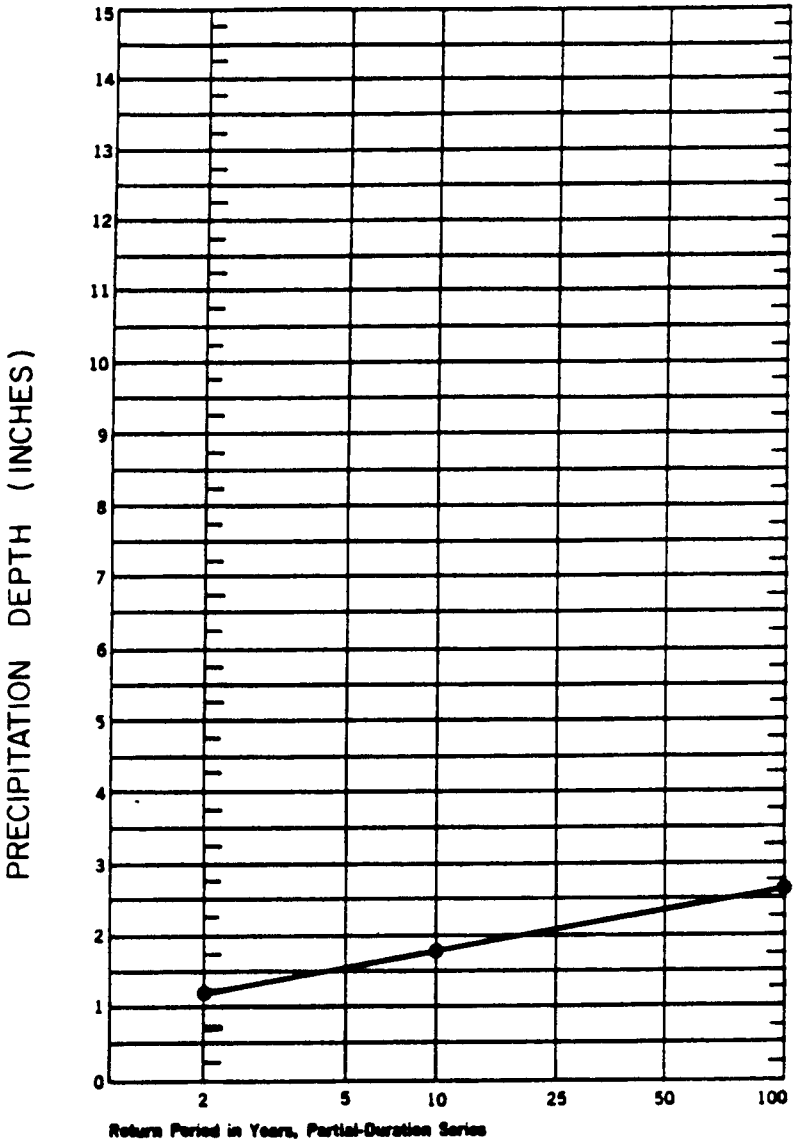
Figure
5-5b

TABLE 5-2
STANDARD SCS 24-HOUR
TYPE II RAINFALL DISTRIBUTION
FOR TR-20 INPUT

Minute Intervals				
15	30	45	60	Hour
0.0020	0.0050	0.0080	0.0110	1
0.0140	0.0170	0.0200	0.0230	2
0.0260	0.0290	0.0320	0.0350	3
0.0380	0.0410	0.0440	0.0480	4
0.0520	0.0560	0.0600	0.0640	5
0.0680	0.0720	0.0760	0.0800	6
0.0850	0.0900	0.0950	0.1000	7
0.1050	0.1100	0.1150	0.1200	8
0.1260	0.1330	0.1400	0.1470	9
0.1550	0.1630	0.1720	0.1810	10
0.1910	0.2030	0.2180	0.2360	11
0.2570	0.2830	0.3870	0.6630	12
0.7070	0.7350	0.7580	0.7760	13
0.7910	0.8040	0.8150	0.8250	14
0.8340	0.8420	0.8490	0.8560	15
0.8630	0.8690	0.8750	0.8810	16
0.8870	0.8930	0.8980	0.9030	17
0.9080	0.9130	0.9180	0.9220	18
0.9260	0.9300	0.9340	0.9380	19
0.9420	0.9460	0.9500	0.9530	20
0.9560	0.9590	0.9620	0.9650	21
0.9680	0.9710	0.9740	0.9770	22
0.9800	0.9830	0.9860	0.9890	23
0.9920	0.9950	0.9980	1.0000	24

TABLE 5-3
STANDARD SCS 24-HOUR
TYPE IIA CUMULATIVE RAINFALL DISTRIBUTION
FOR TR-20 INPUT

Minute Intervals				
15	30	45	60	Hour
0.0005	0.0015	0.0030	0.0045	1
0.0060	0.0080	0.0100	0.0120	2
0.0143	0.0165	0.0188	0.0210	3
0.0233	0.0255	0.0278	0.0320	4
0.0390	0.0460	0.0530	0.0600	5
0.0750	0.1000	0.4000	0.7000	6
0.7250	0.7500	0.7650	0.7800	7
0.7900	0.8000	0.8100	0.8200	8
0.8250	0.8300	0.8350	0.8400	9
0.8450	0.8500	0.8550	0.8600	10
0.8638	0.8675	0.8713	0.8750	11
0.8788	0.8825	0.8863	0.8900	12
0.8938	0.8975	0.9013	0.9050	13
0.9083	0.9115	0.9148	0.9180	14
0.9210	0.9240	0.9270	0.9300	15
0.9325	0.9350	0.9375	0.9400	16
0.9425	0.9450	0.9475	0.9500	17
0.9525	0.9550	0.9575	0.9600	18
0.9625	0.9650	0.9675	0.9700	19
0.9725	0.9750	0.9775	0.9800	20
0.9813	0.9825	0.9838	0.9850	21
0.9863	0.9875	0.9888	0.9900	22
0.9913	0.9925	0.9938	0.9950	23
0.9963	0.9975	0.9988	1.0000	24



EXAMPLE

2 yr. 1 hr rainfall (calculated) = 1.19"
100 yr. 1 hr rainfall (calculated) = 2.64"
10 yr. 1 hr rainfall (interpolated) = 1.78"

REFERENCE : NOAA Atlas 2, Volume 3 - Colorado

NOTE: This example is for Colorado Springs as indicated on the Isopluvials.

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The City of Colorado Springs / El Paso County
Drainage Criteria Manual

RAINFALL DEPTH - DURATION RELATIONSHIP

Date
OCT. 1987

Figure
5 - 6

5.3.4 Rainfall Excess

Rainfall excess values for the 24-hour design storm are determined from the United States Soils Conservation Service runoff curves for the accumulative rainfall amounts and appropriate runoff curve number (U.S. Department of the Interior, 1974). A runoff curve number (CN) represents the relative value of the basin hydrologic soil-cover complex as a direct runoff producer. A greater amount of direct runoff is expected from a storm for a basin with a higher runoff curve number than from one with a lower number.

Determination of a weighted CN for a given drainage basin runoff includes several factors. A curve number is assigned to each of four major soil groupings. Modifications can be made to account for other variables. The four hydrologic soil groups, according to their infiltration and transmission loss rates, are:

GROUP A <u>Low runoff potential.</u> Soils having <u>high infiltration rates</u> even when thoroughly wetted. These consist chiefly of deep, well to excessively drained sands or gravel. These soils have a high rate of water transmission in that water readily passes through them.

GROUP B Soils having <u>moderate infiltration rates</u> when thoroughly wetted. These consist chiefly of moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

GROUP C Soils having <u>slow infiltration rates</u> when thoroughly wetted. These consist chiefly of soils with a layer that impedes downward movement of water or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

GROUP D <u>High runoff potential</u>. Soils having <u>very slow infiltration rates</u> when thoroughly wetted. These consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over a nearly impervious material. These soils have a curve number for very slow rate of water transmission.

<u>Group A soils will not be allowed for use in the City of Colorado Springs/El Paso County in areas where overlot grading or fill operations have occurred</u>. Maps and other information showing soil series are available for viewing at local SCS offices. Refer to the Soil Survey of El Paso County, Colorado as prepared by the U.S. Department of Agriculture, Soil Conservation Service.

For design purposes, AMC II (THE NORMAL ANTECEDENT MOISTURE CONDITION) SHALL BE USED FOR DETERMINING RUNOFF FROM A 24-HOUR STORM. This practice is consistent with the recommendations and procedures outlined by the SCS in NEH-4 (SCS, 1985) and TR-55 (SCS, 1986). Appropriate CN values for various soil types and cover complexes, based on AMC II, are given in Figure 5-7 and Tables 5-4 and 5-5.

AMC III has been used for determing runoff from a 2-hour storm. This has the effect of increasing the runoff curve number according to the conversions in Table 5-6 and 5-7, representing the observed condition that peak flood discharges on the front range of Colorado often result from a series of consecutive-day rainfall occurrences creating saturation conditions, followed by a high intensity, short duration thunderstorm. The validity of this assumption is based on the fact that due to the mountain range and its diminishing effect on high-moisture atmospheric conditions, the moisture bearing capacity of the front range is low in the sense that the potential for high-intensity, long duration (i.e., 24-hour) storms on successive days is minimal, hence AMC II. The potential is much greater, however, for successive low volume rainfall events, hence AMC III for the short duration, high intensity storm. It is recommended that AMC III is considered only for basins approximately six square miles or less.

THE USE OF AMC III IS NOT RECOMMENDED FOR STORM WATER RUNOFF ANALYSIS AND DRAINAGE FACILITY DESIGN WITHIN THE CITY/COUNTY LIMITS. The Engineering Division is to be consulted prior to any use of AMC III. The AMC III data is provided herein for information only.

In the case of a diversity of soil and cover conditions within a given drainage basin, weighted CN can be determined in the following manner, similar to the composite C in the Rational Method.

$$C = \frac{\sum_{i=1}^{n} CN_i A_i}{A_t} \qquad (5\text{-}5)$$

where:

CN_i = individual CN corresponding to subarea i;

n = total number of CN's in drainage areas; and

A_i = area of subarea for which curve number is applicable

A_t = total drainage area for which weighted curve number is applicable;

CN = weighted CN

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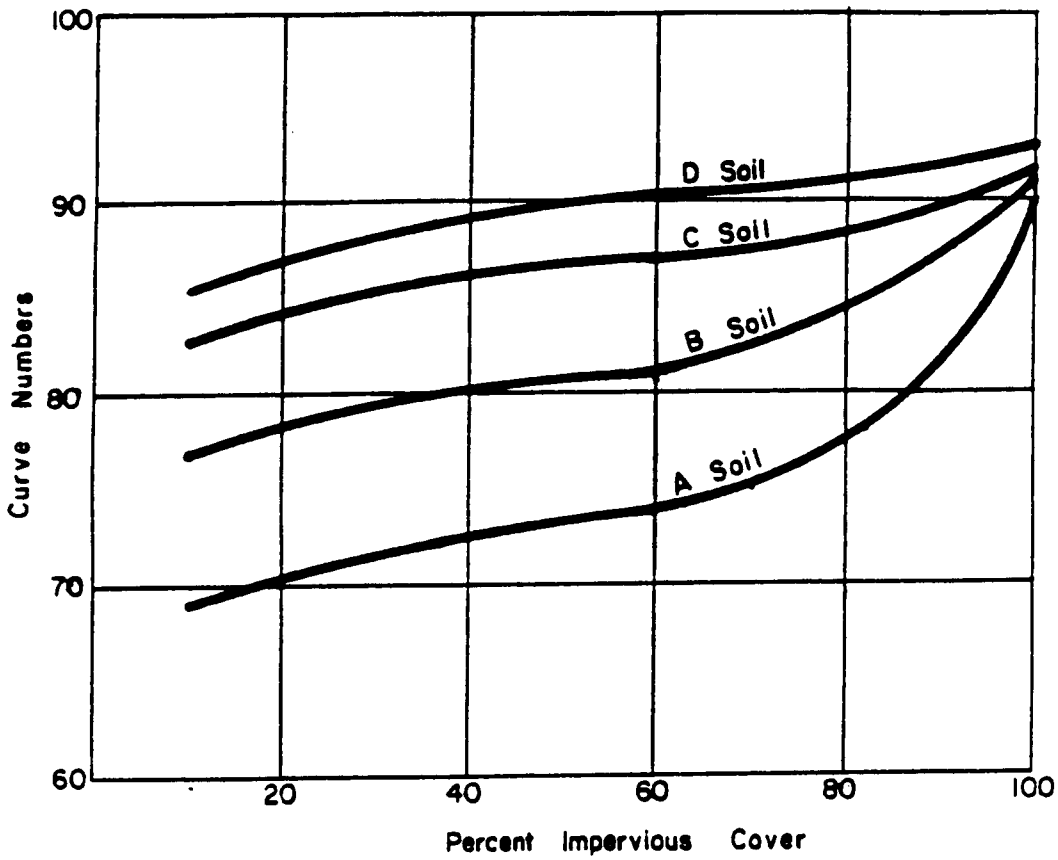

URBAN HYDROLOGIC SOIL COVER COMPLEX
& ASSOCIATED CURVE NUMBERS

REFERENCE : Pikes Peak Area Council of Governments Areawide Urban Runoff Control Manual

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Urban Hydrologic Soil Cover
Complex and Associated Curve Numbers

Date
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Figure
5-7

TABLE 5-4
RUNOFF CURVE NUMBERS FOR HYDROLOGIC SOIL COVER COMPLEXES - RURAL CONDITIONS
(Antecedent Moisture Condition II, and Ia = 0.2 S)
(From: U.S. Dept. of Agriculture, Soil Conservation Service, 1977)

Land Use	Cover Treatment or Practice	Hydrologic Condition	Runoff Curve Number by Hydrologic Soil Group A	B	C	D
Fallow	Straight Row	----	77	86	91	94
Row Crops	Straight Row	Poor	72	81	88	91
	Straight Row	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	Cont. & Terraced	Poor	66	74	80	82
	Cont. & Terraced	Good	62	71	78	81
Small Grain	Straight Row	Poor	65	76	84	88
	Straight Row	Good	63	75	83	87
	Contoured	Poor	63	74	82	85
	Contoured	Good	61	73	81	84
	Cont. & Terraced	Poor	61	72	79	82
	Cont. & Terraced	Good	59	70	78	81
Close-seeded legumes 1/ or rotation meadow	Straight Row	Poor	66	77	85	89
	Straight Row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
	Cont. & Terraced	Poor	63	73	80	83
	Cont. & Terraced	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		----	59	74	82	86
Roads (dirt) 2/		----	72	82	87	89
(hard surface) 2/		----	74	84	90	92

1/ Close-drilled or broadcast
2/ Including right-of-way

TABLE 5-5
RUNOFF CURVE NUMBERS FOR HYDROLOGIC SOIL COVER COMPLEXES - URBAN AND SUBURBAN CONDITIONS 1/
(Antecedent Moisture Condition II)
(From: U.S. Dept. of Agriculture, Soil Conservation Service, 1977)

Land Use		Hydrologic Soil Group A	B	C	D
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: 2/					
Acres per Dwelling Unit	Average % Impervious 3/				
1/8 acre or less	65	77*	85	90	92
1/4 acre	38	61*	75	83	87
1/3 acre	30	57*	72	81	86
1/2 acre	25	54*	70	80	85
1 acre	20	51*	68	79	84
Paved parking lots, roofs, driveways, etc.		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 the National Engineering Handbook (U.S. Dept. of Agriculture, Soil Conservation Service, 1972).
2/ 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.
3/ The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

* Not to be used wherever overlot grading or filling is to occur.

TABLE 5-6
RUNOFF CURVE NUMBERS FOR HYDROLOGIC SOIL COVER COMPLEXES - RURAL CONDITIONS
(Antecedent Moisture Condition III, and Ia = 0.2 S)
(From: U.S. Dept. of Agriculture, Soil Conservation Service, 1977)

NOTE: THIS TABLE TO BE USED FOR INFORMATION ONLY

Land Use	Cover Treatment or Practice	Hydrologic Condition	Runoff Curve Number by Hydrologic Soil Group A	B	C	D
Fallow	Straight Row	----	89	94	97	98
Row Crops	Straight Row	Poor	86	92	95	97
	Straight Row	Good	83	90	94	96
	Contoured	Poor	85	91	93	95
	Contoured	Good	82	88	92	94
	Cont. & Terraced	Poor	82	88	91	92
	Cont. & Terraced	Good	79	86	90	92
Small Grain	Straight Row	Poor	82	89	93	95
	Straight Row	Good	80	88	93	95
	Contoured	Poor	80	88	92	94
	Contoured	Good	78	87	92	93
	Cont. & Terraced	Poor	78	86	91	92
	Cont. & Terraced	Good	77	85	90	92
Close-seeded legumes 1/ or rotation meadow	Straight Row	Poor	82	89	94	96
	Straight Row	Good	76	86	92	94
	Contoured	Poor	81	88	93	94
	Contoured	Good	74	84	90	93
	Cont. & Terraced	Poor	80	87	91	93
	Cont. & Terraced	Good	70	83	89	91
Pasture or range		Poor	84	91	94	96
		Fair	69	84	91	93
		Good	59	78	88	91
	Contoured	Poor	67	83	92	95
	Contoured	Fair	64	77	88	93
	Contoured	Good	15	55	85	91
Meadow		Good	50	76	86	90
Woods		Poor	65	82	89	93
		Fair	56	78	87	91
		Good	43	74	85	89
Farmsteads		----	77	88	92	94
Roads (dirt) 2/		----	86	92	95	96
(hard surface) 2/		----	88	93	96	97

1/ Close-drilled or broadcast
2/ Including right-of-way

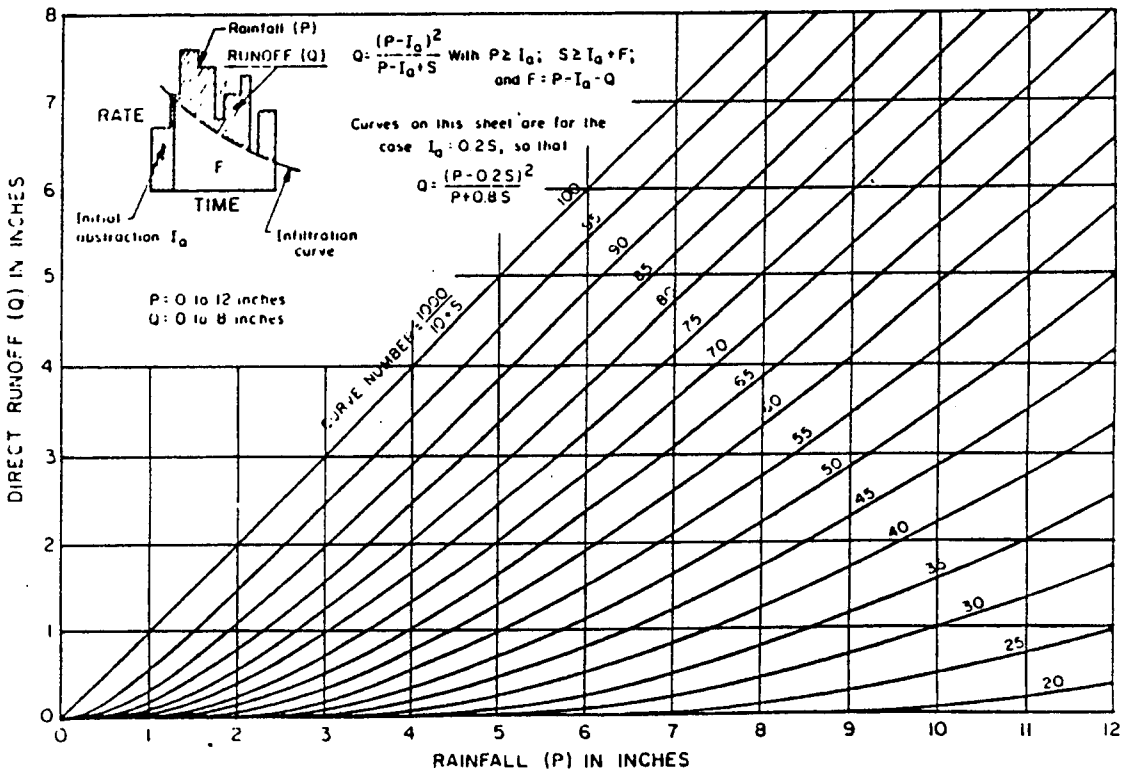
TABLE 5-7
RUNOFF CURVE NUMBERS FOR HYDROLOGIC SOIL COVER COMPLEXES - URBAN AND SUBURBAN CONDITIONS 1/
(Antecedent Moisture Condition III)
(From: U.S. Dept. of Agriculture, Soil Conservation Service, 1977)

NOTE: THIS TABLE TO BE USED FOR INFORMATION ONLY

Land Use		Hydrologic Soil Group A	B	C	D
Open spaces, lawns, parks, golf courses, cemeteries, etc.					
Good condition:	grass cover on 75% or more of the area	59*	78	88	91
Fair condition:	grass cover on 50% to 75% of the area	69*	84	91	93
Commercial and Business areas (85% Impervious)		96*	97	98	98
Industrial Districts 72% Impervious)		92*	95	97	98
Residential: 2/					
Acres per Dwelling Unit	Average % Impervious 3/				
1/8 acre or less	65	89*	94	96	97
1/4 acre	38	78*	88	93	95
1/3 acre	30	75*	86	92	94
1/2 acre	25	73*	85	91	94
1 acre	20	70*	84	91	93
Paved parking lots, roofs, driveways, etc.		99	99	99	99
Streets and Roads:					
paved with curbs and storm sewers		99	99	99	99
gravel		89*	94	96	97
dirt		86*	92	95	96

1/ For a more detailed description of agricultural land use curve numbers, refer to the National Engineering Handbook (U.S. Dept. of Agriculture, Soil Conservation Service, 1972).
2/ 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.
3/ The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

* Not to be used wherever overlot grading or filling is to occur.



REFERENCE : Mockus, Victor; Estimating direct runoff amounts from storm rainfall: Central Technical Unit, October 1955, Soil Conservation Service

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Graphical Relationship Among Precipitation, Curve Numbers, and Direct Runoff

Date: OCT. 1987

Figure: 5-8

After a determination of the weighted CN for the contributing basin is made, Q (direct runoff in inches) can be related to the rainfall with equations developed by the SCS for typical Colorado drainage basins:

$$C = \frac{1000}{10+S} \qquad (5\text{-}6)$$

where:

S = maximum potential difference between precipitation and runoff in inches; and

CN = weighted curve number.

and:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad (5\text{-}7)$$

where:

P = storm rainfall in inches;

S = maximum potential difference between precipitation and runoff in inches; and

Q = direct runoff in inches.

Incremental rainfall excess values are determined based upon the above and then applied to the developed unit hydrograph for a given drainage basin, which is discussed in the following section, 5.3.5.

5.3.5 UNIT HYDROGRAPH DEVELOPMENT

The unit hydrograph is one of the most commonly used forms of runoff model, and is the hydrograph resulting from a unit volume of rainfall excess distributed uniformly over a basin at a uniform rate during a specified period of time. It represents the hydrological characteristics of a basin and its response to precipitation.

Unit hydrographs for City/County drainage basins shall be determined by utilizing the curvilinear unit hydrograph procedure developed by the SCS (U.S. Soil Conservation Service, 1972). This method is favored over the triangular unit hydrograph method since basin response can be modeled more effectively.

Values of unit hydrograph time to peak discharge from beginning of storm rainfall, t_p and unit hydrograph peak discharge, q_p, are computed from the following relationships:

$$t_p = \frac{D}{2} + 0.6\ t_c = 0.67\ t_c \text{ where } (D=0.133\ t_c) \qquad (5\text{-}8)$$

D = period of rainfall excess in hours;

t_c = runoff time of concentration in hours; and

t_p = time to peak flow rate from beginning of direct runoff in hours

and:

$$q_p = \frac{484AQ}{t_p} \qquad (5\text{-}9)$$

where:

A = drainage area in square miles;

Q = One (1.00) inch of rainfall excess, or direct runoff;

t_p = (as above) and T_p;

q_p = unit hydrograph peak discharge in cfs.

The computation of the coordinates for a basin unit hydrograph is then approximated for the time ratios and discharge ratios using the dimensionless unit hydrograph represented in Table 5-8.

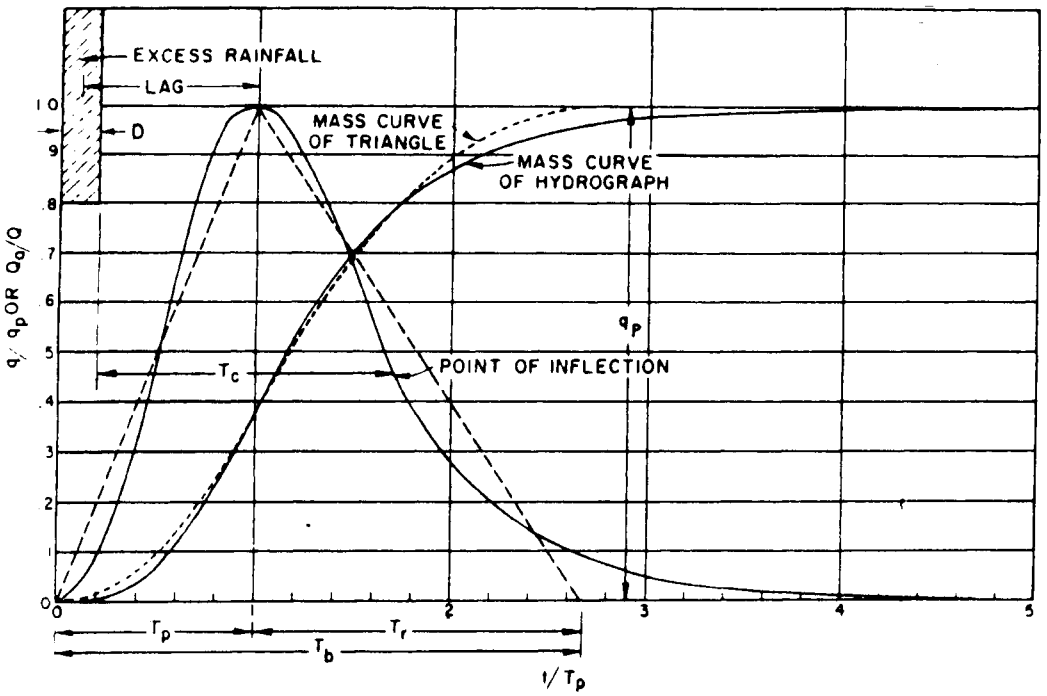
5.3.6 SCS Dimensionless Unit Hydrograph

The SCS dimensionless unit hydrograph was derived from a large number of natural unit hydrographs from watersheds varying widely in size and geographic locations. This dimensionless hydrograph (Figure 5-9) has its ordinate values expressed in a dimensionless ratio q/q_p or Q_a/Q and its abscissa values as t/T_p.

By expressing the coordinates of the unit hydrograph as ratios of the peak, and time to peak the dimensionless hydrograph can easily be modified by specific basin parameters to represent a unit hydrograph for the watershed. The ratios for the dimensionless unit hydrograph are given in Table 5-8. The curvilinear unit hydrograph has 37.5% of the total volume in the rising side which is represented by one unit of time and one unit of discharge. The SCS unit hydrograph procedure is demonstrated in Example 2.

TABLE 5-8
RATIOS FOR DIMENSIONLESS UNIT HYDROGRAPH AND MASS CURVE

Time Ratios (t/T_p)	Discharge Ratios (q/q_p)	Mass Curve Ratios (Qa/Q)
0.0	.000	.000
.1	.030	.001
.2	.100	.006
.3	.190	.012
.4	.310	.035
.5	.470	.065
.6	.660	.107
.7	.820	.163
.8	.930	.228
.9	.990	.300
1.0	1.000	.375
1.1	.990	.450
1.2	.930	.522
1.3	.860	.589
1.4	.780	.650
1.5	.680	.700
1.6	.560	.751
1.7	.460	.790
1.8	.390	.822
1.9	.330	.849
2.0	.280	.871
2.2	.207	.908
2.4	.147	.934
2.6	.107	.953
2.8	.077	.967
3.0	.055	.977
3.2	.040	.984
3.4	.029	.989
3.6	.021	.993
3.8	.015	.995
4.0	.011	.997
4.5	.005	.999
5.0	.000	1.000



REFERENCE : SCS National Engineering Handbook, Section 4 - Hydrology

Figure 5-9 Dimensionless curvilinear unit hydrograph and equivalent triangular hydrograph

Example 2

Determine the peak discharge resulting from the 10-year, 2-hour storm for the same basin represented in Example 1. The basin has a drainage area of 300 acres and is composed of seventy percent industrial (72% impervious) and 30 percent open space (fair condition). The soils within the basin consists of a B industrial and D open space hydrologic soil group.

1. Compute the basin weighted curve number. From Table 5-5 for a B and D Hydrologic Soil Group:

   Industrial District (72% Impervious) CN = 88
   Open space, fair conditions: CN = 84

   Weighted CN:

   CN = 0.7 (88) + 0.3 (84) = 86.8
   Use CN = 87

2. Time of Concentration and Lag

   Use the time of concentration computed in Example 1.

   Tc = 29.2 minutes

   Lag = 0.6 Tc
   L = 0.6 (29.2 minutes)
   L = 17.5 minutes

3. Compute the design storm total rainfall (P) for a 10-year, 2-hour storm.

   For a 10-year, 1-hour storm, from Figure 5-6.

   P10-1 = 1.8 inches

   A 10-year, 2-hour storm equals 115.7% of a 10-year, 1-hour storm.

   P10-2 - 1.157 (P10-1)
   P10-2 = 1.157 (1.8 inches) = 2.1 inches

4. Compute the Initial Abstraction (Ia)

   $S = \frac{1000}{CN} - 10$ CN=87 for AMC II condition

   $S = \frac{1000}{87} - 10 = 1.49$ inches

   Ia = 0.2 (S)
   Ia = 0.2(1.49)
   Ia = 0.30 inches

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5. Compute the Direct Runoff Q in inches from equation 5-7.

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

$$Q = \frac{(2.1 - 0.2(1.49))^2}{(2.1 + 0.8(1.49))} = 0.98 \text{ inches}$$

6. Compute the SCS unit hydrograph parameters

a) Duration of unit excess rainfall, D from equation 5-8:

D = 0.133 tc
D = 0.133 (29.2 minutes) = 3.9 minutes
Use D = 5 minutes as a minimum time increment.

b) Time to peak, Tp, again from equation 5-8

$$tp = \frac{D}{2} + 0.6\ tc$$

$$Tp = \frac{5 \text{ minutes}}{2} + 0.6\ (29.2 \text{ minutes}) \quad tp = 20 \text{ minutes}$$
or tp = 0.33 hours

Note: there is some flexibility in selecting a value for D. It should not be greater than 0.25 (Tp). However for basins small enough so that 0.25 (Tp) is less than 5 minutes, use the 5 minute increment as a minimum.

c) Unit hydrograph peak discharge (Qp)

Using Equation 5-9:

$$Qp = \frac{484\ A\ Q}{tp}$$

by definition Q = 1.0 inch
A = 300 acres or 0.47 square miles

$$Qp = \frac{484\ (0.47)(1.0)}{0.33 \text{ hours}} = 689 \text{ cfs/in direct runoff}$$

Table 5-8 presents the SCS curvilinear dimensionless unit hydrograph and mass curve.

7. Compute the Design Storm Distribution, Table 5-9; columns 1 and 2 are obtained from Figure 5-5a. The cumulative rainfall is based on a total rainfall depth of 2.05 inches over a 2 hour period.

Note that runoff does not occur until the cumulative rainfall exceeds the initial abstraction ($P>I_a$). At 20 minutes, the cumulative rainfall (P=0.525 in.) exceeds the initial abstraction (I_a=0.299 inches) and the direct runoff can be calculated from Eq. 5-7.

9/30/90

TABLE 5-9
DESIGN STORM CONSTRUCTION

TIME (Min)	RAINFALL DIST. (%)*	<--------P---------> INCREM. RAINFALL (In)	ACCUM. RAINFALL (In)	INITIAL ABSTRACT. (In)	<-------Q-------> ACCUM. RUNOFF (In)	INCREM. RUNOFF (In)
5	2.0	0.036	0.036	0.036	0.000	0.000
10	3.7	0.067	0.103	0.103	0.000	0.000
15	8.2	0.149	0.252	0.252	0.000	0.000
20	15.0	0.272	0.525	0.299	0.030	0.030
25	25.0	0.454	0.978	0.299	0.212	0.183
30	12.0	0.218	1.196	0.299	0.337	0.124
35	5.6	0.102	1.298	0.299	0.400	0.064
40	4.3	0.078	1.376	0.299	0.451	0.051
45	3.8	0.069	1.445	0.299	0.497	0.046
50	3.2	0.058	1.503	0.299	0.537	0.040
55	3.2	0.058	1.561	0.299	0.578	0.041
60	3.2	0.058	1.619	0.299	0.619	0.041
65	3.2	0.058	1.677	0.299	0.661	0.042
70	3.2	0.058	1.735	0.299	0.704	0.045
75	3.2	0.058	1.793	0.299	0.747	0.043
80	2.5	0.045	1.839	0.299	0.781	0.034
85	1.9	0.034	1.873	0.299	0.808	0.026
90	1.9	0.034	1.908	0.299	0.834	0.026
95	1.9	0.034	1.942	0.299	0.861	0.027
100	1.9	0.034	1.977	0.299	0.887	0.027
105	1.9	0.034	2.011	0.299	0.914	0.027
110	1.9	0.034	2.046	0.299	0.941	0.027
115	1.7	0.031	2.076	0.299	0.966	0.024
120	1.3	0.024	2.100	0.299	0.984	0.019
	115.7	2.100				0.984

* % of 1 hour rainfall,
P = 1.79

TABLE 5-10
UNIT HYDROGRAPH

TIME RATIOS (t/Tp)	t TIME (Min)	DISCHARGE RATIOS (q/qp)	q DISCHARGE (cfs)
0.1	2.0	0.030	20
0.2	4.0	0.100	68
0.3	6.0	0.190	130
0.4	8.0	0.310	211
0.5	10.0	0.470	320
0.6	12.0	0.660	450
0.7	14.0	0.820	559
0.8	16.0	0.930	634
0.9	18.0	0.990	675
1.0	20.0	1.000	682
1.1	22.0	0.990	675
1.2	24.0	0.930	634
1.3	26.0	0.860	586
1.4	28.0	0.780	532
1.5	30.0	0.680	464
1.6	32.0	0.560	382
1.7	34.0	0.460	314
1.8	36.0	0.390	266
1.9	38.0	0.330	225
2.0	40.0	0.280	191
2.1	42.0	0.244	166
2.2	44.0	0.207	141
2.3	46.0	0.177	121
2.4	48.0	0.147	100
2.5	50.1	0.127	87
2.6	52.1	0.107	73
2.7	54.1	0.092	63
2.8	56.1	0.077	52
2.9	58.1	0.066	45
3.0	60.1	0.055	37
3.1	62.1	0.048	32
3.2	64.1	0.040	27
3.3	66.1	0.035	24
3.4	68.1	0.029	20
3.5	70.1	0.025	17
3.6	72.1	0.021	14
3.7	74.1	0.018	12
3.8	76.1	0.015	10
3.9	78.1	0.013	9
4.0	80.1	0.011	7
4.1	82.1	0.010	7
4.2	84.1	0.009	6
4.3	86.1	0.007	5
4.4	88.1	0.006	4
4.5	90.1	0.005	3
4.6	92.1	0.004	3
4.7	94.1	0.003	2
4.8	96.1	0.002	1
4.9	98.1	0.001	1
5.0	100.1	0.000	0

8. Unit Hydrograph construction

Using the values of $\frac{t}{T_p}$ and $\frac{q}{q_p}$ given in table 5-8 and the computed values of T_p and q_p (T_p=20 min., q_p=689 cfs per in direct runoff), the unit hydrograph ordinates are computed (Table 5-10).

The unit hydrograph ordinates presented in Table 5-10 are plotted on graph paper from which 5-minute ordinates can be read. The five-minute unit hydrograph ordinates are presented in Table 5-11.

Table 5-11, Five-minute Unit Hydrograph Ordinates

Time (min)	q (cfs/in)
0	0
5	99
10	320
15	596
20	682
25	611
30	465
35	291
40	142
45	131
50	87
55	58
60	38
65	26
70	17
75	11
80	8
85	5
90	3
95	2
100	0
105	0
110	0

9. Hydrograph Construction

The final hydrograph is obtained by multiplying the thirty-minute duration unit hydrograph ordinates by the design storm rainfall values (direct runoff). The process of convoluting the runoff hydrograph is illustrated on Table 5-12. The peak discharge occurs at time of 20 minutes and equals 682 cfs.

5.3.7 Calibration

Calibration of the SCS Method is recommended to test for reasonableness of the flows and to match design points of Basin Master plans. Use of stream gaged data as discussed in Section

5.4.1 is the most desired approach to calibration. When this data is not available, methods outlined in Section 5.4.2 may be utilized within this range of application. Also, the Colorado Urban Hydrograph Procedure may offer a way to quickly check for comparable peak flows. Figures 5-11a, b, c, and d have also been provided for reasonable checks of 24-hour storm peak discharges.

Figure 5-11d is used with the SCS Graphic Method as explained in the publication Procedures for Determining Peak Flows in Colorado by the Soil Conservation Service, U.S. Department of Agriculture, March 1980. This method is used for a watershed where runoff can be represented by one curve number and is limited to to peak discharge distribution for a watershed where valley routing or subbasin hydrograph combinations is not required.

Hydrology for Drainage Basin Planning Studies which have been accepted by the City/County must be utilized by the engineer to establish hydrology for smaller sub-basins. This may require calibration of the hydrologic method used by the engineer to match the overall basin hydrology set in the Drainage Basin Planning Study. Proposed differences in hydrology must be approved by the City/County.

For additional information in the use and development of the hydrologic models and routing methods see the National Engineering Handbook, Hydrology, Section 4, August 1972 by the U.S. Department of Commerce, National Technical Information Service.

5.3.8 SCS Hydrograph Computer Program

A computer program is available to calculate hydrographs using the SCS Hydrograph procedure. This program is available for use on both large mainframe computers and on small personal computers.

TABLE 5-12

TIME (Min)	UNIT HYD GPH (cfs)	INCREMENTAL RUNOFF 0.000	0.000	0.000	0.030	0.183	0.124	0.064	0.051	0.046	0.040	0.041	0.041	0.042	0.043	0.043	INCREMENTAL RUNOFF 0.034	0.026	0.026	0.027	0.027	0.027	0.027	0.024	0.019	STORM HYD GPH (cfs)	TIME (Min)
5.0	99	0.00																								0.0	5
10.0	320	0.00	0.00																							0.0	10
15.0	596	0.00	0.00	0.00																						0.0	15
20.0	682	0.00	0.00	0.00	2.92																					2.9	20
25.0	611	0.00	0.00	0.00	9.47	18.04																				27.5	25
30.0	465	0.00	0.00	0.00	17.65	58.46	12.26																			88.4	30
35.0	291	0.00	0.00	0.00	20.19	108.92	39.75	6.28																		175.1	35
40.0	192	0.00	0.00	0.00	18.09	124.59	74.05	20.34	5.02																	242.1	40
45.0	131	0.00	0.00	0.00	13.76	111.63	84.70	37.89	16.27	4.57																260.8	45
50.0	87	0.00	0.00	0.00	8.61	84.92	75.89	43.34	30.31	14.81	3.94															261.8	50
55.0	58	0.00	0.00	0.00	5.67	53.11	57.73	38.83	34.67	27.59	12.76	4.01														234.4	55
60.0	38	0.00	0.00	0.00	3.89	35.01	36.11	29.54	31.07	31.55	23.77	13.00	4.08													200.0	60
65.0	26	0.00	0.00	0.00	2.57	24.01	23.80	18.48	23.63	28.27	27.18	24.22	13.23	4.15												189.6	65
70.0	17	0.00	0.00	0.00	1.71	15.89	16.32	12.18	14.78	21.51	24.36	27.71	24.65	13.45	4.21											176.8	70
75.0	11	0.00	0.00	0.00	1.12	10.58	10.80	8.35	9.74	13.45	18.53	24.82	28.20	25.05	13.65	4.27										168.6	75
80.0	8	0.00	0.00	0.00	0.76	6.89	7.19	5.53	6.68	8.87	11.59	18.88	25.26	28.66	25.43	13.84	3.38									143.0	80
85.0	5	0.00	0.00	0.00	0.51	4.66	4.69	3.68	4.42	6.08	7.64	11.81	19.22	25.68	29.09	25.79	10.94	2.59								156.8	85
90.0	3	0.00	0.00	0.00	0.34	3.13	3.17	2.40	2.94	4.02	5.24	7.79	12.02	19.53	26.07	29.50	20.38	8.39	2.61							147.5	90
95.0	2	0.00	0.00	0.00	0.22	2.07	2.13	1.62	1.92	2.68	3.47	5.34	7.92	12.22	19.83	26.43	23.32	15.62	8.44	2.62						135.9	95
100.0	0	0.00	0.00	0.00	0.16	1.38	1.41	1.09	1.30	1.75	2.31	3.53	5.43	8.05	12.40	20.11	20.89	17.87	15.73	8.50	2.64					124.6	100
105.0	0	0.00	0.00	0.00	0.10	1.00	0.94	0.72	0.87	1.18	1.50	2.35	3.60	5.52	8.18	12.57	15.89	16.01	18.00	15.84	8.56	2.66				115.5	105
110.0	0	0.00	0.00	0.00	0.05	0.63	0.68	0.48	0.58	0.79	1.02	1.53	2.39	3.65	5.61	8.29	9.94	12.18	16.12	18.12	15.94	8.61	2.67			109.3	110
115.0	0	0.00	0.00	0.00	0.00	0.32	0.43	0.35	0.38	0.52	0.68	1.04	1.56	2.43	3.71	5.68	6.55	7.62	12.27	16.23	18.23	16.04	8.66	2.40		105.1	115
120.0	0	0.00	0.00	0.00	0.00	0.01	0.22	0.22	0.28	0.35	0.45	0.70	1.06	1.59	2.47	3.76	4.49	5.02	7.67	12.35	16.34	18.35	16.14	7.79	1.85	101.1	120
125.0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.11	0.18	0.25	0.30	0.46	0.71	1.07	1.61	2.51	2.97	3.44	5.06	7.72	12.43	16.44	18.46	14.52	5.98	94.2	125
130.0	0		0.00	0.00	0.00	0.00	0.00	0.00	0.09	0.16	0.22	0.31	0.47	0.72	1.09	1.63	1.98	2.28	3.47	5.09	7.77	12.51	16.54	16.60	11.15	82.1	130
135.0	0			0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.14	0.22	0.31	0.48	0.73	1.10	1.29	1.52	2.29	3.49	5.12	7.82	12.58	14.88	12.75	64.8	135
140.0	0				0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.14	0.23	0.32	0.48	0.74	0.87	0.99	1.53	2.31	3.51	5.16	7.87	11.32	11.43	47.0	140
145.0	0					0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.14	0.23	0.32	0.49	0.59	0.67	1.00	1.54	2.33	3.54	5.19	7.08	8.69	31.9	145
150.0	0						0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.14	0.23	0.33	0.39	0.45	0.67	1.00	1.55	2.34	3.56	4.67	5.44	20.8	150
155.0	0							0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.15	0.24	0.26	0.30	0.45	0.68	1.01	1.56	2.35	3.20	3.50	13.8	155
160.0	0								0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.15	0.19	0.20	0.30	0.46	0.68	1.02	1.57	2.12	2.46	9.2	160
165.0	0									0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.12	0.14	0.20	0.30	0.46	0.69	1.02	1.41	1.63	6.0	165
170.0	0										0.00	0.00	0.00	0.00	0.00	0.00	0.06	0.09	0.14	0.20	0.30	0.46	0.69	0.92	1.00	4.0	170
175.0	0											0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.09	0.15	0.20	0.30	0.46	0.62	0.71	2.6	175
180.0	0												0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.09	0.15	0.20	0.31	0.42	0.48	1.7	180
185.0	0													0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.09	0.15	0.20	0.28	0.32	1.1	185
190.0	0														0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.09	0.15	0.18	0.21	0.7	190
195.0	0															0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.09	0.13	0.14	0.4	195
200.0	0																0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.08	0.10	0.2	200
205.0	0																	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.06	0.1	205
210.0	0																		0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.0	210
215.0	0																			0.00	0.00	0.00	0.00	0.00	0.00	0.0	215
220.0	0																				0.00	0.00	0.00	0.00	0.00	0.0	220
225.0	0																					0.00	0.00	0.00	0.00	0.0	225
230.0																							0.00	0.00	0.00		
																								0.00	0.00		
																									0.00		

Peak Rates of Discharge for Small Watersheds

(24 Hour Type IIA Storm Distribution).

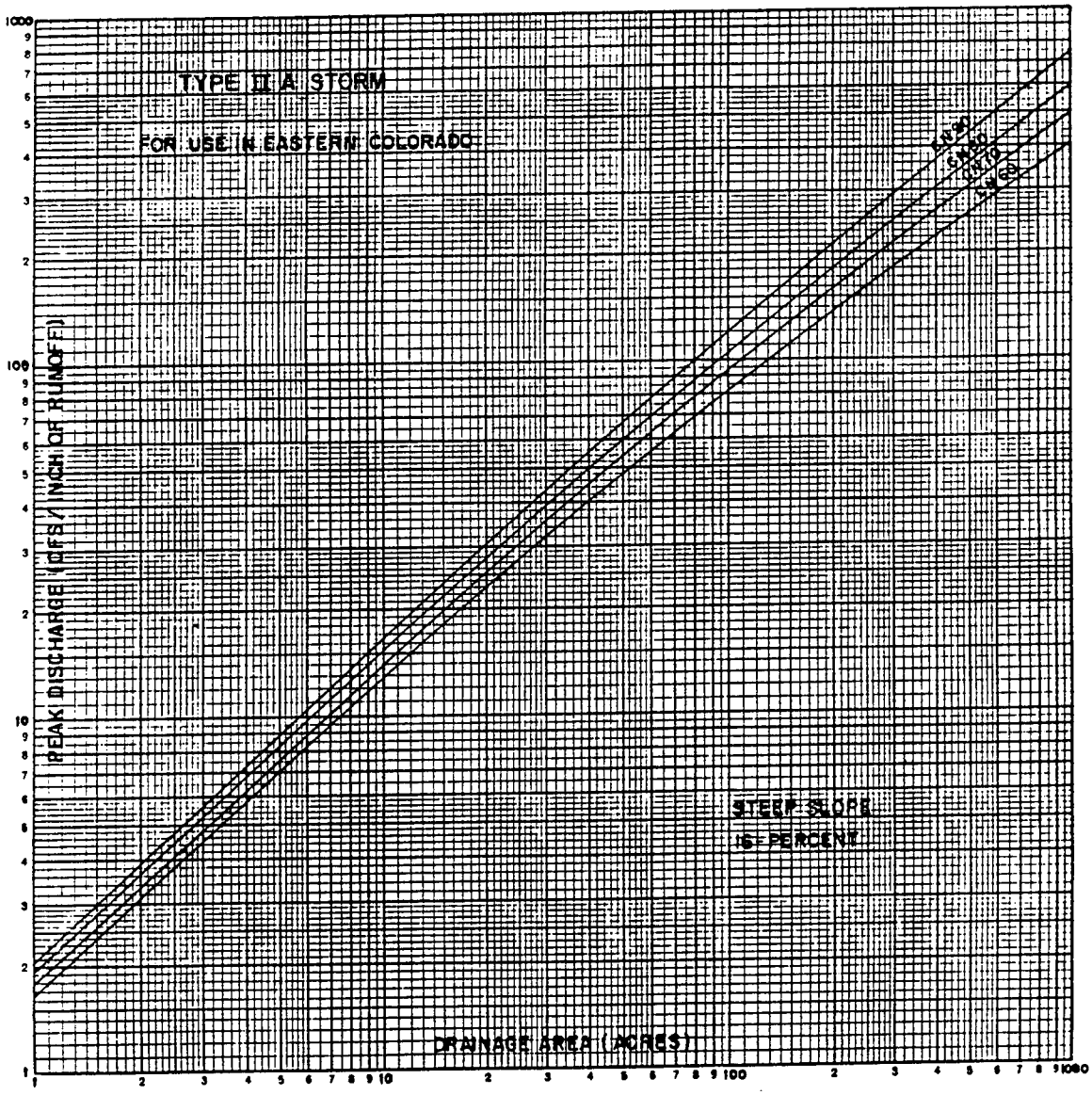

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Figure
5-11a

HDR Infrastructure, Inc.
A Centerra Company

Peak Rates of Discharge for Small Watersheds
(24 Hour Type IIA Storm Distribution).

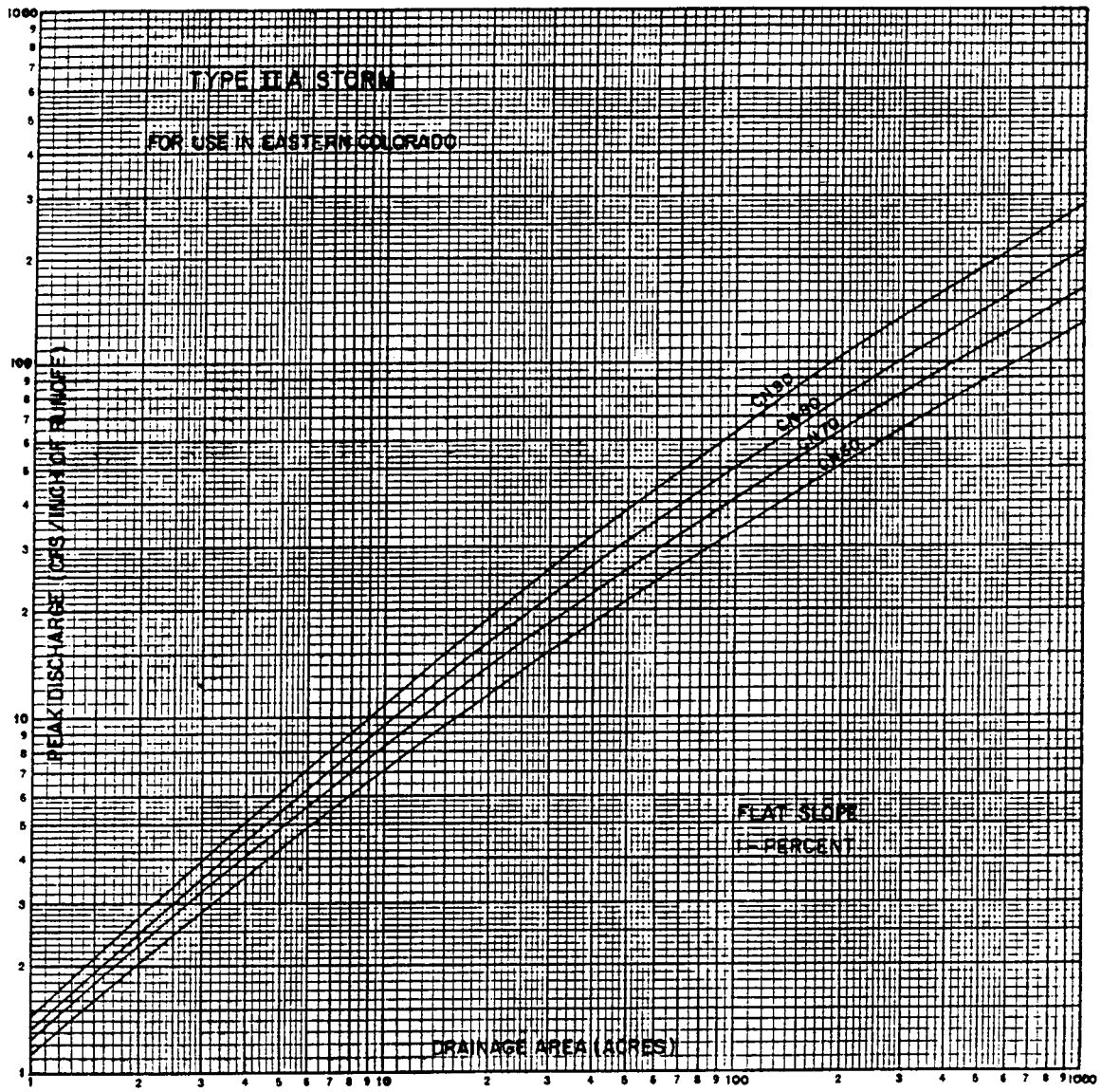

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Figure
5-11b

HDR Infrastructure, Inc.
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Peak Rates of Discharge for Small Watersheds

(24 Hour Type IIA Storm Distribution).

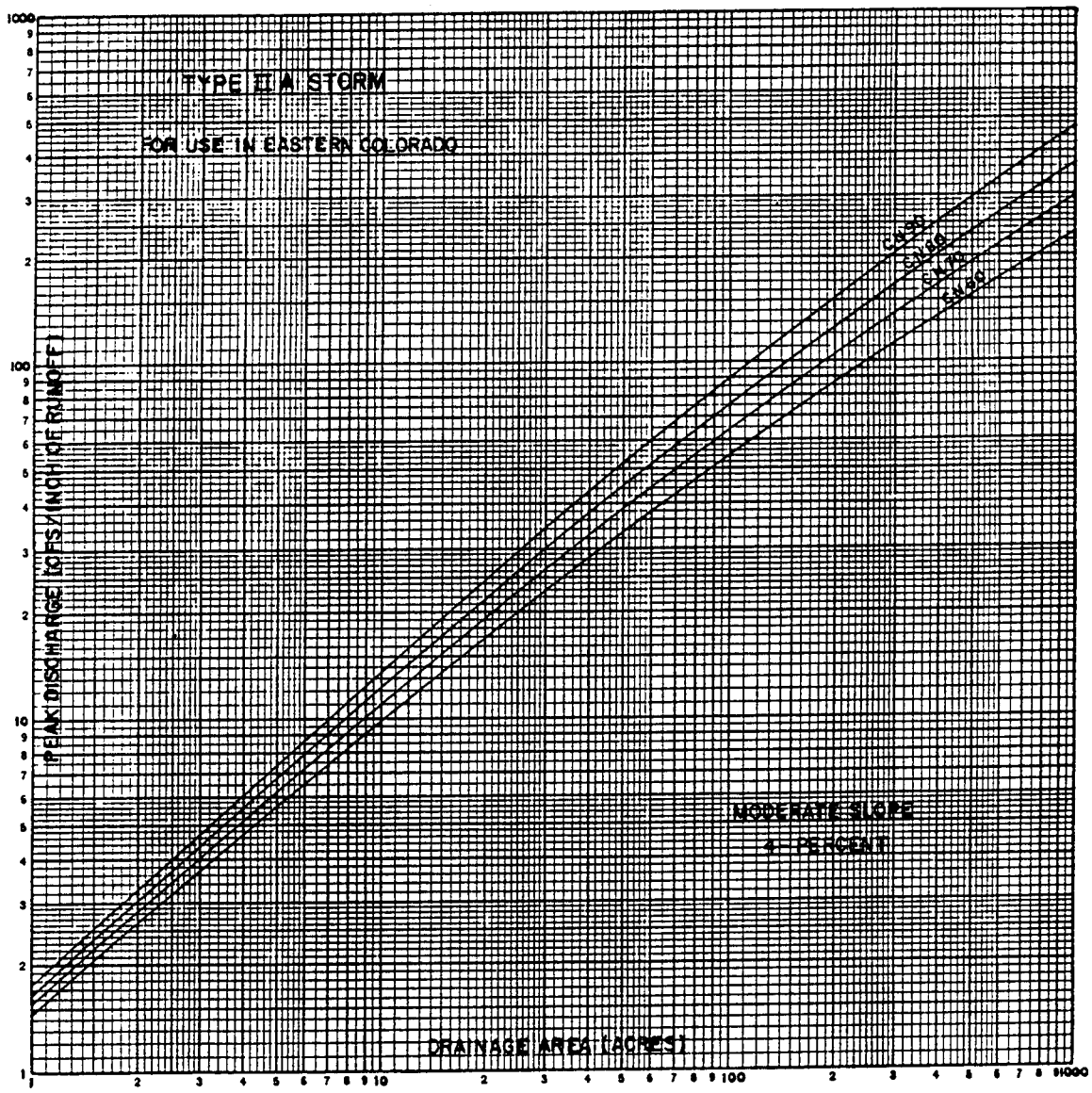

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Figure
5-11c

Peak Discharge in
csm Per inch of Runoff
Versus
Time of Concentration, Tc
Type II Storm Distribution, 24 Hour
Type II-A Storm Distribution, 24 Hour

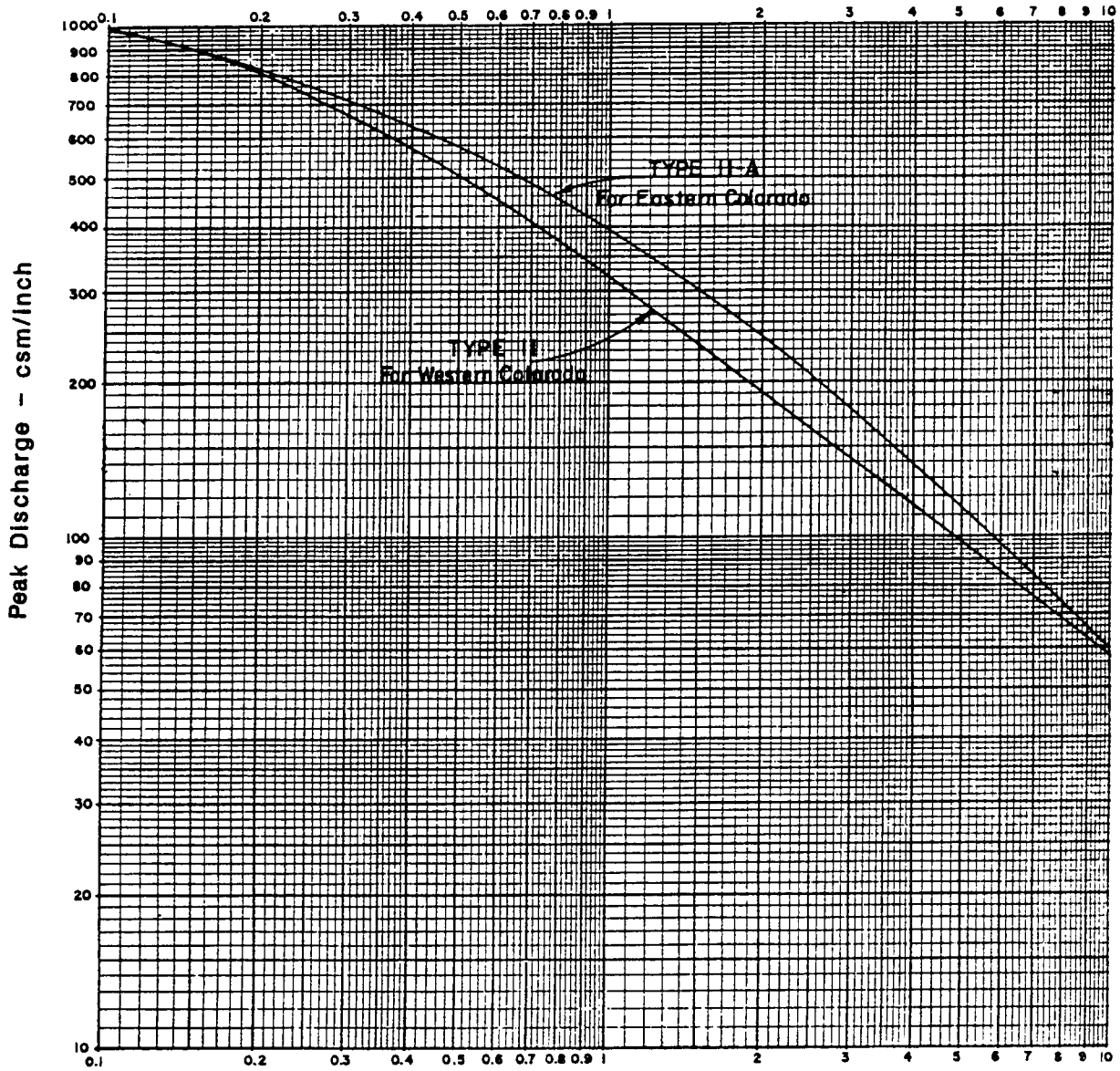

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Figure
5-11d

5.4 Statistical Analysis of Streamflow

A statistical or regional approach appears to be most appropriate for those areas in El Paso County where peak flows are from basins larger than 10 square miles and from basins where peak flows occur from seasonal snowmelt. When large basins are expected to undergo significant urbanization, computer simulation techniques should be utilized.

For these larger drainage areas, the experienced hydrologist may prefer to estimate design flood flows from long-term records of discharge, since results are more dependable than from using the Rational Method, unit hydrograph method, or other synthetic methods.

In urban hydrology, the preferred statistical approach may not be applicable. Key reasons for this include the almost total lack of adequate runoff records in urban areas, the effects of rapid urbanization, and because records from basins which have been gaged for a satisfactory period usually represent undeveloped conditions. Once an area undergoes urbanization, the gage records that represent natural conditions no longer directly apply. Thus, use of the computer simulation models become more appropriate and will generally be necessary.

The methods discussed herein for statistical and regional analyses should be used only by experienced hydrologists with adequate knowledge of the area. These regional methods can greatly overestimate or underestimate the runoff potential of a given drainage basin, hence a comparison with other hydrologic procedures or information in the immediate area must be made in order to produce reliable results.

5.4.1 Flood Frequency Determination

The statistical analysis approach to streamflow has its greatest applicability to gaged natural streams with similar hydrologic characteristics, the basins of which will remain in a natural state. Such streams include those with large basins where the urbanization effect on runoff will be negligible, and on small streams where the basin primarily consists of land not developable or not anticipated to develop.

In determining the size of flood peaks statistically, i.e., the probability of extreme flood discharges, the logic involved is that nature over a period of years has defined a flood magnitude-frequency relationship that can be derived by study of actual flood occurrences. A period of record for a particular river/stream where the floods have been measured and recorded is considered to be a representative period, and floods have been measured and recorded is considered to be a representative period, and floods that occurred during this period can be assumed to occur in a similar future period, that is, the period may be expected to repeat itself.

The purpose of statistical flood analysis is to use the recorded runoff events for a given period of record as a means of extrapolating to a longer period of time. For a 10-year period, the largest record flood is generally considered to have a recurrence interval of about 10 years. For any given year, the probability of a flood of any given frequency occurring in that year is the same as the probability of it happening in any other year. Thus, the 100-year flood has a one (1) percent chance of being equalled or exceeded in any given year.

The statistical procedure applicable to El Paso County is the standard method for the determination of peak flows as described by the U.S. Water Resources Council (1981). This procedure utilizes the Log-pearson Type III frequency distribution. Any independent statistical analysis of records in the City/County should, for the sake of consistency, follow the procedure outlined by the U.S. Water Resources Council. Discussion on a multi-disciplinary approach to evaluating the flood hydrology of foothill streams in Colorado (studied and reported on by the USGS in 1982), to evaluate floods from rainfall, snowmelt and or a combination of rain on snow. The approach is based upon the flood frequency determination of recorded runoff data.

A listing of stream-gaging stations in the City/County is presented in Table 5-13.

5.4.2 Regional Streamflow Analysis

The concept of regional streamflow analysis is based on developed multiple regression relationships. Once these relationships have been developed for gaged drainage basins, they can be utilized for nearby ungaged basins. Gaged basins must be selected carefully since their land-use characteristics, topography, shape and precipitation patterns must be comparable to the ungaged areas in which the regression relationship would be used. The method can be used for seasonal runoff as well as annual flood runoff volumes and peak flood flows. In El Paso County, the regional approach can also be utilized for estimating snowmelt runoff.

There are two specific recommended approaches to regional analysis for use in the City/County. The first is termed the Technical Manual 1 - Colorado Water Conservation Board Method; the second is a method recently published by the USGS in WRI-III. Technical Manual 1 may be used for rural basins greater than 100 square miles. The USGS WRI-III method may be used for rural basins from 0.3 to 20 square miles.

The TM No. 1 Method - The Colorado Water Conservation Board (1976) in cooperation with USGS, published Technical Manual 1 for estimating flood characteristics of natural-flow streams in Colorado. This manual presents methods for estimating peak-flow data for the 10-, 50-, 100- and 500-year floods at sites on natural-flow streams in Colorado. However, the following limitations need to be observed when using the methods described.

TABLE 5-13
USGS STREAM GAGING STATIONS FOR ALL STATIONS

USGA GAGE NO.	GAGE LOCATION	PERIOD OF RECORD
07099215	Turkey Creek near Fountain, CO	1978-
07099220	Little Turkey Creek near Fountain CO	1978-
07100000	N. Catamount Creek near Green Mountain Falls	1935-50
07100500	South Cascade Creek at Cascade CO	1935-50
07101000	Lion Creek near Halfway, CO	1908-50
07101500	Sheep Creek near Halfway CO	1908-50
07102000	South Ruxton Creek at Halfway, CO	1906-30
07102500	Cabin Creek near Halfway, CO	1906-30
07103000	Fountain Creek at Manitou, CO	1926
07103500	Sutherland Creek near Manitou, CO	1918-30
07103700	Fountain Creek near Colorado Springs, CO	1958-
07103747	Monument Creek at Palmer Lake, CO	1977-
07103750	Monument Creek at Monument, CO	1976-77
07103780	Monument Creek above N Gate Blvd. at USAF Academy, CO	1985-
07103800	West Monument Creek at USAF Academy, CO	1970-
07103900	West Monument Creek near Pikeview, CO	1957-70
07103950	Kettle Creek near Black Forest, CO	1976-85
07103990	Cottonwood Creek at Mouth at Pikeview, CO	1985-
07104000	Monument Creek at Pikeview CO	1938-49, 1976-
07104500	Templeton Gap Floodway at Colorado Springs, CO	1951-81
07105000	Bear Creek near Colorado Springs, CO	1918-30
07105500	Fountain Creek near Colorado Springs, CO	1921-24, 1976-
07105780	B Ditch Drain near Security, CO	1981-
07105800	Fountain Creek at Security, CO	1964-
07105820	Clover Ditch near Widefield, CO	1981-
07105900	Jimmy Camp Creek at Fountain, CO	1976-
07105920	Little Fountain Creek above Keaton Reservoir near Fort Carson, CO	1978-
07105924	Womack Ditch near Fort Carson, CO	1978-
07105928	Little Fountain Creek near Fort Carson CO	1978-
07105940	Little Fountain Creek near Fountain CO	1978-
07105945	Rock Creek above Fort Carson Reservation, CO	1978-
07105950	Rock Creek near Fort Carson, CO	1978-
07106000	Fountain Creek near Fountain, CO	1938-54, 1985
06757600	Kiowa Creek at K-79 Res near Eastonville, CO	1955-1965

1) The methods are not applicable to urban areas unless the effects of urbanization on flood characteristics are insignificant. An example of this condition would be a large stream flowing through an urban area.

2) The methods are not applicable to streams where manmade works, such as improved channels, flood-detention structures, storage reservoirs, and major diversion structures, have a significant effect on flood discharges or depths. For these conditions, open-channel hydraulic studies or stream-system studies will be required to evaluate the flood characteristics.

3) The estimating techniques are least reliable for sites on streams in mixed-population flood areas. Flood-estimating techniques for these streams need to be studied in more detail, but an interim estimating technique is presented.

4) The applicability of regression equations for computing flood characteristics at ungaged sites having basin and climatic parameters outside of the ranges listed in the tables in this report, is untested and, therefore, not recommended. Technical Manual 1 shall not be used for basins less than 100 square miles.

5) The flood-depth estimating equations shall not be used in reaches of streams affected by backwater from downstream structures.

Technical Manual 1 should not be used without careful comparison to other methods such as statistical methods, computer simulation,and other techniques. Experience has indicated that the use of Technical Manual 1 can result in an overestimation of flood flows for small basins by up to 10 times the value obtained using typical rainfall-runoff techniques and can underestimate by up to 10 times the value obtained using typical rainfall-runoff techniques when used on mountainous basins.

USGS Paper WRI-III

A new regionalized methodology from the USGS is contained in the publication Techniques for Estimated Regional Flood Characteristics of Small Rural Watersheds in the Plains Region of Eastern Colorado, by Russell K. Livingston and Donald L. Minojes, WRI 87-4094. This method has application for rural basins with areas ranging from 0.3 to 20 square miles. This publication should be utilized with caution until verification of acceptability by the City/County.

5.4.3 Flood Plain Studies in City/County

The U.S. Army Corps of Engineers, the Federal Insurance Administration, the USGS, SCS, and the Colorado Water Conservation Board have published various flood plain information reports and flood insurance studies on major drainages in urbanizing areas of El Paso County. Information on these and any additional reports may be obtained from El Paso County, City of Colorado Springs, Regional Floodplain Administrator or the Colorado Water Conservation Board.

5.5 Flood Flow Routing/Models

Several flood routing procedures are available in published texts. The basic data needed for the routing computations are the inflow hydrograph, the physical dimensions of a storage unit (i.e., reach of channel, reservoir, detention pond, etc.) and an outflow hydrograph. Flood flow routing in channels or through reservoirs/detention ponds include storage routing techniques.

The derivation of the routing equation for streamflow begins with the conservation of mass assumption which states that the difference between the average inflow and average outflow during some time period, t, is equal to the change in storage during that time period. This can be written in equation form as:

$$I - O = \triangle S/t \qquad (5\text{-}10)$$

where:

I = inflow per unit time;
O = outflow per unit time;
$\triangle S$ = change in storage volume of water; and
t = change in time period.

If inflow during the selected time period is greater than outflow, then S is positive and there will be storage in the drainage system. If inflow is less than outflow during the period, the S is negative and there will be loss of storage in the system.

When flow travels through a reach of a channel, the hydrograph is modified due to travel time and channel storage. In the absence of inflows from tributaries, overland flow, ground water flow or rainfall, the peak of a hydrograph would be somewhat reduced as it traveled downstream due to channel storage. More commonly, however, the effect of local inflows from the above sources exceed the effect of channel storage so that runoff peaks are increased in the downstream direction.

Flow routing procedures range in complexity from simple storage routing procedures to relatively complex procedures based on simultaneous solutions to the hydrodynamic equations dealing with the conservation of momentum and mass. Storage routing is sometimes known as hydrologic routing since the basic equation is the continuity equation. Several methods of hydrologic routing include basic storage routing, Muskingum routing, convex routing, and kinematic wave routing. Hydraulic flow routing, based on the momentum and continuity equations, is usually accomplished by a numerical solution of the governing equations or by the method of characteristics. This type of routing better models the dynamics of unsteady flow conditions. Both hydrologic and hydraulic routing procedures can be used for channel, reservoir, detention pond or basin routing.

Tributary inflows, overland flow and ground water contributions to flow can be added to the inflow or outflow of a given channel reach as appropriate. Normally, channel routing requires that the channel be divided into several reaches. The outflow from one reach becomes the inflow to the next reach downstream. The channel should be divided into reaches having relatively uniform hydraulic properties. The routing interval t should be around 1/5 to 1/3 of the time to peak of the hydrograph being routed. The reach length should not exceed the product of the average flow velocity and the routing interval. Thus, if the routing interval is 15 minutes and the average flow velocity is 8 feet per second (fps), the reach length should be less than

$$L = 8 \frac{ft}{sec} \times 15 \text{ min} \times 60 \frac{sec}{min} = 7200 \text{ ft.}$$

Generally for small channels, the nonuniformity of the channel may necessitate shorter reach lengths. The average flow velocity may be computed at bankfull discharge or at a flow equal to 75% of the hydrograph peak flow or at some other appropriate flow rate.

For routing flows through reservoirs or detention ponds, the data required include an inflow hydrograph, the physical dimensions of the storage facility, the maximum outflow release rate, and the hydraulic characteristics of the outlet works, principal spillway and/or emergency spillway. After the inflow hydrograph, stage area-storage capacity relationships, and stage-outflow relationship have been determined, they are combined in a storage routing analysis. The results of a routing analysis include the ordinates of the outflow hydrograph, the depths of storage, and the volumes of storage at each point in time during a given flood duration.

A design example using another procedure for evaluating storage characteristics relative to detention pond sizing, the Modified Rational Method Analysis, is presented in Chapter 11.

SECTION III

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CHAPTER 6 DESIGN CRITERIA

6.1 General Statement

The design criteria stated herein sets the minimum standards to be met by the engineer. Any exceptions to the design criteria must gain approval from the City/County before being submitted for use. The design criteria is organized in such a manner as to follow the supporting Chapters 7-12. The supporting chapters provide design methods and examples which utilize the design criteria.

6.2 Street Drainage Design Criteria

Design criteria for the collection and transport of runoff on public streets is based on a reasonable frequency of traffic and pedestrian interference. For example, flow on a local street is allowed to spread to the top of curb during a 10-year frequency storm. During the 10-year period, many storms will occur which will produce less runoff and will not inundate the entire street. This is consistent with the policy that states that the number one priority for use of streets is traffic.

Planning and design for storm runoff must be considered from the viewpoint of both the regularly expected storm occurrence, that is, the initial storm, and the major storm occurrence. The objectives of the major storm runoff planning and design is to eliminate inconvenience, frequently recurring minor damage, and high street maintenance costs.

6.2.1 Street Capacity for Initial Storms

Determination of street capacity for the initial storm shall be based upon pavement encroachment. The pavement encroachment for the initial storm shall be limited as set forth in Table 6-1.

In all cases the flow encroachment shall not extend past the street right-of-way.

The storm drain system should begin at or before the point where the maximum encroachment is reached.

6.2.2 Street Capacity for Major Storm

Determination of the allowable capacity for major storm shall be based upon allowable depth and inundated area. The allowable depth and inundated area for the major storm shall be limited as set forth in Table 6-1.

6.2.3 Calculating Capacity

The street capacity shall be calculated using Manning's Formula with an "n" value applicable to the actual boundary conditions encountered, when the allowable depth and inundated area is determined from Table 6-1. Also see Section 7.3

TABLE 6-1

Allowable Use of Roads and Streets

Street Classification	Use of Streets for Initial and Major Storms: Initial Storm	Use of Streets for Initial and Major Storms: Major Storm	Cross Flow In Streets For Initial and Major Storms: Initial Storm	Cross Flow In Streets For Initial and Major Storms: Major Storm
Hillside Residential (Less Than 32' F/C to F/C)	No curb overtopping, maximum street flow = 25 cfs, whichever is most limiting.	Same as Type A (Local/ Residential) below.	Same as Type A (Local/ Residential) below.	Same as Type A (Local/ Residential) below.
Type A (Local/Residential)	No curb overtopping, flow may spread to crown of street or top of curb, whichever is the most limiting.	Residential dwellings, public, commercial and industrial buildings shall not be inundated at the ground line. The depth of water at the gutter flow line shall not exceed 12 inches.	6 inches of depth in cross pan or gutter flow line	12 inches of depth at gutter flow line
Type A (Local with Roadside Ditch)	Flow must not encroach upon street shoulder area.	Residential dwellings, public, commercial and industrial buildings shall not be inundated at the ground line. The depth of flow shall not exceed 6 inches at the shoulder.	Requires culvert. Flow shall not encroach upon street shoulder.	Requires culvert, depth of flow shall not exceed 6 inches at the street shoulder.
Type B (Collector or Minor Arterial)	No curb overtopping. Flow spread must be limited to a maximum 20 foot spread from each curb face.	Same as Type A (Local/ Residential) above.	Where cross pans are allowed, depth of flow shall not exceed 6 inches at flow line	12 inches of depth at gutter flow line
Type B (Collector or Minor Arterial with Roadside Ditch)	Flow must not encroach upon street shoulder area.	Same as Type A (Local with Roadside Ditch) above.	Requires culvert. Flow shall not encroach upon street shoulder.	Requires culvert. Depth of flow shall not exceed 6 inches at the street shoulder.
Type C (Arterial)	No curb overtopping. Flow may encroach only onto one outside lane in each direction and must leave at least one 12 foot lane free of water in each direction.	Residential dwellings, public, commercial and industrial buildings shall not be inundated at the ground line. The depth of water shall not exceed 8 inches at the gutter flow line with no curb overtopping.	No allowable cross flow	No allowable cross flow
Type C (Arterial with Roadside Ditch)	Flow must not encroach upon street shoulder.	Residential dwellings, public, commercial and industrial buildings shall not be inundated at the ground line. Depth of flow shall not encroach upon street shoulder.	Requires culvert. Flow shall not encroach upon street shoulder.	Requires Culvert. Depth of flow shall not encroach upon street shoulder
Type D (Highway/Freeway)	No encroachment is allowed on any traffic lanes.	No encroachment on any traffic lanes.	No allowable cross flow	No allowable cross flow

6.2.4 Ponding

The term ponding shall refer to areas where runoff is restricted to the street surface by sump inlets, street intersections, low points, intersections with drainage channels, or other features. Ponding shall be considered gutter or ditch flow for purposes of meeting encroachment requirements for initial and major storms.

6.2.5 Cross-Street Flow

Cross-street flow is classified into two general categories. The first type is runoff which has been flowing down the street in a gutter and when the gutter capacity is exceeded, the flow crosses the street to the opposite gutter. The second type is flow from some external source, such a drainageway which is intersecting the street, which will flow across the crown of a street when the conduit capacity beneath the street is exceeded.

Cross street flow commonly occurs in urbanized areas under any one or combination of three conditions: 1) when gutter runoff is conveyed across the street crown to the opposite gutter; 2) when cross pans are used; and 3) when the flow in a drainageway exceeds the capacity of a road culvert and subsequently overtops the crown of the street. Cross-street flow depth shall be limited as set forth in Table 6-1.

6.2.6 Gutter Capacity

Pavement Encroachment

Limitations at intersections for pavement encroachment shall be as noted in Table 6-1, for allowable Initial/Major Storm Runoff Encroachment. Flows approaching intersections may have to be reduced ahead of the intersection by means of additional inlets in order to maintain the allowable encroachment.

Capacity

The capacity of each gutter approaching an intersection shall be calculated based upon the most critical cross section.

a. Continuous Grade Across Intersection - When the gutter flow will be continued across an intersection, the slope used for calculating capacity shall be that of the gutter flow line crossing the street.

b. Flow Direction Change at Intersection - When the gutter flow must undergo a direction change at the intersection greater than 45 degrees, the slope used for calculating capacity shall be the effective gutter slope, defined as the average of the gutter slopes at 0 feet, 25 feet and 50 feet from the geometric point of intersection of the two gutters.

c. Flow Interception by Inlet - When gutter flow will be intercepted by an inlet on continuous grade at the intersection, the effective gutter slope shall be utilized for calculations. Under this condition, the points for averaging shall be 0 feet, 25 feet and 50 feet upstream from the inlet.

6.2.7 INLETS

Where gutter or street capacity as shown on Table 6-1 is exceeded, inlets shall be installed to effectively intercept runoff. Inlet type, sizing and location shall be as defined in Chapter 7.

A standard in curb opening inlet type is the Colorado Department of Highways M-604-12 Type R inlet for 6" or 8" curb heights. Other types shall be as allowed by the standard construction specification or as otherwise approved by the City/County.

6.3 Storm Drain Design Criteria

6.3.1 Design Storm Runoff Frequency Criteria

The City/County design storm runoff frequency criteria relates to the function of operational and economic criteria with a special emphasis on public safety. As discussed in other sections of this Drainage Criteria Manual, some types of facilities or developments such as parks do not require high levels of protection and periodic flooding is acceptable subject to approval by City/County. However, for all facilities, the designer must consider the impact of a 10-year initial and a 100-year major storm runoff and provide for its passage with no loss of life or major property damage.

6.3.2 Pipe Sizes and Material Types

Pipes which are to become an integral part of the public storm drain system shall have a minimum diameter of 15 inches. If alternate shapes are required for utility clearance or special conditions, a designer/developer must contact the City/County for approval.

The selection of pipe material type and installation conditions normally determines the design life of the pipe. Pipe design life shall be a minimum of 50 years as certified by the design engineer on the construction plans. All requirements for which the design life is based must be met by the design and installation. For example, bedding requirements, scour, corrosion, soil and water chemistry are critical to meeting the pipe design life.

In selecting a roughness coefficient, consideration shall be given to the average conditions during the useful life of the structure. An increased "n" value shall be used primarily in analyzing old conduits where alignment is poor and joints have

become rough. If, for example, concrete pipe is being designed at a location and there is reason to believe that the roughness would increase through erosion or corrosion of the interior surface, slight displacement of joints, or entrance of foreign materials, a roughness coefficient should be selected which, in the judgment of the designer, will represent the average condition. Any selection of "n" values below the minimum or above the maximum, either for monolithic concrete structures, concrete pipe, or corrugated metal pipe, shall have the approval of the City/County.

The coefficients of roughness listed in Appendix A, Figure 6-C are for use in the nomographs contained herein, or for direct solution of Manning's Equation.

6.3.3 Pipe Grades and Flow Velocities

Storm Drains should operate with velocities of flow sufficient to prevent excessive deposition of solid material; otherwise, objectionable clogging may result. The controlling velocity occurs near the bottom of the conduit and is considerably less than the mean flow velocity in the pipe. Storm drains shall be designed to have a minimum mean velocity of 2.5 fps. Table 6-2 indicates the grades for both concrete pipe (n = 0.013) and for corrugated metal pipe (n = 0.024) to produce a velocity of 2.5 fps, which is considered to be the lower limit of scouring velocity. Any variance must be approved by the City/County or other appropriate agency. Outlets on pipes of minimum grade should be designed to avoid sedimentation at the outfall.

Maximum flow velocities in conduits are important mainly because of the possibilities of excessive abrasion on the storm drain inverts. For storm drain velocities in excess of 18 ft/sec the engineer must demonstrate that the higher velocities will be acceptable by the pipe manufacturers to meet an equivalent design life of 50 years.

6.3.4 Manhole Locations

Manholes shall be located at intervals as indicated in Table 6-3. Manholes shall be located as required for conduit junctions, changes in grade, changes in alignment, and ends of curved sections.

TABLE 6-2

MANHOLE SPACING

Pipe Size	Maximum Permissible Spacing
15"-36"	500 Feet
42"-60"	600 Feet
Over 66"	750 Feet

TABLE 6-3

MINIMUM SLOPE REQUIRED FOR SCOURING VELOCITY*

Pipe Size (inches)	Concrete Pipe Slope (ft/ft)	Corrugated Metal Pipe Slope (ft/ft)
15	0.0021	0.0074
18	0.0018	0.0060
21	0.0015	0.0049
24	0.0013	0.0041
27	0.0011	0.0035
30	0.0009	0.0031
36	0.0007	0.0024
42	0.0006	0.0020
48	0.0005	0.0016
54	0.0004	0.0014
60	0.0004	0.0012
66	0.0004	0.0011
72	0.0003	0.0010
78	0.0003	0.0009
84	0.0003	0.0008
96	0.0002	0.0007

* These slopes are based on pipe flowing at 0.8 depth

This table indicates grades for concrete pipe (n=0.013) and for corrugated metal pipe (n=0.024) to produce a velocity of 2.5 fps.

For determining hydraulic conditions in partially filled pipes, see Table XVI contained in the publication "Design and Construction of Sanitary and Storm Sewers", 1969 by the ASCE and WPCF.

6.3.5 Pipe Connections

The use of prefabricated wye and tee connections shall be restricted for pipes less than 48" in diameter. Larger connections must be made by field connections unless otherwise approved by the City/County.

Properly installed tongue and groove concrete pipe should have an interior joint gap of 3/8" for pipe less than 36" diameter installed in a straight run. Under maximum deflected conditions the exterior joint gap shall be no greater than 1". For pipes greater than 36" diameter, the general rule is that the joint gap should not exceed 1/3 the joint depth.

6.3.6 Alignment

In general, storm drain alignment between manholes shall be straight. Long radius curves may be allowed to conform to street alignment. Short radius curves may be used in place of elbows on larger pipes greater than 36" in order to reduce head losses. Curves may be produced by angling the joints or by fabricating beveled ends. Angled joints shall be kept at a minimum to maintain a tight joint. Curvature shall be limited to those specified by pipe manufacturer recommendations. Mitered bends shall be limited to 45° maximum.

6.3.7 Utilities

In the design of a storm drainage system, the engineer is frequently confronted with the problem of grade conflict between the proposed storm drain and existing utilities such as water, gas, telephone, electrical and sanitary sewer lines. When conflicts arise between a proposed drainage system and a utility system, the owner of the utility system shall be contacted, and an appropriate design shall be prepared and approved by City/County.

6.4 Culvert and Bridge Flow Design Criteria

6.4.1 Design Frequency Overtopping Criteria

Culverts shall be designed to pass the 10-year runoff and 100-year flood in accordance with the following Table. When flows are allowed to overtop certain roadway classifications, adequate embankment projection must be employed to prevent the roadway from eroding and potential failure.

TABLE 6-4

ALLOWABLE CULVERT OVERTOPPINGS*

Drainage Classification	Initial Drainage System Maximum Depth	Major Drainage System Maximum Depth
Types A & B (Local/Residential and Collector)	Headwater Depth Ratio is Less Than Pipe Height	12 inches of depth at the gutter flow line
Types A & B (Local/Residential and collector with	Headwater Depth Ratio is Less Than Pipe Height	6 inches of depth at the street
Types C & D (Arterial, Highway or Freeway with or without Roadside Ditch)**	Headwater Depth Ratio is Less Than Pipe Height	No shoulder encroachment

* A culvert is defined as any buried structure with both a clear opening less than 200 square feet, and span less than 20 feet and capacity less than 1500 cfs.

**On state highways, CDOH criteria shall be utilized.

TABLE 6-5

ALLOWABLE CULVERT HEADWATER DEPTHS FOR DESIGN FLOWS

Clear Opening (Ft^2)	Hw/D
200 or Greater	See allowable bridge clearance
200 to 50	$\frac{500 - Area}{300}$
50 or Less	Greater than 1.5 If approved by City/County.

The Engineer shall consider various factors in determining permissible headwater depths such as backwater effects, possible flooding, embankment erosion, overtopping and public safety.

6.4.2 Allowable Clearance for Bridges and Other Major Drainageway Crossings

All structures classified as bridge shall not be overtopped. For clear span bridges, the minimum clearance between the bridge low chord and the water surface profile shall be a minimum of 2 feet for the 100-year design flow. For box culverts classified as bridges or culverts at major drainageways (100-year flows greater than 1500 cfs) adequate freeboard shall be provided for the passage of debris and should be no less than 2 feet.

At the option of the City/County the designer may design culverts as storm drains with a 10-year design frequency. Under this option the proposed culvert shall be placed on line and grade to permit connection to the future storm drain. The major storm impacts shall be investigated for both the culvert configuration and the proposed storm sewer configuration. The major storm analysis of the storm drain shall include both upstream and downstream reaches sufficient to show that provisions for the major storm can be made when the storm drain is constructed.

6.4.3 Culvert Discharge Velocities

The velocity of discharge from culverts should be limited to those values presented in Table 10-3 and 10-4. Consideration must be given to the effect of high velocities, eddies or other turbulence on the natural channel, downstream properties and roadway embankments, as necessary.

Discharge velocities which exceed this criteria must have special energy dissipator structures at their outlets as shown in Chapter 10.

It is recommended that a minimum velocity of 2.5 feet per second at the design flow be maintained in all drainage structures to prevent siltation. Where doubt exists concerning silt or scour, protection commensurate with the value of the structure and surrounding property shall be installed to ensure that damage to or failure of the structure will not occur.

6.5 Open Channel Design Criteria

6.5.1 Channel Hydraulics

Manning's equation shall be used to approximate flow relationships in open channels.

Mannings Equation $$Q = \frac{1.486}{n} R^{2/3} S^{1/2} A$$

Q = Flow (cfs)
R = Hydraulic Radius
S = Slope (ft/ft)
A = Area (ft^2)
n = Roughness Coefficient (See Table 10-1 and 10-2)

6.5.2 Channel Velocity

Concrete, riprap, or soil cement linings as approved by the City/County shall be used where channel bottom velocities exceed 6.0 ft/sec. Grass lined channels shall not be used where velocity exceeds permissible velocities in Table 10-4 or the Froude number is greater than 0.9 for the 100-year storm.

6.5.3 Channel Cross Sections

Criteria pertaining to channel cross sections is as follows:

Side Slopes

Maximum side slopes permissible for grass lined channels is 4H:1V. Trapezoidal channels lined with concrete grouted riprap, or soil cement shall have maximum side slopes of 2H:1V. Existing channels with ROW restrictions and channels flowing under bridge abutments may have steeper side slopes as approved by the City/County. Loose riprap lined channels shall have a maximum side slope of 2.5H:1V.

Depth

Channel depth shall not exceed 5.0′ for the 100-year storm when the 100-year flow is 1500 cfs or less.

Bottom Width

Minimum bottom width shall be as necessary to accommodate construction and maintenance equipment, but shall be not less than 8 feet for channels conveying more than 400 cfs.

Low Flow Channel

Low flow channels shall generally be designed for the 10-year event. A minimum conveyance capacity of down to 10% of the 100-year storm event may be allowed if overbank conditions and scour velocities permit or only as otherwise approved by City/County Engineer.

Freeboard

The following formula shall be used to determine freeboard:

$$H = 1.0 + .025\ V\ D^{0.33}$$

where V = average channel velocity in fps and,

D = depth of flow in feet

Additional freeboard is required on the outside bank of curved channels for superelevation as determined in Chapter 10, Section 10.5.6.

Maintenance and Access Roads

All open channels shall have a minimum 15 foot wide maintenance road parallel and adjacent to the channel. 15 foot wide access roads shall also be provided to major drainage way structures to provide public maintenance as determined by the City/County. Adequate access easements should be provided.

6.5.4 Energy Dissipators

Where velocity and energy levels are excessive (See Table 10-1 thru 10-3) one or more of the following methods of dissipating energy will be required.

a) Impact Type Stilling Basin

b) Contra Costa Type

c) Plunge Pool Type (Riprap Basin)

For additional detail relative to design and construction of energy dissipators, See Chapter 10, Section 10.8.

6.5.5 Drop Structures

Depending on height of fall and materials used in construction, drop structures are generally categorized as follows:

Fall	Type of Structure
0-4 feet	Vertical and Sloping Drop
0-15 feet	Sloping Drop, Rectangular Inclined or Pipe Drop
> 15 feet	Baffled Chute

Riprap, grouted riprap, concrete, and other materials may be utilized in these applications as approved by City/County.

Drops should be designed to allow 200 feet between consecutive structures where possible.

6.5.6 Transitions

Means of transitioning in and out of in-channel structures is important to the safety of the structure with respect to scour damage. Appropriate headwall/wingwall configurations should be determined based on type of structure, type of channel lining, hydraulic profile, energy gradient, and estimated scour depths (see Section 10).

6.5.7 Riprap

The use of riprap shall be limited to erosion protection at structures, and channel areas where grass lining is not possible due to right-of-way constraints or excessive velocities. Riprap gradation shall be determined by the Urban Drainage and Flood Control District procedure for Froude numbers less than 0.8, and by the Office of Surface Mining method for steep slopes for Froude numbers greater than 0.8 (see Section 10, Reference 5).

Riprap protection for flood control channels and appurtenant structures should be designed so that storm damage requiring substantial reconstruction will not result from any flood that could reasonably be expected to occur during the service life (minimum 50 years) of the channel or structure. Nominal periodic maintenance as determined by the City/County is excepted.

6.6 Detention Storage Criteria

The structural classification of State of Colorado jurisdictional dams are identified below (refer to Colorado Office of the State Engineer "Design Review Manual" for dams and dam safety, effective September 30, 1988.

Where a reseasonable determination of the category or classification of the dam structure cannot be made by the design engineer, a final determination shall be obtained from the state engineer's office in writing:

A **"Jurisdictional Dam"** is a dam which impounds water above the elevation of the natural surface of the ground creating a reservoir with a capacity of more than 100 acre-feet, or creates a reservoir with a surface area in excess of 20 acres at the high-water line, or exceeds 10 feet in height measured vertically form the elevation of the lowest point of the natural surface of the ground where that point occurs along the longitudinal centerline of the dam up to the flowline crest of the emergency spillway of the dam. For reservoirs created by excavation, the vertical height shall be measured from the invert of the outlet. The State Engineer shall have final authority over determination of the vertical height.

A **"Non-jurisdiction Dam"** is less than the size and capacity of a jurisdictional dam.

A **"Flood Control Dam"** is a special purpose dam which is normally dry and has an ungated outlet structure which will drain the water impounded during the flood. The jurisdictional size and classification of the dam are determined assuming the reservoir is full to the emergency spillway.

A **"Diversion Dam"** is a dam constructed for the purpose of diverting water from the natural stream bed into a canal, tunnel, ditch, or pipeline, and which is not designed or operated for the purpose of impounding significant amounts of water. A dam that both impounds water and diverts water into a canal, tunnel, ditch, or pipeline shall be considered a dam and is subject to these rules.

6.6.1 Hazard Classification (Per Rule 4)

"Classification of a Dam" is the placement of a dam into a category based upon an evaluation of the consequences of the failure of the dam absent flooding conditions, assuming the reservoir is at the high-water line. No loss of life nor significant damage is expected to occur if the increased depth of flow velocity and the depth of flow at a critical area is less than seven (7) in the incremental zone.

A **"Class I"** dam is a dam for which loss of human life is expected in the event of failure of the dam.

A **"Class II"** is a dam for which significant damage is expected to occur, but no loss of human life is expected in the event of failure of the dam. Significant damage is defined as damage to structures where people generally live, work, or recreate, or public or private facilities exclusive of unpaved roads and picnic areas. Damage means rendering the structures uninhabitable or inoperable.

A **"Class III"** dam is a dam for which loss of human life is not expected, and damage to structures and public facilities as defined for a "Class II" dam is not expected in the event of failure of the dam.

A **"Class IV"** dam is a dam for which no loss of human life is expected, and which damage will occur only to the dam owner's property in the event of failure of the dam.

6.6.2 Size Classification (Per Rule 4)

A **"Minor Dam"** does not exceed 20 feet in vertical height and 100 acre feet in capacity.

A **"Small Dam"** is greater than 20 feet in vertical height but equal to or less than both 40 feet and 1000 acre-feet in capacity, or is greater than 100 acre feet but equal to or less than both 1,000 acre-feet in capacity and 40 feet in vertical height.

A **"Intermediate Dam"** is greater than 40 feet in vertical height but equal to or less than both 100 feet and 50,000 acre-feet in capacity, or is greater than 1,000 acre-feet in capacity but equal to or less than both 50,000 acre-feet in capacity and 100 feet in vertical height.

A **"Large Dam"** is greater than 100 feet in vertical height, or greater than 50,000 acre-feet in capacity.

6.6.3 Spillway Design Requirements

The inflow design flood (IDF) requirements for determining the spillway capacity are summarized in the following table. These requirements may be waived for good cause shown. Spillways designed in accordance with these rules will not be required to be enlarged due to subsequent revisions to the probable maximum precipitation estimates, unless, in the opinion of the State Engineer, there is a substantial threat to public safety.

Dam Class	I	II	III	IV
Dam Size				
Large	PMP*	.75 PMP	100 YR	50 YR
Intermediate	PMP	.50 PMP	100 YR	50 YR
Small	PMP	.50 PMP	100 YR	25 YR
Minor	.50 PMP	100 YR	50 YR	25 YR

*PMP = Probable Maximum Precipitation

6.6.4 Non-Jurisdictional Dams

For non-jurisdictional dams, i.e., those that do not or would not fall under State of Colorado purview, the designer must evaluate the various factors as identified in the engineer check list, presented in Chapter 11 for the established dam classification.

The minimum inflow design flood required for the principal spillway of a non-jurisdictional dam is the 10-year event, whereas the design for the combined principal and emergency spillway system shall be to safely pass the 100-year inflow design flood.

For design purposes, the principal spillway storm shall be routed through the detention pond with a maximum release rate which does not exceed the downstream peak discharge requirements (See Chapter 11). Flood flow releases shall be assumed to start at the elevation of the lowest ungated outlet. The structure shall be sized to handle the principal spillway storm without any emergency spillway flow.

The emergency spillway storm must also be routed through the storage pond area with a starting water surface elevation equal to the elevation of the lowest ungated outlet with clogged principal spillway conditions, unless otherwise approved by City/ County. Site conditions will dictate whether a riser pipe or excavated spillway is selected to handle the emergency spillway storm. The emergency spillway shall be designed to handle the emergency spillway storm without causing over topping and failure of the storage structure.

Procedures for design storm development shall be in accordance with those outlined in Section II of this manual.

APPENDIX 6A
STORM DRAINS AND CULVERTS
DESIGN CRITERIA AND REQUIREMENTS FOR DESIGN

633 Design Criteria for Pipe

This Criteria is to be used in conjunction with Section 630 of the City of Colorado Springs Standard Specifications requirements and Section 700 of the Colorado Department of Highways Standard Specification for Road and Bridge Construction for the selection of pipe material for installation conditions encountered in the field.

Pipe materials acceptable for installation as storm drains are:

Reinforced Concrete Pipe (RCP)
Reinforced Concrete Box (RCB)

Corrugated Steel Pipe - Galvanized (CSP)
Aluminized Corrugated Steel Pipe - Type 2 (ACSP)

Ribbed Polyvinyl Chloride Pipe (RPVC)
Smooth Wall Polyvinyl Chloride (SPVC)
Profile Wall Polyethylene Pipe (PWPE)
Corrugated Polyethylene Pipe (CPE)

633.1 Service Life

The minimum specified service life shall be assured by the design, materials and installation of storm drains and culverts constructed as public facilities in rights-of-way, easements and within roadways with the following classifications:

A. Minimum 100 year service life for freeways, expressways and major arterials however, unless otherwise allowed by the City/County.

B. Minimum 50 year service life for minor arterials, collectors, industrial, frontage roads, residential, alleys and all other roadways not noted in "A" above and in areas outside of roadways unless otherwise required by the City/County.

Thermoplastic pipe for storm drain and culvert application shall be limited to installations for 50 year service life.

The Design Engineer shall substantiate the intended service life with appropriate engineering, field and test data as may be required by the City/County.

The limit of service life is defined as the point where the pipe or culvert fails structurally, wears or corrodes to the point of perforation or leakage, or becomes misaligned to the point where it does not function hydraulically as intended.

The pipe installation shall meet the minimum design requirements noted herein for abrasion, corrosion, chemical deterioration, and structural integrity.

633.2 Testing of Installation Site

The installation site may necessitate subsurface investigation by the project owner to determine the suitability of trench condition for the pipe and culvert. Where required by the project engineer, geotechnical data, test holes and soil samples along the pipeline alignment shall be provided by a geotechnical engineer in order to substantiate the soil characteristics, bedding requirements and special design requirements if rock, groundwater or other unsuitable soil conditions and soil types are encountered.

The testing of the native soil types is considered essential to proper design, installation and long term performance in the use of flexible pipe materials.

Where required, additional soil and ground water tests shall be submitted to the City/County substantiating the selection of pipe materials. At a minimum, tests shall include but not be limited to:

A. For concrete pipe, test data shall be provided for sulfates and chlorides.

B. For steel pipe, test data shall be provided for the pH (ion concentration) and electrical resistivity.

633.3 Sulfates and Chlorides

For precast concrete pipe installations the following limits should be observed in the selection of cement where pH (ion concentration) is between 5 and 9:

Cement	Percent Water-Soluble Sulfate (As SO_4) In Soil Samples	Parts per Million Sulfate (As SO_4) In Water Samples
Type I	0 to 0.10	0 to 150
Type II	0.10 to 0.20	150 to 1,500
Type V (1)	0.20 to 2.00	1,500 to 10,000
Type V (2)	over 2.00	over 10,000

(1) Type V or approved Portland pozzolon cement.
(2) Type V plus approved Portland pozzolon cement.

Note: A modified Type II cement containing less than 5.0% Tricalcium Aluminate as required by ASTM C-150 for Type V cement may be used when certified by the pipe manufacturer.

Where chlorides exceed 1.00% for soils or 1,000 parts per million in water, additional protection for reinforcing steel should be provided to extend the service life. Consideration should be given to increased concrete cover, higher quantity concrete with low permeability, minimizing cracks and voids as much as possible or the application of a barrier type protective coating as approved by the City/County.

633.4 Field Resistivity Survey and Sampling for Corrosion Tests

For corrugated steel pipe installations, the soil and groundwater properties are to be substantiated by standardized measurements of pH and resistivity of the native soil and water to predict metal loss and service life. This determination shall also apply to select imported bedding material to be used for trench backfill.

The useful service life due to metal pipe corrosion shall be determined by the Chart; "Service Life Based on Corrosion, Galvanized-Corrugated Steel Pipe" (see Figure 6A) and the chart; "Service Life Based on Corrosion, Aluminum Coated (Type 2) - Corrugated Steel Pipe" (see Figure 6B).

The geotechnical engineer shall make sufficient resistivity determinations at various locations along the pipe trench and within the pipe zone to adequately represent the entire reach. If the resistivity is reasonably uniform, a minimum of three soil samples from different locations will be required. If various locations show resistivity that differs significantly from the average for the area being surveyed, additional soil samples shall be taken, particularly in those areas with resistivity below the average.

Field resistivity tests may be performed by use of a portable earth resistivity meter for indication of the soluble salts in the soil or water and may be used as a guide for selecting samples that will be further tested in the laboratory.

The suggested field and laboratory methods for determining pH and resistivity is the California test method 643-B.

633.5 Chemical Resistance

In the selection of the pipe materials by the design engineer, consideration shall be given to any detrimental stormwater constituents which may effect the performance of the material and the design is to be modified as necessary.

Thermoplastic pipe shall not be used for installations where it may be subject to chemical attack by substances indicated as detrimental by the manufacturer's listing.

634.0 Requirements for Pipe Design

All appropriate data required for design, type of materials and construction or installation shall be noted on the construction plans and/or specifications. Any additional design requirements of the Drainage Criteria Manual shall also be noted.

634.1 Corrosion Design

For steel pipe, one of the factors for selection of wall thickness (gage) shall be based on the potential for corrosion. The pH and resistivity of the soil and groundwater shall be determined as noted in Section 633.4 and minimum wall thickness shall be selected from Figure 6A or Figure 6B and any other appropriate engineering data.

Where tests indicate that the pH is below 6 or above 8 with resistivity below 2,500 ohm-cm for galvanized steel or where the pH is below 6.1 or 8.5 with resistivity below 1,000 ohm-cm for aluminized steel (Type 2), steel pipe should not be considered for installation in native soils unless additional corrosion mitigating measures are employed as approved by the City/County.

634.2 Abrasion Design

It is to be assumed that a bed load will be present during at least a portion of the pipe service life. Design velocities established for the pipe flowing full or one-half full shall not exceed the following limits unless remedial measures for abrasion are applied to at least the invert of the pipe (at a minimum the lower 25% of round pipe and lower 40% of pipe-arch or elliptical pipe):

A. Concrete Pipe

Where velocities exceed 18 feet per second (FPS), additional protection shall be provided in the form of increased cement content (eg. 8 sack mix), increased cover over reinforcing steel (up to 1-1/2") or the use of harder aggregate.

B. Corrugated Steel Pipe

B.1 Galvanized - Corrugated Steel Pipe

Where design velocities are 6 (FPS) or less, no additional invert protection will be required. Where design velocities exceed 6 FPS, paved invert protection shall be provided.

B.2 Aluminum Coated (Type 2) - Corrugated Steel Pipe

Where design velocities are 10 FPS or less, no additional invert protection will be required. Where design velocities exceed 10 FPS, paved invert protection shall be provided.

Invert paving shall consist of asphalt paving (moderately abrasive bed load) or concrete paving (extremely abrasive bed load), shop or field applied. Anticipated or field observed bed load conditions shall be as determined by the design engineer.

Asphalt invert paving shall conform to AASHTO M-190, Type B with the additional provision that the paving shall have a minimum thickness of 1/4 inch above the crest of the corrugations. Asphalt paving shall not be installed within three (3) pipe diameters from the ends of culverts or other installation conditions which may cause exposure to sunlight. In these cases, concrete paving or an alternate pipe material shall be installed in the last section of pipe. Repairs to asphalt paving shall be in conformance with ASTM A-849.

For concrete invert paving the minimum thickness over the top of the corrugations (or top of reinforcement, whichever controls) at the invert shall be a minimum 1/48 of the pipe diameter (or equivalent pipe diameter) or a minimum of 3/4 inch thickness whichever controls. The concrete paving shall have a minimum strength of 4,000 psi and prepared with a minimum of Type IIa cement according to ASTM C-150 with chemical and abrasive resistant fine aggregate according to ASTM C-33. Where required, welded wire fabric reinforcement or deformed reinforcing bars shall be provided to maintain the integrity of the lining and shall be mechanically fastened to the pipe. The minimum area of steel for reinforcement shall not be less than 0.0018 of concrete cross sectional area. Welded wire fabric shall be galvanized and conform to AASHTO M-55. Steel reinforcement bars may be used with a design approved by the engineer.

Coating, paving or lining types other than asphalt or concrete will not be allowed unless otherwise approved by the City/County.

Where velocities exceed 18 feet per second, additional protection shall be provided as recommended by the manufacturer and approved by the City/County.

C. Thermoplastic Pipe

Where design velocities exceed 18 FPS and/or highly abrasive bed-load conditions, additional protection or wall thickness shall be provided in accordance with established engineering principles and manufacturing methods and shall be approved by City/County.

634.3 Structural Design

A. Precast concrete pipe and concrete box section installation shall be in conformance with the appropriate

specifications for crack limitations in the selection of pipe class, bedding and height of cover requirements as determined by the reference specifications in Section 635 and 637 of the City Standard Specifications or related County Specifications (CDOH STD. M-603-2).

Precast pipe class selection shall be based on the D-Load three-edge bearing strength required to produce a 0.01 inch crack. The appropriate load factor and factor of safety shall be considered with trench loadings and the bedding classes required. Normally, the factor of safety is 1.5 for nonreinforced pipe and 1.0 for reinforced pipe.

Wall thickness and reinforcement for cast in place box sections shall conform to the Colorado Department of Highways standard M-601-1 "Single Concrete Box Culvert", M-601-2 "Double Concrete Box Culvert" and M-601-3 "Triple Concrete Box Culvert", exceptions noted.

Where cast in place box sections do not conform with the above referenced standards, the design engineer shall submit structural calculations to the City Engineer for the appropriate live load and dead load design requirements. Live load shall be AASHTO HS 20-44 and dead load shall be an earth load of 84 lbs/cu. ft. with equivalent fluid pressure of 30 lbs/cu. ft.

B. For corrugated steel pipe and pipe arch, the corrugations and wall thickness (gage) shall meet with the minimum cover and height of cover requirements for H-20 highway liveloads conforming with Colorado Department of Highways Standards M-603-1 for "Metal Culvert Pipe", exceptions noted and M-510-1 for "Structural Plate Culvert Pipe", exceptions noted.

C. For thermoplastic pipe, the minimum and maximum height of cover shall conform to the manufacturer's allowable values based on the recommended bedding classifications and H-20 highway loadings or special loading conditions as appropriate. The appropriate design data shall be submitted to the City/County for approval.

Allowable pipe stiffness shall be determined by the parallel plate test according to ASTM D-2412 or equivalent standardized test methods acceptable to the City/County.

Design of the pipe installation shall conform to the minimum requirements of the "Interim Specifications - Bridges 1987 - 1988, AASHTO Section 18, Soil - Thermoplastic Pipe Interaction Systems".

634.4 Hydraulics

A. Pipe capacity and design shall be in accordance with the City/County Drainage Criteria Manual.

B. Acceptable Manning's Roughness Coefficient "n" for pipe materials are:

Reinforced Concrete Pipe (RCP)	0.013
Corrugated Steel Pipe-Galvanized (CSP)	see Figure C
Aluminized Corrugated Steel Pipe (ACSP)	see Figure C
Ribbed Polyvinyl Chloride (RPVC)	0.012
Smooth Polyvinyl Chloride (SPVC)	0.010
Profile Wall Polyethylene (PWPE)	0.012
Corrugated Polyethylene Pipe, Type S (CPE)	0.012

SERVICE LIFE BASED ON CORROSION
GALVANIZED - CORRUGATED STEEL PIPE

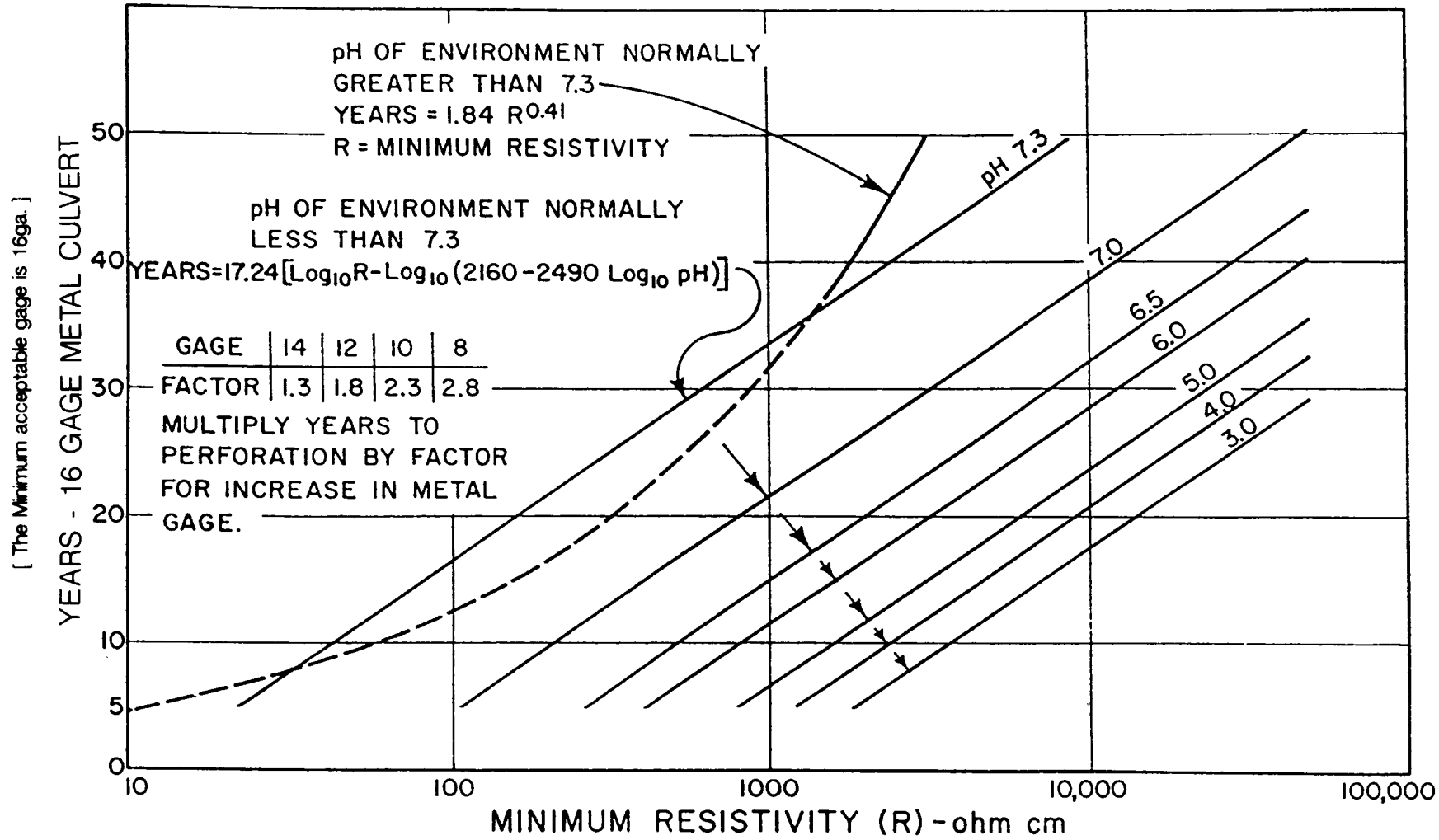


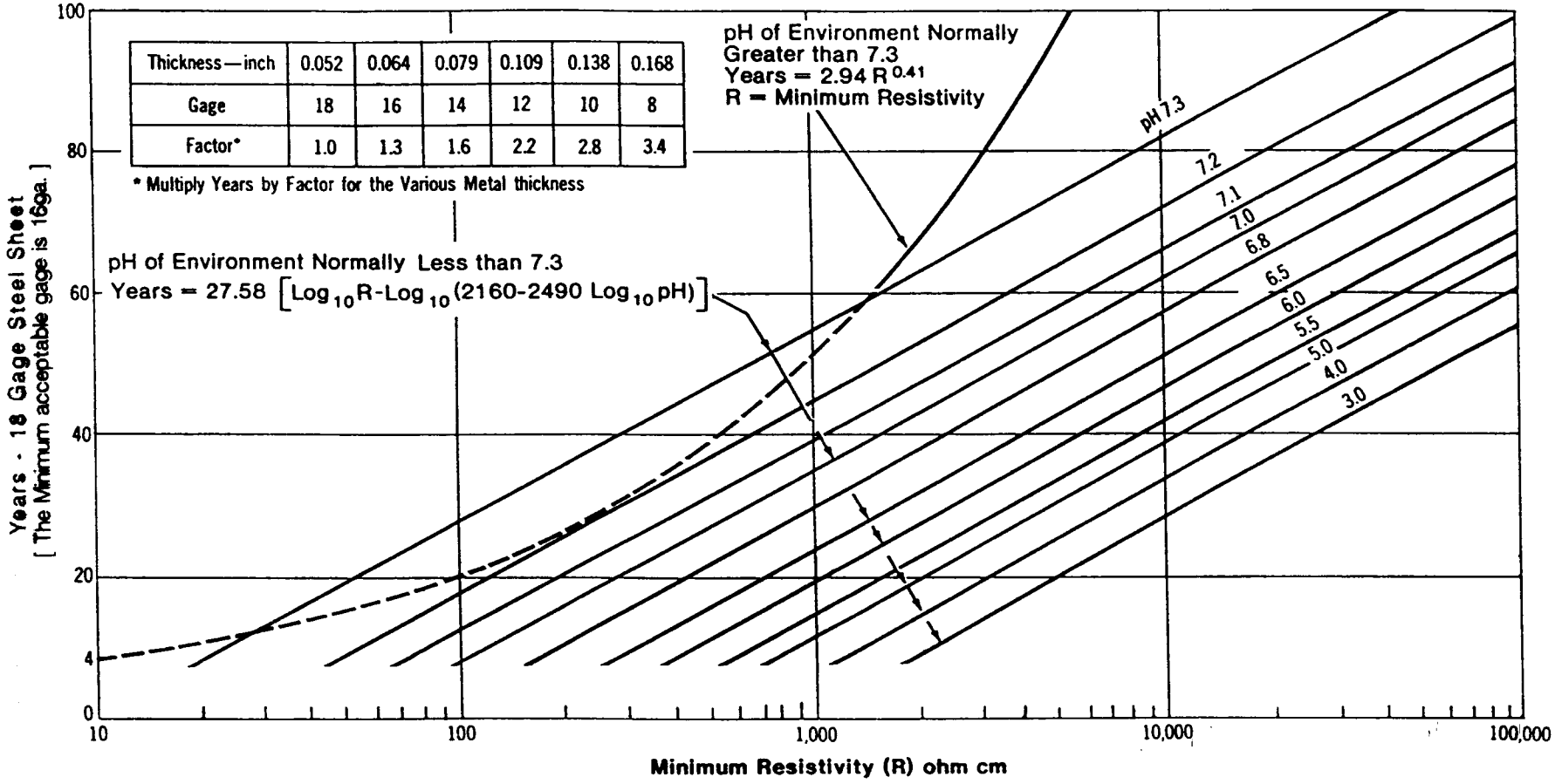
CHART FOR ESTIMATING METAL CULVERT SERVICE LIFE

Source: State of California Dept. of Public Works,
Division of Highways
Test Method No. Calif. 643-B

FIGURE 6A

CITY OF COLORADO SPRINGS · ENGINEERING DIV.

SERVICE LIFE BASED ON CORROSION
ALUMINUM COATED (TYPE 2) - CORRUGATED STEEL PIPE



Thickness—inch	0.052	0.064	0.079	0.109	0.138	0.168
Gage	18	16	14	12	10	8
Factor*	1.0	1.3	1.6	2.2	2.8	3.4

* Multiply Years by Factor for the Various Metal thickness

Source: American Iron and Steel Institute

FIGURE 6B

CITY OF COLORADO SPRINGS ENGINEERING DIV.

Values of Coefficient of Roughness (n) for Standard Corrugated Steel Pipe (Manning's Formula)*

Corrugations	Annular 2⅔ × ½ in.	Helical							
		1½ × ¼ in. (11, 12)		2⅔ × ½ in.					
	All Diameters	8 in.	10 in.	12 in.	18 in.	24 in.	36 in.	48 in.	60 in. and Larger
Unpaved	0.024	0.012	0.014	0.011	0.013	0.015	0.018	0.020	0.021
25% Paved	0.021					0.014	0.017	0.020	0.019
Fully Paved	0.012					0.012	0.012	0.012	0.012

	Annular 3 × 1 in.	Helical—3 × 1 in.					
		48 in.	54 in.	60 in.	66 in.	72 in.	78 in. and Larger
Unpaved	0.027	0.023	0.023	0.024	0.025	0.026	0.027
25% Paved	0.023	0.020	0.020	0.021	0.022	0.022	0.023
Fully Paved	0.012	0.012	0.012	0.012	0.012	0.012	0.012

	Annular 5 × 1 in.	Helical—5 × 1 in.			
		54 in.	60 in.	66 in.	72 in. and Larger
Unpaved	0.025	0.022	0.023	0.024	0.025
25% Paved	0.022	0.019	0.020	0.021	0.022
Fully Paved	0.012	0.012	0.012	0.012	0.012

*AISI

SOURCE:
HANDBOOK OF STEEL DRAINAGE & HIGHWAY PRODUCTS.
AMERICAN IRON AND STEEL INSTITUTE 1983.

FIGURE 6C

CITY OF COLORADO SPRINGS · ENGINEERING DIV.

SECTION III

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CHAPTER 7 STREET DRAINAGE AND STORM WATER INLETS

7.1 General Statement

Streets serve a drainage service even though their primary function is for the movement of traffic. However, good planning of streets can substantially help in reducing the size of, and sometimes eliminating the need for, a storm drain system in newly developing areas. Traffic and drainage uses are compatible only to a point, beyond which drainage must be secondary to traffic needs.

It is important that initial planning for drainage facilities should be completed prior to layout of the streets. This will prevent conflicts between conveyance of major storm flows and streets during the preliminary planning process.

The criteria for conveyance of major and minor runoff events in streets is outlined in Chapters 2 and 6. The following sections provide additional detail with regard to criteria and design methodology.

7.2 Effects of Storm Water Runoff on Street Capacity

Storm water runoff which influences the traffic capacity of a street can be classified as follows.

A. Sheet flow across the pavement when rain falls on the pavement and flows to its edge.

B. Runoff flowing adjacent to the curb.

C. Storm water ponded at low points.

D. Flow across the traffic lane from external sources, such as from a cross-street.

Each of these types of storm water runoff must be controlled within acceptable limits so that the street's main function as traffic carrier will not be unduly restricted.

The effect of each of the above categories of runoff on traffic movement are discussed in the following paragraphs.

7.2.1 Interference Due to Sheet Flow Across Pavement

Rainfall which falls upon the paved surface of a street will flow overland until it reaches a channel. Channels can be created either by curbs and gutters or by roadside ditches. The direction of flow on the street may be determined by the vector addition of the street grade and the crown slope, which is equivalent to drawing the perpendicular to a contour line on the

road as shown on Figure 7-1. The depth of gutter flow will be essentially zero at the crown of the street and will increase as it flows towards the channel. Traffic interference due to gutter flow is essentially of two types: Hydroplaning and splash.

Hydroplaning is the phenomenon of vehicle tires actually being supported by a film of water which acts as a lubricant between the pavement and the vehicle. It generally occurs at speeds estimated for arterial streets. Its effect can be minimized by achieving a relatively rough pavement or by reducing travel speed.

Traffic interference due to splash results from gutter flow of excessive depth caused by water traveling a great distance or at a very low velocity before reaching a gutter. Its effect can be minimized by increasing the street crown slope which will decrease both the time and distance required for water to reach the gutter. The crown slope, however, must be kept within acceptable limits to prevent side-slipping of traffic during frozen surface conditions and to allow the opening of doors when parked adjacent to curbs. An exceedingly wide pavement section contributing flow to one curb will also affect the depth of flow. This may be due to superelevation of a curve, off-setting of the street crown due to warping of curbs at intersections, or many traffic lanes between street crown and the gutter. Consideration should be given to all of these factors to maintain a depth of sheet flow within acceptable limits.

7.2.2 Interference Due to Gutter Flow

Water which enters a street will flow in the gutter of the street until it reaches an outlet, such as a storm drain inlet or a channel. Figure 7-1 illustrates the movement of gutter flow down a street when there is a storm drain system. As the flow is conveyed downhill and additional areas contribute to the runoff, the width of flow will increase and progressively encroach upon the traffic lane. If vehicles are parked adjacent to the curb, the top width of flow will have little influence on traffic until it exceeds the width of the vehicle by several feet. However, on streets where parking is not permitted, as with many arterial streets, whenever the flow width exceeds a few feet it will significantly affect traffic. Field observations indicate that vehicles will crowd adjacent lanes to avoid curb flow.

As the flow width increases, it becomes impossible for vehicles to operate without driving through water, and they again begin to use the inundated lane. At this point, the traffic velocities will be significantly reduced as the vehicles begin to drive through the deeper water. Splash from vehicles traveling in the inundated lane obscures the vision of drivers of vehicles moving at a higher rate of speed on the open lane.

Eventually, if width and depth of flow become great enough, the street will become ineffective as a traffic carrier. During these periods, it is imperative that emergency vehicles such as fire trucks, ambulances and police cars be able to traverse the street by moving along the crown of the roadway.

The street classification is also important when determining acceptable interference to traffic movement. A local street, and to a lesser extent a collector street, can safely function with greater traffic lane encroachment by flows than can an arterial.

7.2.3 Interference Due to Ponding

Storm runoff ponded on the street surface because of a change in grade or the crown slope of intersecting streets has a substantial effect on traffic. A major problem with ponding is that it may reach depths greater than the curb and remain on the street for long periods of time. Another problem is that ponding is localized in nature and vehicles may enter a pond moving at a high rate of speed.

The manner in which ponded water affects traffic is essentially the same as for curb flow; the width of spread onto the traffic lane is the critical parameter. Ponded water will often bring traffic on a street to a complete halt. In this case, incorrect design of only one facet of an entire street and storm drainage system will render the remainder of the street system useless during the runoff period.

7.2.4 Interference Due to Water Flowing Across Traffic Lane

Whenever storm runoff moves across a traffic lane, a serious impediment to traffic flow could occur. The cross flow may be caused by superelevation of a curve or a street intersection exceeding the capacity of the higher gutter sending flow from one side to the other side of the street. The problem associated with this type of flow is the same as for ponding in that it is localized in nature and vehicles may be traveling at high speed when they reach the location. If the velocity of vehicles is naturally slow and use is light, such as on local streets, cross-street flow does not cause sufficient interference to be objectionable.

The depth and velocity of cross-street flow should always be maintained within such limits that it will not have sufficient force to affect moving traffic. If a vehicle which is hydroplaning enters an area of cross street flow, even a minor force could be sufficient to move it laterally towards the gutter.

7.2.5 Effect on Pedestrians

In areas where pedestrians frequently use sidewalks, splash due to vehicles moving through water is a problem. It must also be kept in mind that under certain circumstances, pedestrians will be required to cross ponded water adjacent to curbs.

Since the majority of pedestrian traffic will cease during the actual rainstorm, less consideration needs be given to the problem while the rain is actually falling. Ponded water, however, remaining after the storm has passed, must be negotiated by pedestrians.

Interception capacity of grate, curb opening and combination inlets should be adjusted to account for accumulation of debris during a storm event. The adjustment factors (F) shown in Table 7-1 shall be applied to the resulting gross area requirement for the inlets.

7.3 Calculating Capacity

When the allowable encroachment has been determined, the gutter and street capacity shall be computed using the following modified Manning's formula.

Gutter cross sections usually resemble a triangular shape with the curb forming the vertical leg of the triangle. The gutter may have a straight cross slope, a cross slope composed of two straight lines (V-shape), composite cross slopes or parabolic sections for older pavements.

The modified Manning equation is utilized in triangular channels to better describe the hydraulic radius of a gutter section. The Manning equation is integrated across the section width. The equation in terms of cross slope and width of flow on the pavement is:

$$Q = \frac{0.56}{nS_x} S^{0.5} (TS_x)^{2.67}$$

where:
- Q = flow rate ft^3/sec
- T = width of flow (spread) ft
- S_x = cross slope, ft/ft
- S = longitudinal slope, ft/ft
- n = Manning's n coefficient

The resistance of the curb face is neglected in the equation since the resistance is negligible when the cross slope is 10 percent or less.

Figure 7-2 can be used for a direct solution of the above equation using Manning's n value of 0.016. For the discharge of a composite cross slope, use Figures 7-2 and 7-3.

Design Example
Gutter Carrying Capacity, Initial Storm

Given: 6" vertical curb
2'-wide by 2"-deep gutter
2% pavement crown slope
36' street width, curb to curb
Crown offset to 1/4 point for cross fall
Collector street
Street grade = 5%
Assume n = 0.016

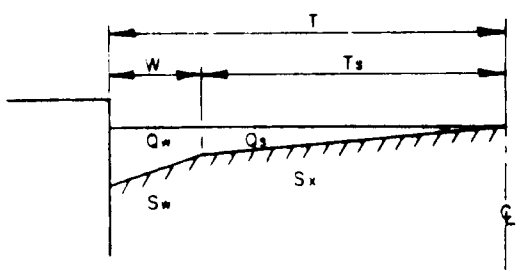
Find: Capacity, each gutter.
1. Determine allowable pavement encroachment.
From Table 6-1

2. Calculate capacity for each gutter.

$T = 9$ ft, $S_W = 0.08$

$W = 2$ ft, $S_X = 0.02$

$T_S = 7$ ft, $S = 0.05$



9/30/90

Left Side

Using Nomograph, Figure 7-2.

$T = T_s = 7$ ft.

Therefore $Q_s = 2.1$ cfs.

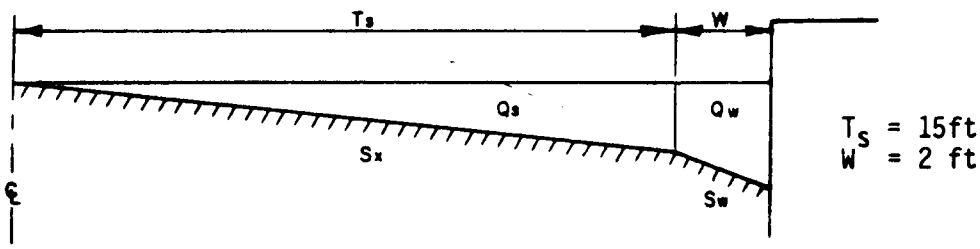
Using Figure 7-3

$\frac{W}{T} = 0.22$ $\qquad \frac{S_w}{S_x} = 4.0$

Therefore $E_o = 0.62$

Q_t total $= \frac{Q_s}{1-E_o} = 5.53$ cfs

$Q_w = Q_t - Q_s = 5.53 - 2.10 = 3.43$ cfs



Right Side

Using nomograph, Figure 7-2

$T = T_s = 15$ Ft.

Therefore $Q_s = 16$ cfs

Using Figure 7-3

$\frac{W}{T} = 0.12$ $\qquad \frac{S_w}{S_x} = 4.0$

Therefore $E_o = 0.37$

$Q_t = 25.4$ cfs

$Q_w = 9.4$ cfs

7.4 Storm Water Inlet Types and Use

The hydraulic capacity of a gutter inlet depends upon its geometry and the characteristics of the gutter flow. The inlet capacity governs both the rate of water removal from the gutter and the amount of water that can enter the storm drain system. Many costly storm drains flow at less than design capacity because the storm runoff cannot enter the drains efficiently. Inadequate inlet capacity or poor inlet location may cause flooding on the traveled way which creates a safety hazard or at times interrupts the normal flow traffic.

In addition to its hydraulic function, the inlet is frequently located in or near the path of vehicular traffic. Water-borne debris and trash may be deposited on the inlet causing complete or partial clogging. Often freedom from clogging and noninterference with traffic requires an inlet of a specific type rather than the most efficient inlet from a hydraulic point of view. For example a curb-opening inlet might be used where a grate inlet would be more efficient.

Gutter inlets can be divided into three major classes, each with many variations. These classes are (1) curb-opening inlets, (2) grated inlets, and (3) combination inlets (see Figure 7-4).

A brief description of the inlets types follow:

1. Curb-opening inlets. These inlets are standard for both the City and County and consist of a vertical opening in the curb through which the gutter flow passes.

2. Grate inlets. These inlets consist of an opening in the gutter covered by one or more grates. Grate inlets shall not be utilized for roadway applications except as approved by the City/County.

3. Combination inlets. These units consist of both a curb-opening and grate inlet acting as a unit and shall be utilized only with specific approval by the City/County.

The term inlet capacity is used herein to mean the <u>catch of the inlet</u> under a given set of conditions rather than the maximum water that can be intercepted by the inlet if the flow is increased without limit. The efficiency of an inlet is the flow intercepted by the inlet, Q_1, divided by the flow in the gutter, Q. The flow that bypasses the inlet, Q_c, is termed carry-over.

A major factor in the capacity of a curb-opening inlet is the depth of water in the gutter immediately adjacent to the opening. The capacity of an efficient grate inlet depends principally upon the quantity of water flowing in the section formed by projecting the grate width upstream. Thus, an increase in longitudinal

slope reduces inlet capacity (except for some grates or where deflectors are used) while an increase in transverse (cross) slope increases inlet capacity. Increase in length of a curb-opening inlet and increases in width of a grate opening increases the capacity of the inlet. For grate inlets, the efficiency of the grate opening is an important factor in inlet capacity.

For a curb-opening inlet, depressing the gutter increases the capacity of the inlet. The amount of the depression has more effect on the capacity than the arrangement of the depressed area with respect to the inlet. However, this may effect the flow of traffic and drivability on the street surface and is generally not permitted.

Most investigators have pointed out that the efficiency of an inlet is greatly increased by allowing a small percentage of the flow to bypass the inlet. For a given gutter discharge, the catch of flow for each additional increment of width (grate inlets) or length (curb-opening inlets) becomes rapidly less. Thus, the cost of catching the small amount of flow near the thin edge of the triangular flow channel approaches or exceeds the cost of catching the greater amount flowing nearer to the curb.

TABLE 7-1

INLET SIZING ADJUSTMENT FOR CLOGGING

	Adjustment Factor (F)
Curb Opening Inlet Length at Sump (Clogging)	1.25
Grate Inlet Area at Sump	2.0
Combination Inlets (Grate & Hood) at Sump	1.5
Grate Inlet Area for Continuous Grade	1.6

Example:

Calculated Grate Opening Inlet Size: $A = 10\ Ft^2$
Based on Effective Area

Required Grate Opening Area Size $= A \times F$
$= 10.0 \times 2.0$
$= 20.0\ Ft^2$

9/30/90

7.5 Grate Inlets

Grate inlets are best suited for parking lots, medians or areas isolated from roadways and where clogging with debris is not a problem. When the longitudinal grade exceeds about one percent, grates with rectangular longitudinal bars (parallel to flow) are much more efficient than grates with transverse (normal to flow) rectangular bars. As the grade increases, longitudinal bar grates become increasingly superior to transverse bar grates both in regard to interception of flow and decrease in tendency to catch debris. However, some cross-bar grates of special design with bars curved in the vertical plane have been found in manufacturers tests to intercept more of the flow than longitudinal bar grates. In general, an unclogged, efficient, grate inlet on a continuous grade will intercept all water flowing within a width equal to that of the grate.

For an efficient grate inlet, all rectangular bars should be parallel with the flow and the openings should equal at least 50 percent of the width of the grate. The clear length of the opening should be sufficient to allow the water to fall through the opening without striking the far end of the grate. The required clear length of the bar (L_b) can be computed by the experimentally determined formula:

$$L_b = \frac{V}{2} (d + d_b)^{0.5} \qquad (7\text{-}1)$$

where:

V = mean approach velocity in the width of the grate opening, in feet per second;

L_b = length of clear opening of grate, in feet;

d = depth of flow at the curb, in feet;

d_b = depth of the bar, in feet;

Figure 7-5 is a graphical solution of the above equation for pavement with a roughness coefficient n = 0.015 and depth of bar at 3-1/2 inches. The effect of gutter slope on the length of clear opening needed for an efficient grate is illustrated by Example 1.

EXAMPLE 1

Given: Pavement and gutter cross slope, S_x = 1/4 inch per foot (0.021 foot per foot)' 2-foot concrete gutter; n = 0.015; allowable flow spread on pavement, 6 feet (total width = 8 feet); depth of grate bars, 3-1/2 inches.

Find: Length of clear opening of grate bars required for an efficient grate when longitudinal slope (S_o) is one percent; 10 percent.

Solution:

1. Use Figure 7-5. On upper left diagram for T = 8 feet move horizontally to S_x curve, 0.021.

2. From intersection in Step 1 move vertically downward to S_x curve, 0.021 in left center diagram.

3. From intersection in Step 2, move horizontally to the right diagram and read L_b on the bottom scale. At the intersection of the curve for S_o = 0.01, L_b = 1.1 feet, and at the intersection of curve for S_o = 0.10, L_b = 3.0 feet.

4. For grate inlets, the clogging factor F = 1.6.
For S_o = 0.01, L = L_b(1.6) or 1.76 ft. and
S_o = 0.10, L = L_b(1.6) or 4.8 ft.

Example 1 illustrates that as the gutter slope varies so does the length of clear opening required for 100 percent interception of the water approaching the inlet within the width of the grate.

7.5.1 Capacity of Grate Inlets on a Continuous Grade

The capacity of an undepressed grate inlet can be determined by computing the flow in the section occupied by the grate width.

For straight-line sections, the flow intercepted by an undepressed grate can be computed as explained in Figure 7-5. Example 2 illustrates the procedure.

EXAMPLE 2

Given: Q = 1.5 cfs; pavement cross slope, S_x = 1/4 inch per foot; n = 0.016; longitudinal slope = 2 percent; grate inlet, 30 inches wide by 24 inches long; depth of longitudinal bars 3 inches; no cross bracing; 60 percent clear opening.

Find: Spread on the pavement and the flow intercepted by the grate inlet.

Solution:

1. On Figure 7-5, lay a straight end on $S = 0.02$ and $Sx = 0.021$. Mark intersection on the turning line.

2. Lay straight end on turning point and $Q = 1.5$ cfs. Spread or $T = 7.2$ ft.

3. Depth of flow at curb = $d = TS_x$

$$d = 0.021\ (7.2) = 0.15 \text{ ft}$$

4. Depth of flow at outward edge of grate ($W = 2.5$ ft)

$$d_1 = (T-W)S_x\ ;\ (7.2 - 2.5)\ 0.021 = 0.10 \text{ ft}$$

5. From Figure 7-5 using composite sections in which $S_w = S_x = 0.021$ and $T = T_s = 4.7$ ft; read $Q_s = 0.48$ cfs

6. $Q_w = Q - Q_s$

   $= 1.5 - 0.48 = 1.02$ cfs, provided that the grate is efficient.

7. The grate has the necessary requirements for efficiency if the clear opening L_b is sufficient. The mean velocity in the 2.5-foot section over the grate equals the discharge over the grate divided by the area of the section (0.15 ft. deep at the curb and 0.10 ft. deep at the outer edge of the grate, and a width of 2.5 ft.).

$$V = \frac{1.02 \text{ cfs}}{0.31 \text{ ft}^2} = 3.3 \text{ fps}$$

The clear opening required by equation (7-1) is

$$L_b = \frac{V}{2}\ (d+d_b)0.5 = \frac{3.3}{2}\ (0.15 + 0.25)^{0.5} = 1.0 \text{ ft}$$

The grate opening, 2.0 feet, exceeds that required; therefore, the grate is efficient.

The 2.5-foot grate intercepts 68 percent of the total flow, 1.02/1.5 = 0.68, in Example 2 to intercept 100 percent of the flow a grate 7.2 feet wide is required, or about three times the width required for 67 percent interception.

Ordinarily, the width of the grate remains constant and the spacing of inlets are varied to limit the flow spread on the pavement to the desired quantity. The calculations for determining the discharge for 100 percent interception by a given grate are presented in Example 3.

EXAMPLE 3

Total Interception by Undepressed Grate

Given: The same conditions as in Example 2.

Find: The gutter flow (Q) that would be totally intercepted by an efficient grate inlet, 30 inches wide.

Solution:

1. The flow spread on the pavement (T) is limited to the grate width (2.5 feet) for 100 percent interception.

$$d = TS_x = 2.5\ (0.021) = 0.05\ \text{ft}$$

2. Using Figure 7-2 and composite section flows with T = W = 2.5 ft, S = 0.02 and S_x = 0.021, the intersection of the flow line is off the graph. Using the equation:

$$Q = \frac{0.56}{n} S_x^{1.67}\ S^{0.5}\ T^{2.67}$$

$$Q_w = 0.09\ \text{cfs}$$

Examples 2 and 3 indicate the advantage of designing grate inlets for partial interception. For 100 percent interception under the conditions of these examples, the grate width would have to be tripled, or about 17 separate 30-inch grate inlets, each intercepting 0.09 cfs, would be required.

For composite straight-line gutter sections, the flow intercepted by an undepressed efficient grate is computed as explained in Figure 7-2.

7.5.2 Capacity of Grate Inlets in a Sag Location

A grate inlet in a sag location functions first as a weir having a crest length roughly equal to the outside perimeter (P) along which the flow enters. Bars are disregarded and the side against the curb is not included in computing P. Weir operation continues to a depth (d) of about 0.4 foot above the top of grate and the discharge intercepted by the grate is:

$$Q_i = \frac{3.0\ Pd^{1.5}}{F} \qquad (7\text{-}2)$$

where:

Q_i = rate of discharge into the grate opening, in cfs.
P = perimeter of grate opening, in feet, disregarding bars and neglecting the side against the curb;
d = depth of water at grate, in feet; and
F = clogging factor

When the depth at the grate exceeds about 1.4 feet, the grate begins to operate as an orifice and the discharge intercepted by the grate is:

$$Q_i = \frac{0.67\ A}{F} (2gd)^{0.5} = \frac{5.37Ad}{F}^{0.5} \qquad (7\text{-}3)$$

where:

A = clear opening of the grate, in square feet;
g = acceleration of gravity, 32.2 feet per second;
d = depth of ponded water above top of grate, in feet; and
Q_i = rate of discharge into the grate opening, in cfs.
F = clogging factor

Between depths over the grate of about 0.4 foot and about 1.4 feet, the operation of the grate inlet is indefinite due to vortices and other disturbances. The capacity of the grate is somewhere between that given by the above equations.

Because of vortices and the tendency of trash to collect on the grate, the clear opening or perimeter of a grate inlet should be 100% greater (a factor of 2.0) than that required by the equations in order to remain below the design depth over the grate. (See Table 7-1)

These equations are solved graphically with Figure 7-6.

EXAMPLE 4

Given: Grate inlet, 2 feet by 2.5 feet with 50 percent clear opening, located in a sag with one 2-foot side against the curb.

Find:

1. Depth of water when Q = 2.0 cfs; and

2. Depth of water when Q = 10.0 cfs.

Solution:

1. For Weir operation, compute perimeter of grate opening (P) ignoring the bars and omitting any side over which the water does not enter, such as when one side is against the face of a curb. Divide the result by 2 to allow for partial clogging of the grate.

   P = 2 + 2.5 + 2.5 = 7.0 feet

   Effective P = $\frac{7}{2}$ = 3.5 feet

2. For orifice operation, compute the total area of clear opening (A), excluding area taken up by bars, and divide by 2.0 to allow for partial clogging of the grate.

   Clear opening = .50 (2 x 2.5) = 2.5 square feet

   Effective area = $\frac{2.5}{2.0}$ = 1.25 square feet

3. Enter Figure 7-6 at the abscissas, using curve A with effective P values and curve B with effective A values and read the depth, in feet, of ponding over the grate.

   (1) When Q = 2.0 cfs,

   using P = 2.5 and curve A, d = 0.34 foot.

   (2) When Q = 10 cfs,
   using A = 1.25 ft^2 and curve B, d = 2.5 feet

If the grate has appreciable cross slope so that the side away from the curb is higher than the curb side, the inflow over the side should be determined separately from that over the ends. In weir control, use the depth at the middle of the grates for end inflow and the depth away from the curb for side inflow.

7.6 Curb-opening Inlets

Curb-opening inlets are used in many locations because they offer little interference to traffic and are relatively free from clogging by debris.

The curb-opening inlet discussed in this section is based on the City/County standard specifications. The equations presented apply only if the cross section of the street has a uniform slope to the face of the curb.

7.6.1 Capacity of Curb-opening Inlets on a Continuous Grade

The operation of a curb-opening inlet on a grade is usually described in terms of the ratio of the flow intercepted, Q_i, to the approach flow Q which extends a distance T from the curb face. Q_i/Q can be defined in a dimensionless plot against $L_i/(F_w t)$ where Li is the length of the inlet opening and F_w is the Froude Number related to the depth of the approach flow at a distance W from the curb (See Figure 7-7). This is along a line at the outer edge of the inlet depression. The Froude Number is a ratio of the kinetic energy to the gravity force acting on the flow in the gutter.

Figure 7-8 is drawn for a roadway cross slope, $S_x = 0.015$ and $W = 2$ ft. Note that $Q_i/Q = L_i/L_1$ up to the point where the parameter $L_i/F_wT = 0.4$. Beyond that point, the relationship changes abruptly to a curved line for which $Q_i/Q = (L_i/L_3)^{0.4}$. L_i is the value of L_i where the straight line intersects $Q_i/Q = 1.0$, while L_3 is the value of L_i where the curved line intersects $Q_i/Q = 1.0$. L_2 is the value of L_i at the breakpoint between the straight and curved lines.

From the diagram, it is apparent that if we know the value of F_wT and L_i, the value of Q_i/Q can be found from the ordinate scale. Remember that this diagram is for specific values of S_x and W. The position of the S_x line varies with these variables in accordance with the equations on Figure 7-8 while the position of the curved line remains fixed. Solutions for Q_i/Q may be read from Figure 7-9 for W = 2 feet or may be computed using an electronic calculator and the equations tabulated in the examples in Table 7-2. Table 7-2 also presents the equations for F_w and Q in terms of the cross-section variables. It should be noted that the height of the inlet can be 6" or 8" and does not affect this procedure.

Use of Figure 7-9

An example in dotted lines illustrates the use of Figure 7-9. The depth of flow d_w, in the street section at a point W ft from the curb face is the starting point.

1. Enter with S_x (T-2) at top.
2. Follow down to line for n, normally 0.016.
3. Move (left or right) across to longitudinal slope, S.

4. Follow down to spread, T; this establishes a horizontal line.

5. If Q_i/Q is given, enter with Q_i/Q (upper right) and follow across to Limit line A, or line for S_x, whichever is intersected first.

6. Move down to lower margin (Q_i/Q = 0.1) and then diagonally to intersection with line from Step 4.

7. Follow down to length of inlet, L_i.

8. The horizontal line can be extended to line L_3 for 100% interception, then down to L_i.

9. If length of inlet is given, enter with that length, move up to horizontal line established in Step 4, diagonally to Qi/Q = 0.1, then vertically to S_x (or line A) and across to Q_i/Q.

Composite Section

It is a common practice to build gutters with steeper cross slopes than the pavement. This increases the depth of flow at the curb and the discharge for a given spread. Assuming that the width of the gutter is the same as the width W of the inlet depression, it is probable that the increment in flow caused by the steepened gutter cross slope will be picked up by the inlet. On that basis, a family of curves has been calculated by the method given in Figure 7-9 to give the ratio of the increased flow to the flow in the straight section in Figure 7-10. To use this figure, first estimate Q by methods described previously, then, knowing S_x and T, read Q_i/Q on the ordinate scale and multiply by Q to obtain the increment in discharge to be added to Q_i.

Hydraulics of Inlet

It may be helpful to discuss the physical significance of the dimensionless curves in Figure 7-8. If we think of F_wT as a constant for a given flow case, then the abscissa scale is the length of inlet divided by that constant. We could recalibrate the scale to read directly in feet for inlet length. For short inlets, up to the length L_2 where the curve breaks, the inlet is acting as a weir. In fact, the flow intercepted is practically the same as would be intercepted by the same inlet at a sump, using the modified weir equation for that case. The major part of the flow is intercepted (60% or more depending on S_x) up to the length L_2. For greater lengths of inlet, the remainder of the flow moves in gradually as indicated by the lesser increments of Q_i as length increases.

Table 7-2

Computations for Curb - Opening Inlets

STANDARD SECTION

W=2 ft.
n=0.016
S_x=0.02

(1) $L_1 = 2.49\ S_x^{0.3}\ F_w T$

(2) $L_2 = 3.27\ S_x^{0.5}\ F_w T$

(3) $L_3 = 1.65\ F_w T$

(4) $F_w = 16.4 \left[(T-2)\ S_x\right]^{1/6} S_o^{1/2}$

(5) $Q = 35 S_x^{5/3}\ T^{8/3}\ S^{1/2} = .0515\ T^{8/3}\ S_o^{1/2}$

(6) $T = \left[\frac{Q}{35S^{1/2}}\right]^{3/8} S_x^{-5/8} = 3.04 \left(\frac{Q}{S_o^{1/2}}\right)^{3/8}$

(7) $\frac{Q_i}{Q} = \frac{L_i}{L_1}$

(8) $\frac{Q_i}{Q} = \left(\frac{L_i}{L_3}\right)^{0.4}$

INLET NO.	S	$\frac{Q}{S^{1/2}}$ (cfs.)	Q (cfs.)	T (cfs.)	$\frac{F_w}{S^{1/2}}$	F_w	F_wT (ft.)	L_1 .770 (ft.)	L_2 .462 (ft.)	L_3 1.65 (ft.)	$\frac{Q_i}{Q}$	Q_i (cfs)	$\frac{Q_2}{Q}$ .600 (cfs)	L_i (ft.) $Q_i < Q_2$ $L_1 < L_2$	L_i (ft.) $Q_1 > Q_2$ $L_1 > L_2$	USE L_1 (ft.)	ACT. Q_i (cfs)	$Q-Q_i$ $=Q_c$ (cfs)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
1	.01	23.9	2.39	10.0*	12.1	1.21	12.1	9.3	5.6	20.0	1.0 *	2.39	1.43		20.0	20.0	2.39	0
1	.01	23.9	2.39	10.0*	12.1	1.21	12.1	9.3	5.6	20.0	0.80*	1.91	1.43		11.4	12.0	1.95	0.44
2			2.83	10.7	12.2	1.22	12.2	9.4	5.6	20.1	0.81	2.26	1.70		11.9	12.0	2.30	0.53
3			2.92	10.8	12.3	1.23	12.3	9.5	5.7	20.3	0.81	2.37	1.75		11.9	12.0	2.37	0.55
3 alternate											0.99					20.0	2.90	0.02
4	.03	23.9	4.14	10.0	12.9	2.24	22.4	17.2	10.3	37.0	0.58	2.40	2.48	10.0*		10.0*	2.40	1.74
4 alternate 1											0.80*	3.31			21.1	22.0	3.36	0.78
4 alternate 2			0.65	5.0		2.05	10.25			16.9	0.80*	0.52	0.39		10.0*		0.53	0.12

(See text for sequence of computations)
* Required criteria

One foot is 0.3048m One cubic foot is 0.0283m3

Computation Procedure

For those who prefer using an electronic calculator, Table 7-2 illustrates the sequence of steps. As a rule, the designer will be working with a standard inlet and cross section for which S_x, n and W are fixed. The heading Equations (1), (2) and (3), taken from those in Figure 7-8, reduce to the numerical coefficients in the heading of Columns (9), (10) and (11).

Inlet 1:

On the first line, Inlet 1, the designer is to find the inlet length required for 100% interception on a 1% grade with T = 10 ft. The starred numbers represent the required criteria. Column 3 is used if there are a succession of grades for which Q is computed by Equation (6). Similarly, Column 6 is for F_w computed by Equation (4). Multiplying F_wT in Column 8 by the coefficients in the heading of Columns 9, 10 and 11 gives characteristic lengths L_1, L_2 and L_3, respectively. As stated, $Q_i/Q = 1$, so Q_i in Column 13 equals Q. Q_2/Q for the standard conditions is simply 0.462Q/0.770 = 0.600Q, as recorded in the heading for Column 14. Either Column 15 or Column 16 is used to record L_i depending on whether Qi is less than or greater than Q_2 (or in the case where L_i is given, or whether L_i is less than or greater than L_2). In this case, $Q_i > Q_2$, so L_i is computed by Equation (8). Since $Q_i/Q = 1$, then $L_i/L_3 = 1$, or $L_i = L_3$ and $L_i = 20$ as taken from Column 11. Actually, for 100% interception, one may go directly from Column 11 to Column 16. Column 17 records the selected length L_i, usually as a multiple of 5 feet depending on design standards. In the next example the computed length 11.4 becomes 12. If desired, Q_i can then be recomputed by Equation (7) or (8) in Column 18 and subtracted from Q to give the carry-over discharge, Q_c.

Inlet 2:

In the next three examples, the independent variables are the same as the first example, except that $Q_i/Q = 0.8$. For inlet 1, the required length is reduced to 12 feet with $Q_c = 0.44$ cfs. Assuming the increment in runoff for the next subwatershed is the same as for inlet 1, the second inlet will then have $Q = 2.39 + 0.44 = 2.83$ cfs. This requires a recomputation of T as $(2.83/0.10)^{3/8} \times 3.04 = 10.7$ feet in accordance with Equation (6). L_3 changes slightly to 20.1. It is now assumed that the same size of inlet will be used again, so Q_i/Q is computed as $(12/20.1)^{0.4}$ making $Q_i = 0.81\ (2.83) = 2.30$ cfs and $Q_c = 0.53$ cfs. For inlet 3, again using 12 ft, the adjusted value of T becomes 10.8 and $L_3 = 20.3$, which leaves $Q_i/Q = 0.81$ and $Q_i = 2.37$ cfs. The flow intercepted has now become substantially equal to the increment in runoff for the contributing intervening watershed.

Inlet 3:

Supposing the Inlet 3 is just above an intersection making carry-over flow undesirable, the third inlet may be increased to 20 ft to intercept practically all the flow.

If 0.55 cfs can be allowed to pass by Inlet 3, then a cost saving with three 12-foot inlets can be realized by assuming $Q_i/Q = 0.80$.

Inlet 4:

In Inlet 4, it is assumed that a 10 foot inlet is to be used and Q_i/Q is to be found. In this case $L_i = L_2$ so Equation (7) is used. For alternate 1, the length for $Q_i/Q = 0.8$ is to be computed.

For Inlet 4, alternate 2, the problem is to find T which would enable the 10-foot inlet to intercept 80% of the flow. This would indicate how far upstream the inlet would have to be moved to reduce Q to that amount. In this case, $(10/L_3)^{0.4} = 0.80$ so $L_3 = (1/0.80)^{2.5} \times 10 = 17.5$ which must equal $1.65\ F_wT$.

Therefore, $F_wT = 17.5/1.65 = 10.6$. As a first trial, assume $F_w = 2.2$ making $T = 10.6/2.2 = 4.8$. Substituting this T in Equation (4), we find $F_w = 2.03$. A second trial with $F_w = 2.1$ yields T = 5 and gives a computed $F_w = 2.05$ which is close enough. Taking T = 5 by Equation (5), $Q = 0.515(5)^{8/3} \times 0.03^{1/2} = 0.65$ cfs. This is a very small discharge, demonstrating that a 10-foot inlet is ineffective on a 3 percent (0.03) grade. Rather than moving the inlet that far upgrade, it would obviously be more economical to go to a 22-foot inlet at the original location and save the extra length of pipe.

Adequacy Checks for Storms that exceed Design Base

In checking drainage systems for performance with storms greater in magnitude that the design storm, note that the spread of flow on the pavement increases as $Q^{3/8}$, other variables remaining constant. Thus, if runoff is doubled, spread increases only $2^{3/8}$ or 1.3 times. Assuming the inlet has been designed for Q_i/Q = 0.80, this would reduce to about 0.7, but Q_i would increase about (0.7/0.8)2 = 1.75 times. One would then have to check the pipe capacity and particularly the head loss for flow entering the pipe, to see if the greater Qi could be accepted. Depending on consequences of street flooding, consideration might be given to increasing the pipe capacity.

7.6.2 Capacity of Curb-opening Inlets in a Sag Location

The capacity of curb-opening inlets in a sag location depends upon the depth of water at the inlet and the inlet geometry. The inlet operates as a weir until the water submerges the entrance. When the water depth exceeds about 1.4 times the height (h) of the curb-opening entrance, the inlet operates as an orifice. Between weir-type operation and orifice-type operation, the capacity is indeterminate. Figure 7-11 will give the depth of ponding measured at the curb, just above the depressed area. The use of this chart is explained in Example 5.

When the inlet is <u>not</u> depressed, the approximate capacity can be computed by the weir equation:

$$Q_i = 3.0\ L_i d_i^{1.5} \qquad (7\text{-}4)$$

where:

L_i = length of clear opening, in feet;

Q_i = capacity of the inlet, in cfs.

d_i = depth of water above inlet lip, in feet (inlet not depressed);

When the depth at the opening exceeds 1.4 h, the capacity may be computed by the equation:

$$Q_i = 0.67A\ [2g\ (d_i - \frac{h}{2})]^{0.5}$$

where:

A = area of opening, in square feet (hL_i);

d_i = as above (inlet not depressed);

h = height of opening, in feet.

EXAMPLE 5

Given: A curb-opening inlet in a sag; pavement cross slope 0.02; concrete, broom finish (n = 0.016); depression, width = two foot, amount three-inches; height of inlet opening = 0.67 foot; design discharge from both sides of the inlet, Q_1 = 2 cfs, Q_2 = 8 cfs; total Q = 10 cfs.

Find: Maximum depth of ponding (d_{max}) for L_i = 5 feet, 10 feet and 15 feet (clogging neglected in this example).

Solution:

1. From Figure 7-11 the maximum ponding depths are:

L_i	15	feet	10	feet	5	feet
d_{max}	0.29	foot	0.38	foot	0.56	feet
$T=d_{max}/S_x$	14.5	feet	19.0	feet	28.0	feet

2. The maximum depth of ponding at the curb opening may be exceeded in the approach gutter, particularly for low flows. The depth of ponding in the gutter can be checked at the point where the gutter slope is 0.002 by using Figure 7-12.

   For L_i = 10 feet, Q_1 = 2 cfs, d_{max} = 0.38 foot , and d = 0.26 foot.

   The gutter depth for Q_1 is less than the ponding depth at the inlet and water will back up in the gutter channel.

   For Q_2 = 8 cfs, d_{max} = 0.38 foot and d = 0.44 foot (Figure 7-12).

   The gutter depth for Q_2 is greater than the ponding depth at the inlet and the water profile tends to draw down on approaching the inlet.

   For L_i = 15 feet, d_{max} = 0.29 foot and d = 0.26 foot
   For L_i = 5 feet, d_{max} = 0.56 foot and d = 0.44 foot

In addition to illustrating the use of the sag curves, Example 5 shows the possible necessity of picking up gutter flow before it reaches the low point of the sag vertical curve. Flow spreads on the pavement (T) and depths at the curb (d_{max}) noted in step 2 could not be tolerated on a high speed highway. The more common application of the sag curves would be in designing curb-opening inlets or their spacing to keep the depth of ponding and flow spread on the pavement within tolerable limits.

7.7 Slotted and Combination Inlets

The capacity of an unclogged combination inlet on a continuous grade using an efficient grate, is not appreciably greater than that of the grate alone. Therefore, the capacity is computed by ignoring the curb-opening and computing the capacity of the grate opening alone.

Conner and Larson found that diagonal grating bars were superior to transverse bars on combination inlets. Neither investigation compared a diagonal bar grate with the hydraulically superior longitudinal bar grate of comparable size.

7.8 Inlet Locations

In general, inlets should be placed at all low points in the gutter grade and at intersections to prevent the gutter flow from crossing traffic lanes of the intersecting road. In urban locations, inlets are normally placed upgrade from pedestrian crossings to intercept the gutter flow before it reaches the

crosswalk. Where pavement surfaces are warped, as at cross streets, ramps, or in transitions between superelevated and normal sections, gutter flow should be picked up before the cross slope of the pavement begins to change in order to decrease water flowing across the roadway and to prevent or limit icing conditions.

7.8.1 Spacing of Inlets on a Continuous Grade

Inlets should be spaced so as to limit the spread of the water on the pavement to the criterion outlined in Chapter 6.

With the maximum flow spread fixed and with a given pavement cross slope and longitudinal slope, the flow in the gutter is also fixed and can be calculated as previously discussed. The spacing of inlets is equal to the length of pavement needed to generate the discharge corresponding to the allowable spread of flow on the pavement. The flow bypassing each inlet must be included in the flow conveyed to the next inlet.

EXAMPLE 6

Curb-Opening Inlets

Given:

A major arterial 4-lane section with 2 foot curb and gutter section, n = 0.016, width = 4(12) + 2(2) = 52 ft. The pavement cross slope, S_x = 0.02 ft/ft, longitudinal slope S_o = 0.02; composite C for pavement and adjacent areas = 0.6. Rainfall intensity for 100-year storm, i = 5.82. The contributing drainage area is 100′ to each side of the centerline. Discharge in gutter, Q = 8.3 cfs. Length of roadway above first inlet = 400 feet.

Find: Inlet length for 50% interception and spacing between inlets where w = 2.0 ft. and a = 2 inches;

1. Flow spread on pavement = 14 feet. (From Table 6-1)

2. Compute inlet size for 50% interception
   d = 0.28 ft
   n = 0.016
   S_o = 0.02 ft/ft
   T = 14 ft
   Q_i/Q = 0.5
   S_x = 0.02 ft/ft

From Figure 7-9, L_i = 10 ft

3. The flow intercepted by the inlet must be contributed to by the intervening pavement area if the design flow spread is obtained. The area of pavement required is:

$$A = \frac{0.5(8.3)}{(0.6)(5.82)} = 1.19 \text{ ac or } 51{,}768 \text{ sq ft}$$

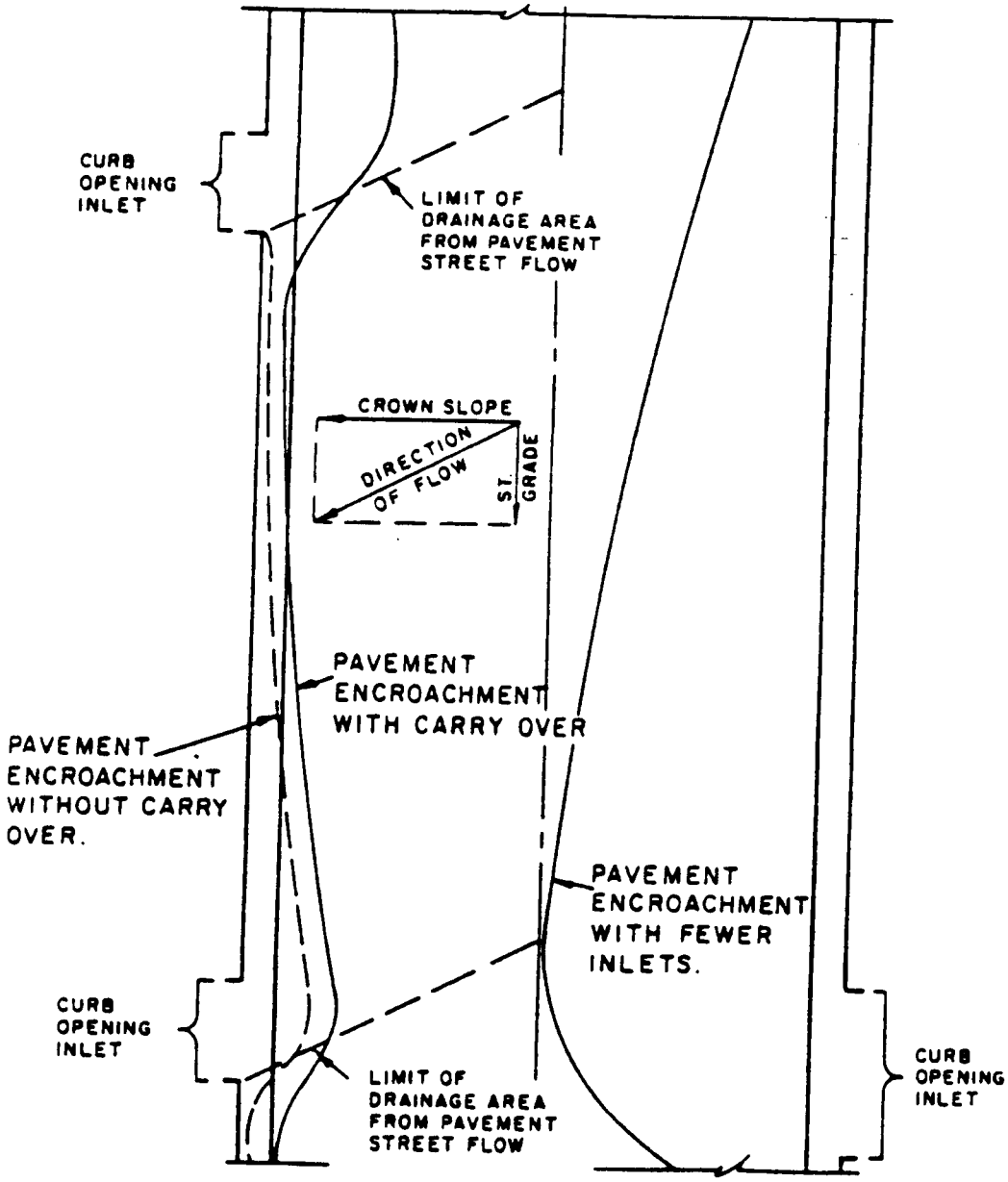
4. The spacing of sucessive inlets after the upstream inlet, assuming no centerline or crown over topping is allowed to occur, is:

$$L = \frac{51768}{100} = 518 \text{ feet}$$

7.9 DEFINITION OF SYMBOLS FOR CHAPTER 7

Symbol	Units	Description
A	sq-ft	Area of cross section
A	ac	Drainage area tributary to the point under design
a	in	Depression of inlet lip below the extension of gutter flow line
C		Runoff coefficient in the Rational formula
d or y	ft	Depth of gutter flow at the curb line
d_b	ft	Depth of grate bars
di	ft	Depth of water above inlet lip at adjacent location where the gutter is not depressed
F_w		Froude Number based on depths and velocity of uniform gutter flow at distance W from curb face
g	ft/sec2	Acceleration of gravity = 32.2 ft/sec^2
h	ft	Height of curb-opening inlet
i	in/hr	Average rainfall intensity during the estimated runoff time of concentration
L	ft	Length of roadway between inlets

L_b	ft	Length of clear opening of grate bars
L_i	ft	Length of curb-opening inlet
L_1	ft	Length of inlet when $Q_i/Q = 1$ on first section of curve for Q_i/Q = function $L_i/(F_wT)$
L_2	ft	Length of inlet at point of intersection of first and second sections of Q_i/Q = function $L_i/(F_wT)$
L_3	ft	Length of inlet when $Q_i/Q = 1$ on second section of Q_i/Q = function $L_i/(F_wT)$
n		Roughness coefficient in the modified Manning formula for triangular gutter flow
P	ft	Perimeter of grate opening, neglecting bars and side against curb
Q	cfs	Rate of discharge in gutter
Q_c	cfs	Rate of discharge bypassing the inlet (carry over)
Q_i	cfs	Rate of discharge intercepted by inlet or total flow for sump inlets
Q_2	cfs	Value of Q_i and L_2
Q_i/Q		Interception rate of curb-opening inlet (also called efficiency of inlet)
S_o or S	ft/ft	Longitudinal slope of pavement
S_x	ft/ft	Cross slope of pavement
T	ft	Top width of water surface (flow spread on pavement)
T_c	min	Runoff time of concentration of watershed
V	ft/sec	Mean velocity of flow
W	ft	Width of depression for curb-opening inlets
X	ft	Distance from curb face in gutter flow computations
Z		Reciprocal of cross slope



PLAN - TYPICAL
STREET WITH INLETS

11-15-68
Denver Regional Council of Governments

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Gutter and Pavement Flow Patterns

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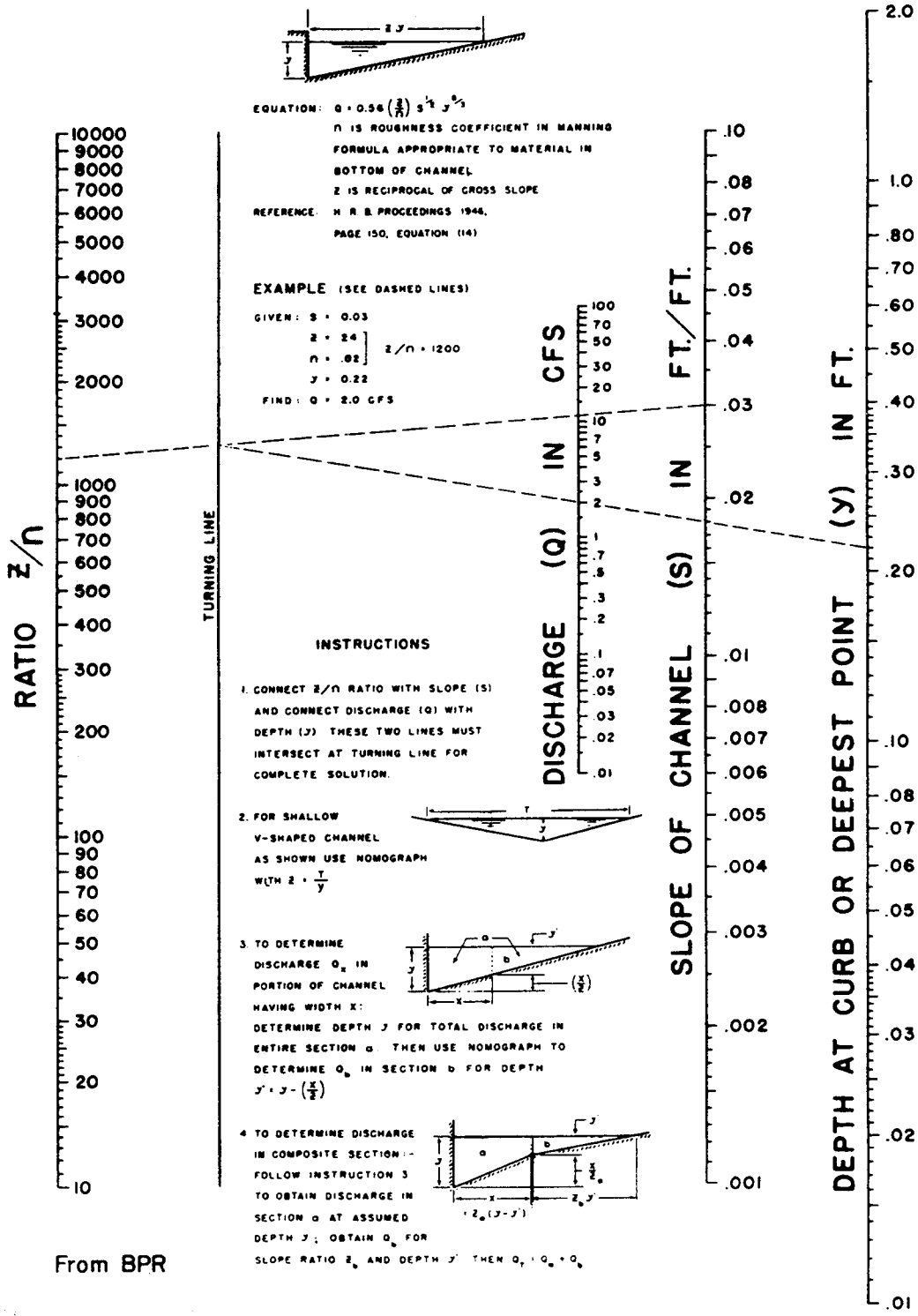
Figure
7 - 1



NONOGRAPH FOR FLOW IN TRIANGULAR GUTTERS
(From U.S. Dept. of Commerce, Bureau of Public Roads, 1965)

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NOMOGRAPH FOR FLOW IN TRIANGULAR GUTTERS.

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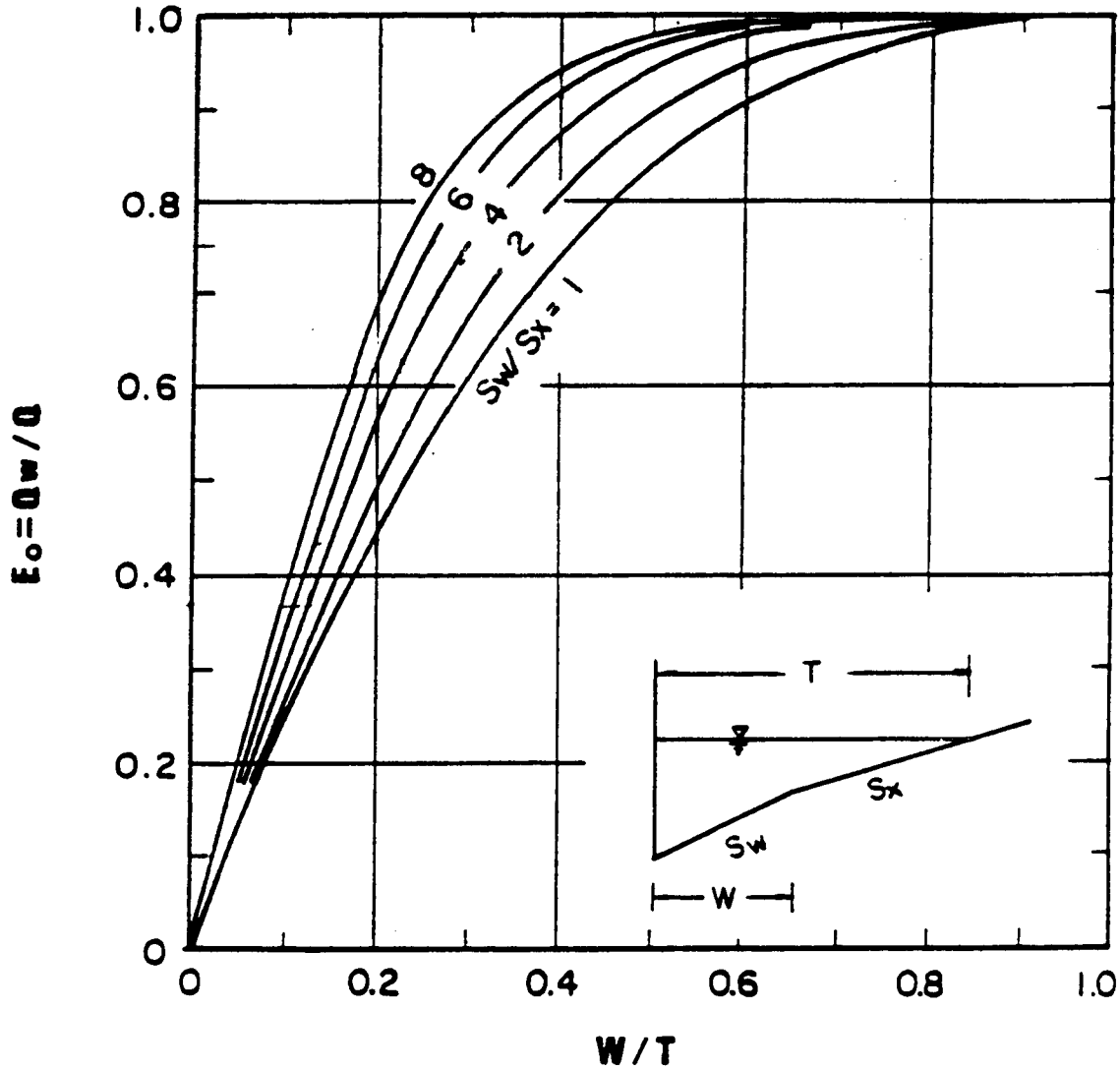
Figure: 7 - 2



REFERENCE : FHWA Hydraulic Engineering Circular No. 12

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Ratio of Frontal Flow to Gutter Flow

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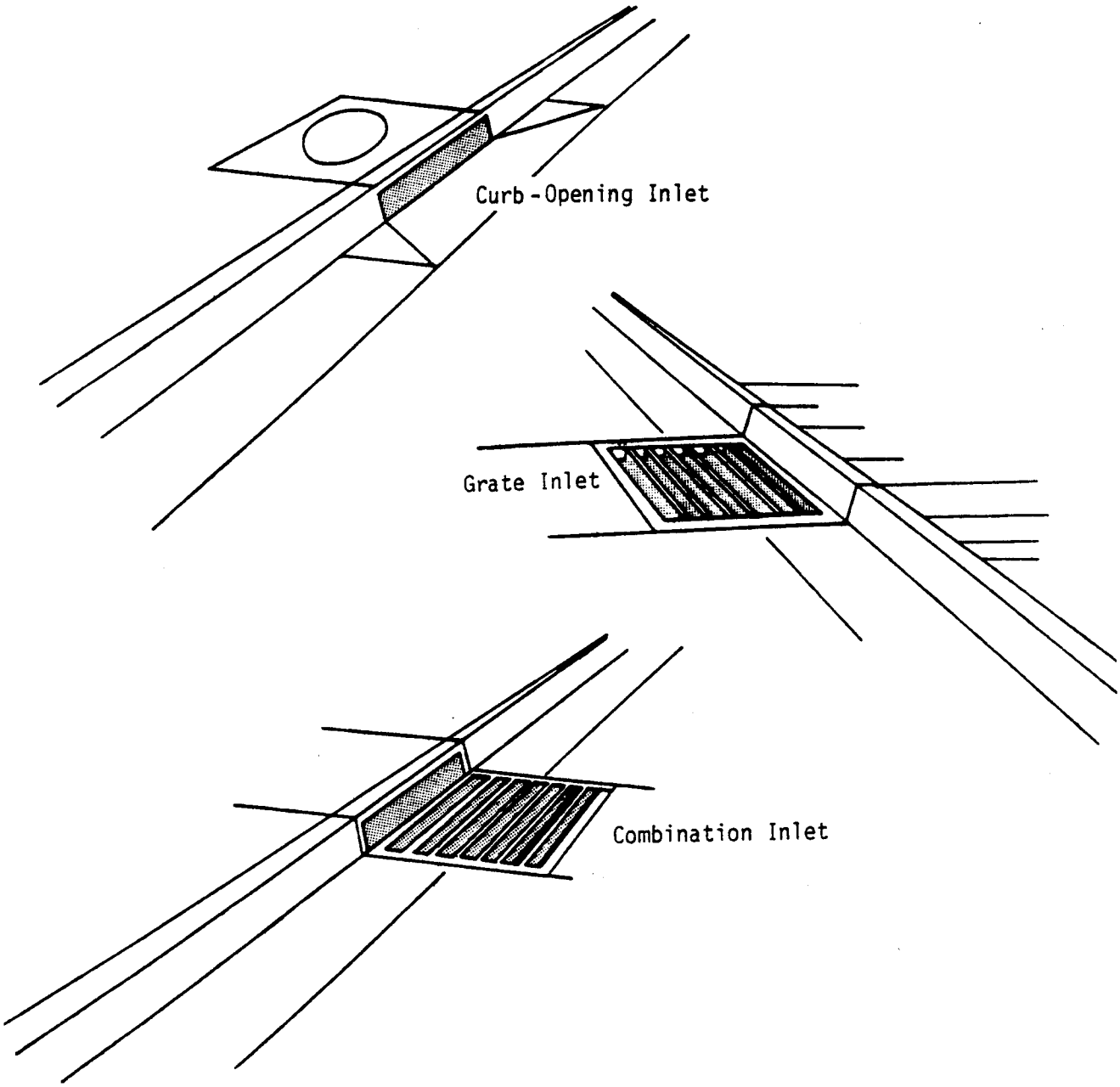
Figure
7 - 3



Inlet Types

REFERENCE : Izzard, Carl. f., Report presented at the Annual Meeting of the National Transportation Board, January 1977; Simplified Method For Design of Curb-opening Inlets

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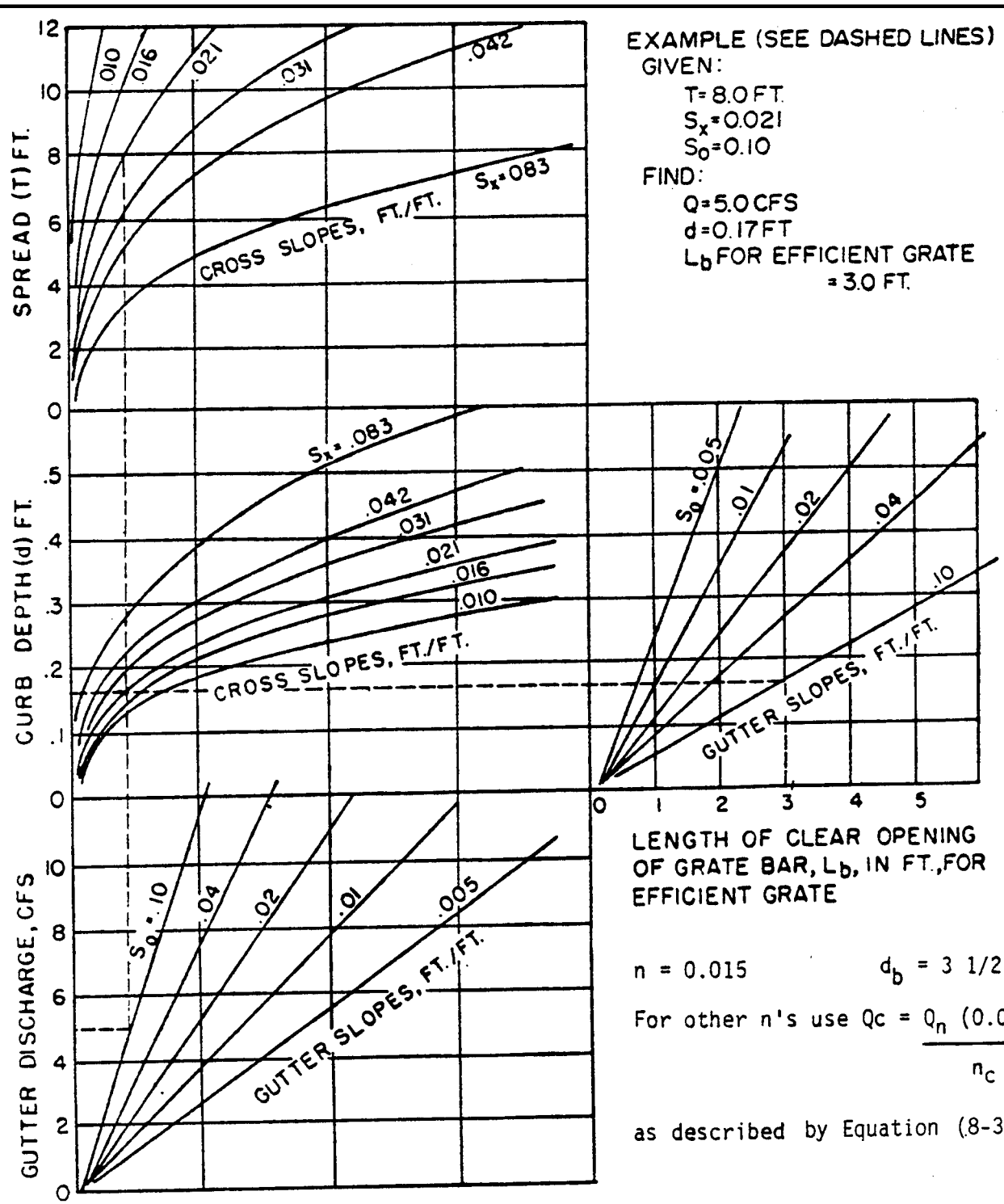
Figure
7 - 4



EXAMPLE (SEE DASHED LINES)

GIVEN:

$T = 8.0$ FT.

$S_x = 0.021$

$S_o = 0.10$

FIND:

$Q = 5.0$ CFS

$d = 0.17$ FT

L_b FOR EFFICIENT GRATE $= 3.0$ FT.

$n = 0.015$ $\qquad d_b = 3\ 1/2$ in.

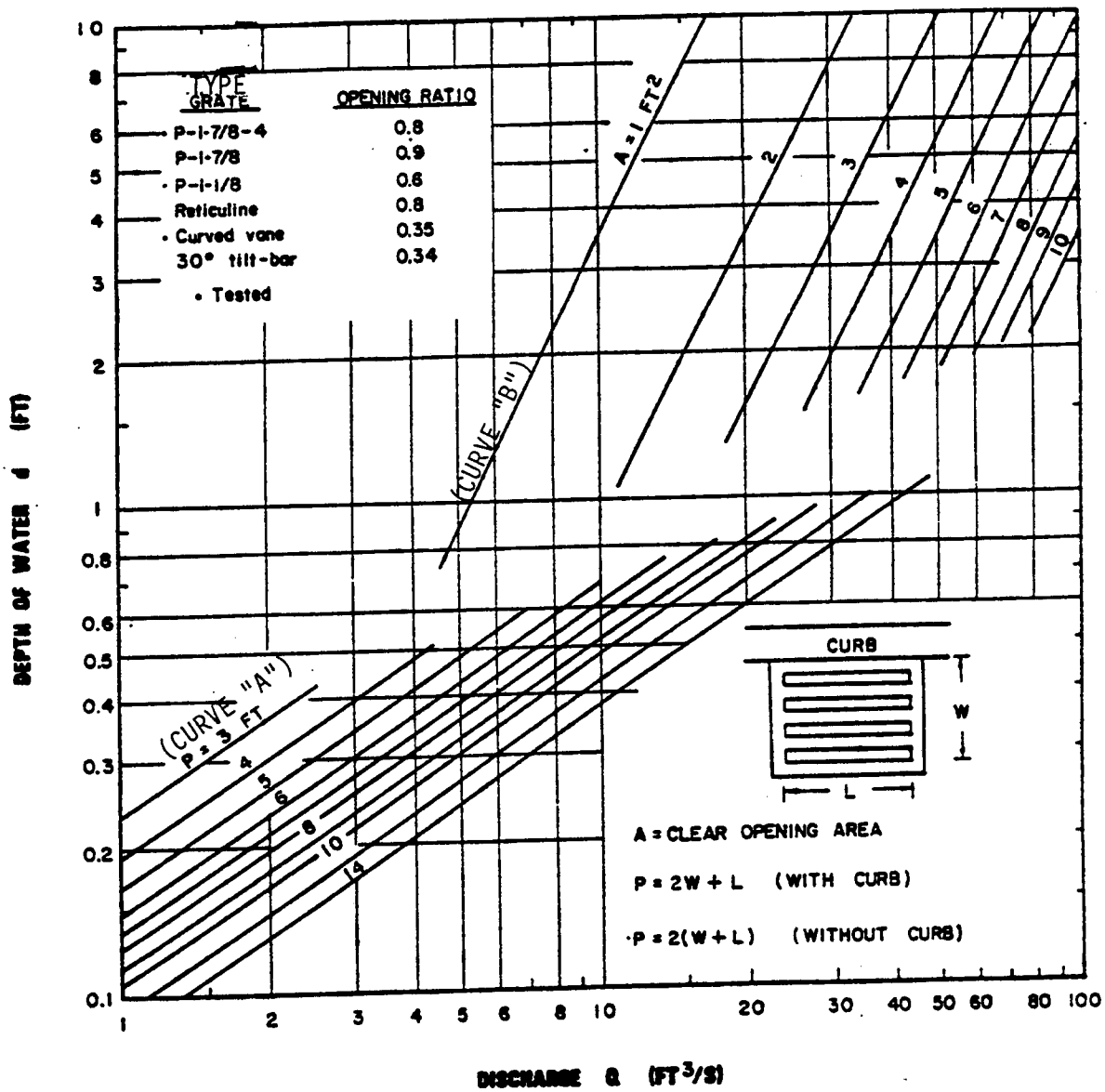
For other n's use $Q_c = \dfrac{Q_n\ (0.015)}{n_c}$

as described by Equation (8-3)

REFERENCE :

Federal Highway Administration, Hydraulic Engineering Circular No. 12, Washington DC, 1969; Drainage of Highway Pavements

Flow Characteristics Curves for Grate Inlets.



Reference: USDOT, March 1984
HEC NO. 12

NOTE: Use with effective P or A

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Hydraulic Capacity of Grate Inlet in Sump

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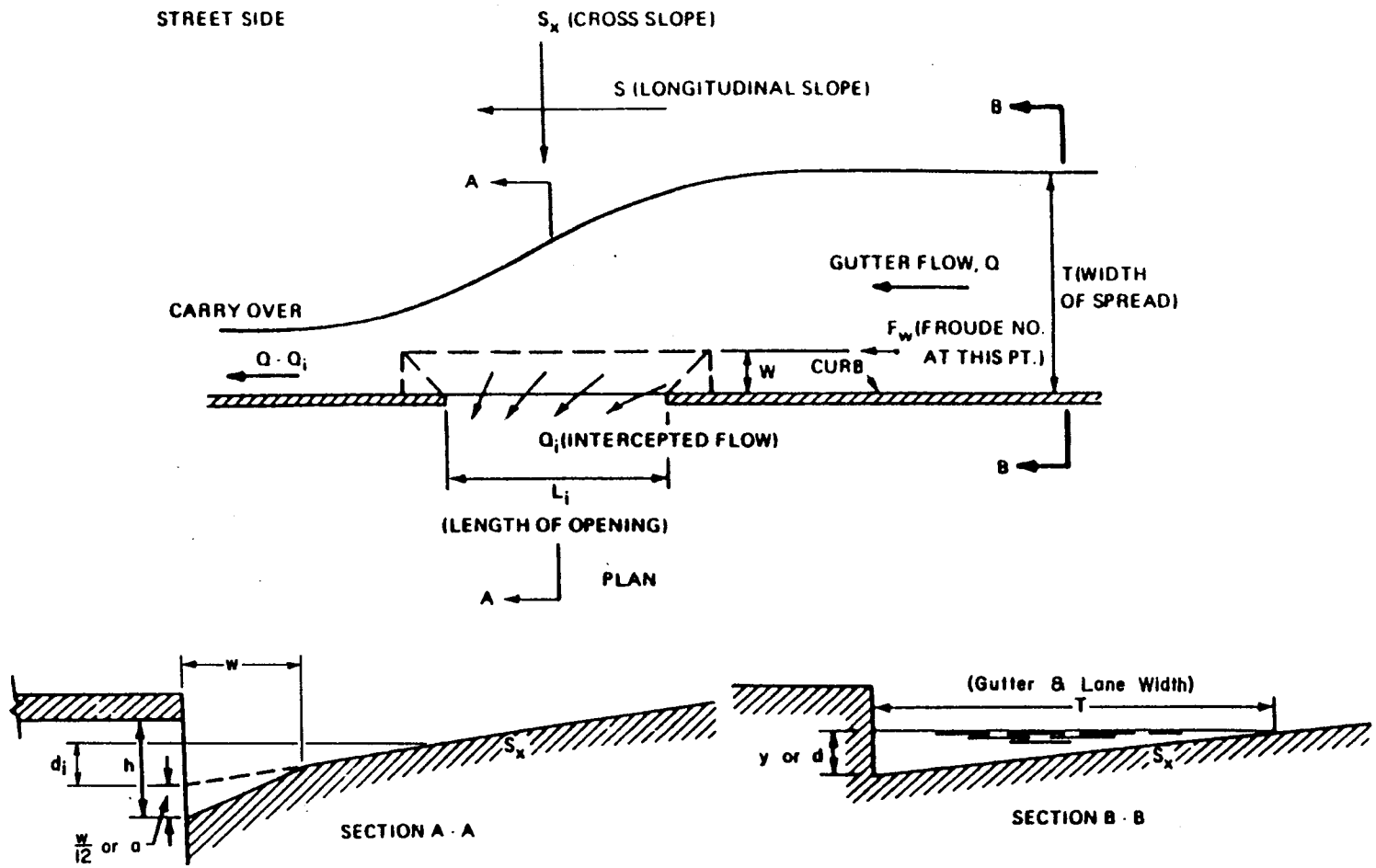
Figure
7 - 6



REFERENCE : Izzard, Carl. f., Report presented at the Annual Meeting of the National Transportation Board, January 1977; Simplified Method For Design of Curb-opening Inlets

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Graphical Definition of Symbols

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Figure
7-7

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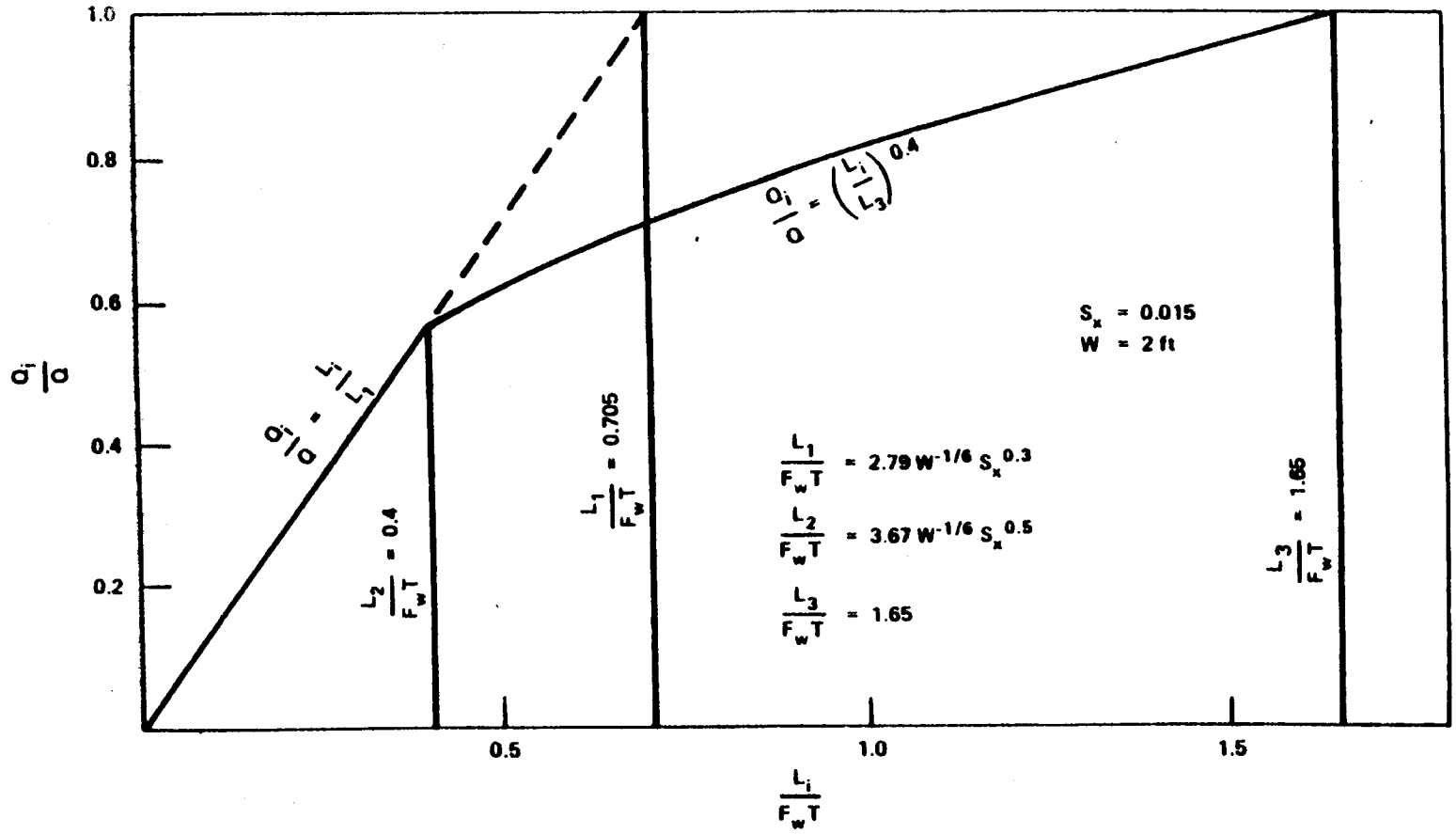

Dimensionless Graph of Q_i/Q vs. L_i/F_wT

REFERENCE :

Federal Highway Administration, Hydraulic Engineering Circular No. 12, Washington DC, 1969; Drainage of Highway Pavements

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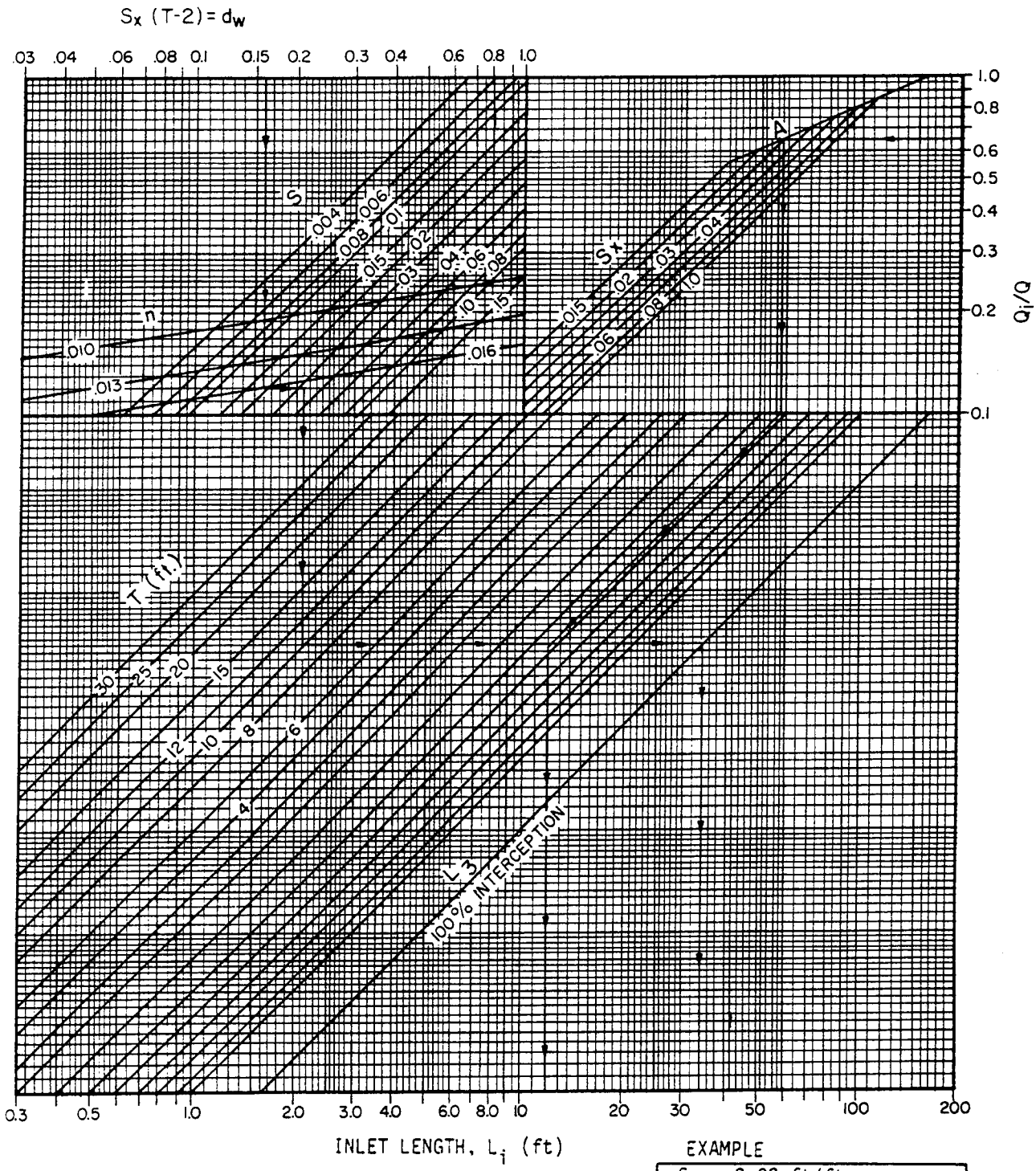
Figure
7 - 8



This chart assumes, w=2 ft., a = 2" and h=6in.

REFERENCE :

Izzard, Carl. f., Report presented at the Annual Meeting of the National Transportation Board, January 1977; Simplified Method For Design of Curb-opening Inlets

EXAMPLE

Given — S_x = 0.02 ft/ft
T = 10 ft.
S = 0.03 ft/ft

Find — L_i = 11.8 ft L_i = 34 ft.
Q_i/Q= 0.65 Q_i/Q=1.0

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CONTINUOUS GRADE
Standard Curb-Opening Inlet Chart

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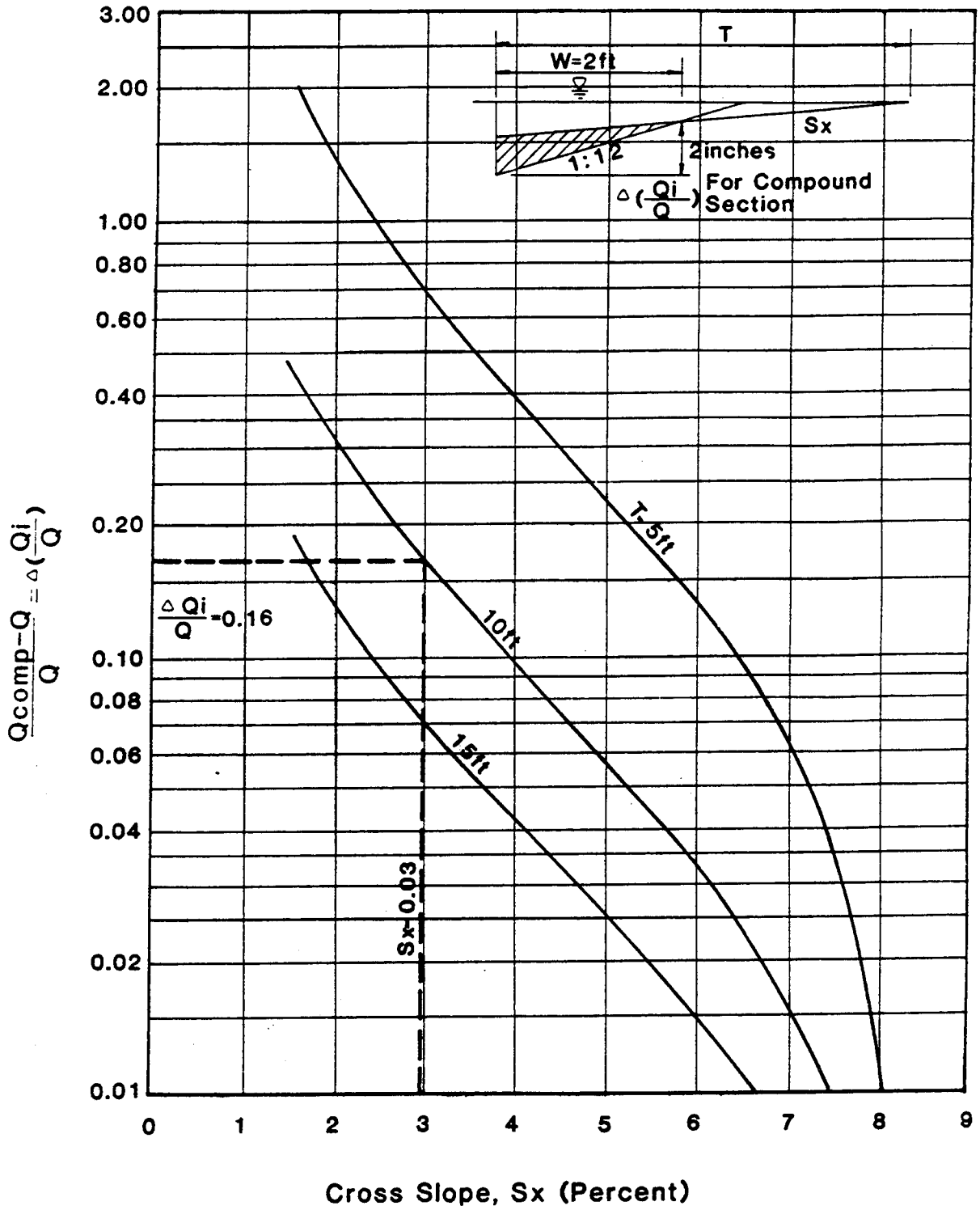
Figure
7-9



REFERENCE : Izzard, Carl. f., Report presented at the Annual Meeting of the National Transportation Board, January 1977; Simplified Method For Design of Curb-opening Inlets

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Additional Inlet Flow Due to Compound Section

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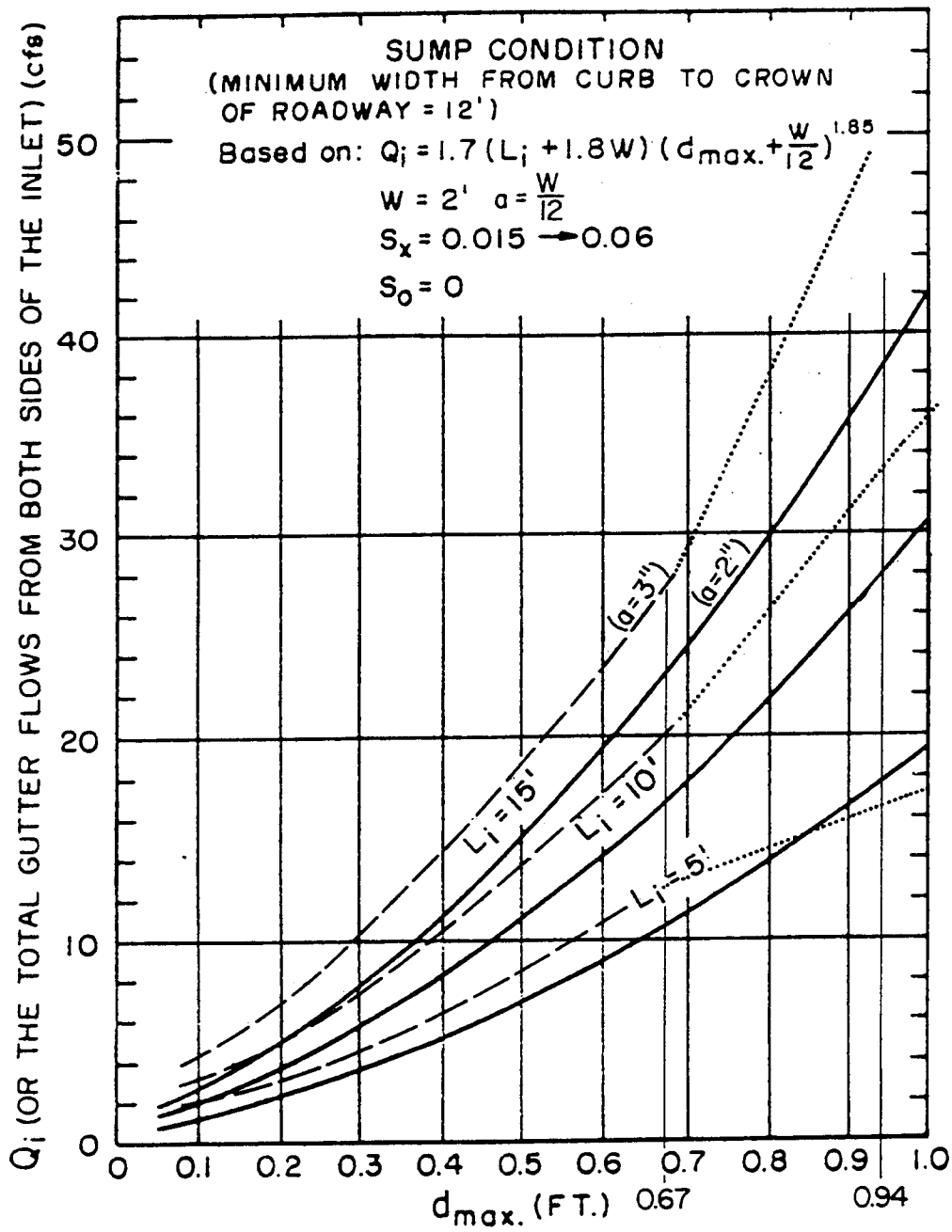
Figure
7 - 10



REFERENCE : Izzard, Carl. f., Report presented at the Annual Meeting of the National Transportation Board, January 1977; Simplified Method For Design of Curb-opening Inlets

— — — (As Modified by El Paso County, per Type R Inlet)

Note: Depth of ponding measured at curb above depressed area ; $a = 3''$, For $d \leq .67$

$Q_i = (1.7 L_i + 6.12)(d_{max} + .25)^{1.85}$; $Q_i = 3.60 L_i (d - .08)^{.5}$ For $d \geq .94$; Note : No Clogging Factor

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Sump Capacity for Curb-opening Inlets

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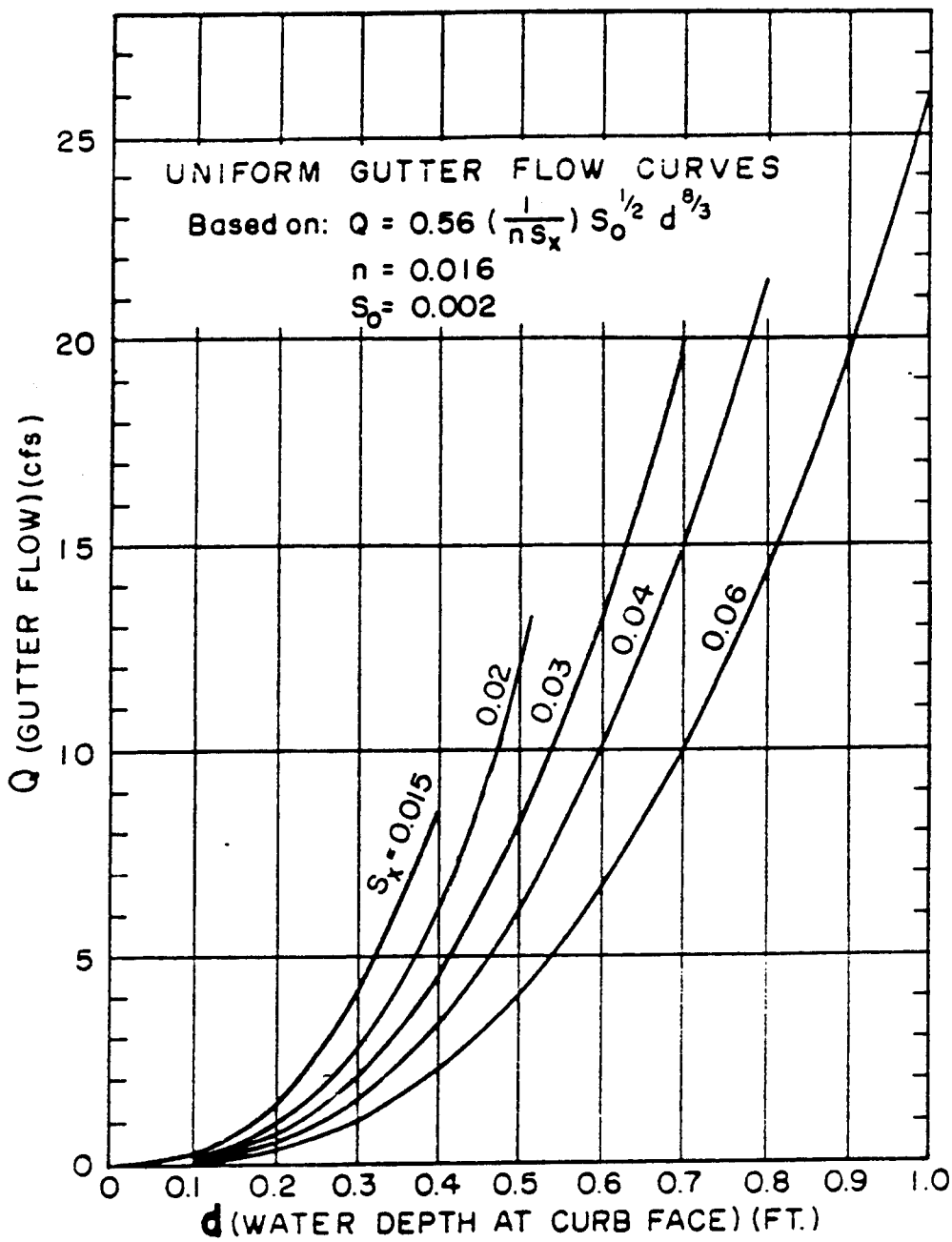

REFERENCE : Izzard, Carl. f., Report presented at the Annual Meeting of the National Transportation Board, January 1977; Simplified Method For Design of Curb-opening Inlets

Uniform Gutter Flow Curves

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Figure
7 - 12

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8. Federal Highway Administration, Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12, Washington, DC, 1969.

9. Izzard, Carl F., Simplified Method For Design of Curb-Opening Inlets, Report presented at the Annual Meeting of the National Transportation Board, January 1977.

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SECTION III

Chapter 8 Storm Drains and Appurtenances

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CHAPTER 8 STORM DRAINS AND APPURTENANCES

8.1 General Statement

The purpose of this chapter is to consider the significance of the hydraulic elements of storm drains and their appurtenances to a storm drainage system. Hydraulically, storm drainage systems are conduits (open or enclosed) in which unsteady and non-uniform free flow often exists. Storm drain conduits are designed for non-pressured (i.e. open channel) flow to satisfy the requirements for unsteady and non-uniform flow.

Storm drains shall be designed by the application of the Manning Equation when flowing in open channel conditions. The hydraulic grade line shall be checked on storm drain designs to determine if the open channel flow assumption is valid. Storm drains operating under pressure flow shall be designed accordingly as outlined in this manual. In the preparation of hydraulic designs, a thorough investigation shall be made of all existing structures and their performance on the waterway in question.

The design of a storm drainage system should be governed by the following six conditions:

1. The system must accommodate the initial surface runoff with minimal damage to physical facilities or interruption of normal traffic. (See Chapter 6)

2. Runoff resulting from the 100-year design storm must be anticipated and discharged without significant damage to physical facilities and minimum interruption of normal traffic.

3. The storm drainage system must have maximum reliability of operation.

4. The construction costs of the system must be reasonable with relationship to the importance of the facilities it protects.

5. The storm drainage system must require minimum maintenance and must be accessible for routine maintenance operations.

6. The storm drainage system must be adaptable to future expansion with minimum additional cost by the consideration of ultimate development on all reaches.

8.2 Flow in Storm Drains

All storm drains shall be designed by the application of the continuity equation and Manning's Equation either through the appropriate charts and nomographs or by direct solutions of the equations as follows:

$Q = AV$; and (Continuity) (8-1)

$$Q = \frac{1.49}{n} AR^{0.67} S_f^{0.5} \quad \text{(Open Channel Manning's)} \quad (8\text{-}2)$$

where:

A = cross-sectional area of pipe, in square feet;

V = velocity of flow, fps;

n = coefficient of roughness of pipe;

R = hydraulic radius = A/W_p, in feet;

W_p = wetted perimeter, in feet;

S_f = friction slope in pipe, in feet per foot, and

Q = pipe flow in cfs.

The following rules shall be observed when designing storm drain sections. These rules are noted as follows:

1. Select pipe size and slope so that the velocity of flow will increase progressively downstream, or at least will not appreciably decrease, at inlets, bends, or other changes in geometry or configuration.

2. Do not discharge the contents of a larger pipe into a smaller one, even though the capacity of the smaller pipe may be greater due to steeper slope. When the capacity of a pipe of same diameter located downstream of a pipe has excess capacity greater than 25% of the design flow, then a smaller pipe no less than 6" in diameter change may be used should it meet design flow requirements. For example, a 36" pipe may be utilized downstream of a 42" pipe should it meet the above flow criteria.

3. At changes in pipe size from a smaller to a larger pipe, match the crowns (inside top surface of the two pipes at the same level rather than matching the flow lines. (When necessary for minimal fall, match the 0.8 diameter point of each pipe.)

4. Conduits are to be checked at the time of their design with reference to critical slope. If the slope of the line is greater than critical slope, the unit will likely be operating under inlet control instead of the originally assumed normal flow.

Pipe Flow Charts, Figures 8-1 through 8-3, are nomographs for determining flow properties in circular pipe. The nomographs are based upon a value of "n" of 0.012 for concrete and 0.024 for corrugated metal. The charts use is demonstrated by the example in Figure 8-1.

For values of "n" other than 0.012, the value of Q should be modified by using the formula below:

$$Qc = \frac{Q_n(0.012)}{n_c} \qquad (8\text{-}3)$$

where:

Q_n = flow from nomograph based on n = 0.012;

n_c = value of "n" other than 0.012; and

Q_c = flow based upon n_c.

This formula is used in two ways. If n_c = 0.015 and Q_c is unknown, use the known properties to find Q_n from the nomograph, and then use the formula to convert Q_n to the required Q_c. If Q_c is one of the know properties, you must use the formula to convert Q_c (based on n_c) to Q_n (based on n = 0.012) first, and then use Q_n and the other known properties to find the unknown value on the nomograph.

EXAMPLE 1

Given: Slope = 0.005; depth of flow, d = 1.8′; diameter, D = 36"; n = 0.018.

Find: Discharge, Q (cfs)

First determine d/D = 1.8′/3.0′ = 0.6. Then enter Figure 8-1 to read Q_n = 34 cfs. Using the formula, Q_c = 34 (0.012/0.018) = 22.7 cfs. (Answer)

EXAMPLE 2

Given: Slope = 0.005; diameter D= 36"; Q = 22.7 cfs; n = 0.018

Find: Velocity of flow (fps)

First convert Q_c to Q_n, so that the nomograph can be used. Using the formula Q_n = 22.7(0.018/0.012) = 34 cfs, enter Figure 8-1 to determine d/D= 0.6. Now enter Figure 8-3 to determine V = 5.2 fps. (Answer)

8.3 Hydraulic Gradients and Profiles of Storm Drains

When storm drains systems are designed for full flow, the designer shall establish the head losses caused by flow resistance in the conduit, changes of momentum, and interference at junctions and structures. This information shall then be used to establish the design water surface elevation at each structure.

The hydraulic and energy grade lines shall be computed and the hydraulic grade line plotted on the construction drawings for the design flow and the design flow should be noted. The friction head loss shall be determined by direct application of Manning's Equation or by use of the appropriate nomographs in this Chapter. Minor losses due to turbulence at structures shall be determined by the procedure described below. The hydraulic grade line shall in no case be closer than one (1) foot to the ground or street surface unless otherwise approved by the City/County. If the storm sewer system could be extended at some future date, the present and future operation of the system must be considered.

8.3.1 Total Energy Head Losses at Structures

Total energy head losses at structures shall be determined for inlets, manholes, wye branches, or bends in the design of full-flow closed conduits. Total energy losses, include minor losses, and the change in velocity head. Minor losses are calculated by the equation $H = K \frac{V^2}{2g}$ for various values of K, see Reference 12.

The minimum head loss used at any structure shall be 0.10 foot, unless otherwise approved.

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.

Head losses for pressure flow in the pipe line may be determined by the Manning's equation for friction loss in pipes:

$$h_f = 185\, n^2 \frac{L}{D^{4/3}} \frac{V^2}{2g}$$

Where: L and D are in feet

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Transition losses per open channel flow can be expressed by:

Contraction; $H_t = 0.1 \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right), \quad V_2 > V_1$

Expansion; $H_t = 0.2 \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right), \quad V_1 > V_2$

Where: V_1 = upstream velocity V_2 = downstream velocity

Transition losses for pressure flow can be expressed by:

Contraction; $H_t = K\left(\frac{V_2^2}{2g}\right) \left[1 - \frac{(A_2)^2}{A_1}\right]^2$

Where: K = 0.5 for sudden contraction
K = 0.1 for well designed transition
A_1 = Cross sectional area of incoming pipe
A_2 = Cross sectional area of outgoing pipe

Expansion: $H_t = K \left(\frac{(V_1 - V_2)^2}{2g}\right)$

Where: K = 1.0 for sudden expansion
K = 0.2 for well designed transition

The above K values are to be used for estimating only. If a more detailed analysis of transition losses are required, then other technical references should be consulted.

In this section, Figures 8-4 to 8-13 enable a designer to include manhole losses in a progressive calculation of pressure elevations proceeding upstream along a storm drain system, determining the water surface elevation, hydraulic grade line, and total energy gradient. These charts and procedures are taken from the Publication "Pressure Changes at Storm Drain Junctions" (Sangster et al, 1958). Full description on the use of these charts is contained in the publication and has been summarized herein.

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 energy grade 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.

8.3.2 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. More detailed instructions for use of the individual charts are presented in the original reference document. The general instructions are as follows:

1. Determine and tabulate the elevation of the outfall pipe pressure line at the branch point or inlet center. 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 manhole from a top inlet, either alone or combining with flow from an upstream pipe.

   d = total depth of water, in feet.
   = (outfall pressure line elevation minus inlet bottom elevation) $+(K)\ V_o^2/2g$.

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 off-set 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_O) or for an entrance formed by the socket end of a standard tongue-and grove concrete pipe.

   Fig. 8-4 - insignificant effect, make no reduction.
   Fig. 8-5 - read directly from chart.
   Fig. 8-6 - reduce K_U by 0.1 for usual proportions of inlet flow; by 0.2 for Q_G about 0.5.
   Fig. 8-7 - reduce K_U and K_L in same manner as Fig. 8-6.
   Fig. 8-8 - insignificant effect, make no reduction.
   Fig. 8-9 - insignificant effect, make no reduction.
   Fig. 8-10, 11, 12 - 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 $h = K \times \frac{V_O^2}{2g}$. The coefficient is a dimensionless number,

   and therefore, the change of pressure will be in feet.

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	8.3.2a	8-4
Flow Straight Through Any Manhole	8.3.2b	8-5
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Manhole on Through Pipeline at Junction of a 90° Lateral Pipe (Large Size Laterals: $D_L/D_O > 0.6$)	8.3.2j,k,l	8-10,8-11
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8.3.2a Figure 8-4

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.

8.3.2b Figure 8-5

Pressure change coefficients are presented in this Figure for use in determining the elevation of the pressure line of the upstream in-line pipe relative to that of the outfall. The pipe center-lines 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 projected 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.

8.3.2c Figure 8-6

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 8-5. As much as half the total flow may enter through a top inlet. The main graph of Figure 8-6 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; there are positive increments for d/D_O less than 2.5 and negative increments for greater depths.

8.3.2d Figure 8-7

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 8-7 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.

8.3.2e Figure 8-8

Pressure change coefficients are presented in this Figure for use in determining the elevation of the pressure 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.

8.3.2f Figure 8-9

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 8-8 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 distance 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, 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.

8.3.2g Figure 8-10, Square Manholes

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.

8.3.2h Figure 8-10, Round Manholes

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.

8.3.2i Figure 8-10, Deflectors

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 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 8-10. These 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.

8.3.2j Figures 8-10 and 8-11, Square Manholes

Pressure change coefficients for use in determining the elevation of the pressure line of the 90° lateral pipe are obtained from Figure 8-10 and the coefficients for the upstream in-line pipe are obtained from Figure 8-11. 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 8-12. 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.

8.3.2k Figures 8-10 and 8-11, Round Manholes

Pressure change coefficients may also be obtained from Figures 8-10 and 8-11 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.

8.3.2l Figures 8-10 and 8-11, Deflectors

Pressure change coefficients are also presented in Figures 8-10 and 8-11 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 8-10 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.

8.3.2m Figure 8-12

Pressure change coefficients are presented in Figure 8-12 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 8-10 and 8-11 may be applied. Figure 8-10 and 8-11 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 Figure 8-12. 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 Figure 8-12 and therefore need not be used.

8.4 Storm Drain System Design Procedure

8.4.1 Preliminary Design Considerations

A. Prepare a drainage map of the entire area to be drained by the proposed improvements. Contour maps may serve as excellent drainage area maps when supplemented by field reconnaissance.

B. Make a preliminary layout of the proposed storm drainage system, locating all inlets, manholes, mains, laterals, ditches, culverts, etc.

C. Outline the drainage area for each inlet in accordance with present and future development.

D. Indicate on each drainage area a code identification number, the area, the direction of surface runoff by small arrows, and the estimated coefficient of runoff for the area.

E. Show all existing underground utilities.

F. Establish design storm rainfall frequency.

G. Establish runoff time of concentration to storm water inlet(s).

H. Establish the typical cross section of each street.

I. Establish permissible flow spread of water on all streets within the drainage area.

J. Include Steps A through I with plans submitted to the City/County for review and approval. The drainage map submitted shall be suitable for permanent filing with the City/County and shall be a good quality copy. (See Chapter 4 Report Guidelines)

8.4.2 Storm Water Inlet System

Determining the size(s) and location(s) of inlets is largely a trail-and-error procedure. Using criteria outlined in Section I1 and Chapters 6 and 7 of this manual, the following steps will serve as a guide.

A. Beginning at the upstream end of the project drainage basin, outline a trial sub-basin and estimate the design storm runoff from it.

B. Compare the calculated runoff to allowable street capacity. If the calculated runoff is greater than the allowable street capacity, reduce the size of the trial sub-basin.

   Repeat this procedure until the calculated runoff equals the allowable street capacity. This is the first point at which a portion of the flow must be removed from the street.

C. Record the drainage area, runoff time of concentration, runoff coefficient and calculated runoff for the sub-basin. This information shall be recorded on the plans or in tabular form convenient for review.

D. If an inlet is to be used to remove water from the street, size the inlet(s) and record the inlet size, amount of intercepted flow, and amount of flow carried over (bypassing the inlet).

E. Continue the above procedure for other sub-basins until a complete system of inlets has been established.

F. After a complete system of inlets has been established, modification should be made to accommodate special situations such as point sources of large quantities of runoff, and variation of street alignments and grades.

G. Record information as in Steps C and D for all inlets.

H. After the inlets have been located and sized, the inlet pipes can be designed.

I. Inlet pipes are sized to carry the volume of water intercepted by the inlet. Inlet pipe capacities may be controlled by the gradient available, or by the entry condition into the pipe at the inlet. Inlet pipe sizes should be determined for both conditions and the larger size thus determined used.

8.4.3 Storm Drain System

After the computation of the quantity of storm water runoff entering each inlet, the storm drain system required to carry the runoff is designed.

1. Storm Drain Pipe

The ground-line profile is now used in conjunction with the previous runoff calculations. The elevation of the hydraulic gradient is established approximately one (1) foot below the ground surface (normally for pressure flow design). When this initial gradient is set and the design discharge is determined, a Manning's flow chart may be used to determine the pipe size and velocity.

It is possible that the initial gradient will have to be adjusted at this point since the intersection of the discharge and the slope on the chart will likely occur between standard pipe sizes.

2. Junctions, Inlets and Manholes

A. Determine the hydraulic gradient elevations at the upstream and downstream ends of the pipe section in question. The elevation of the hydraulic gradient of the upstream end of pipe is equal to the elevation of the downstream end of pipe (hydraulic gradient) plus the product of the length of pipe and the pipe gradient.

B. Determine the velocity of flow for incoming pipe (main line) at junction, inlet, or manhole at design point.

C. Determine the velocity of flow for outgoing pipe (main line) at junction, inlet, or manhole at design point.

D. Compute velocity head for outgoing velocity (main line) at junction, inlet, or manhole at design point.

E. Compute velocity head for incoming velocity (main line) at junction inlet, or manhole at design point.

F. Determine head loss coefficient from Figures 8-4 through 8-13.

G. Compute head loss at junction, inlet or manhole using the equation on Page 8-9, Item 7.

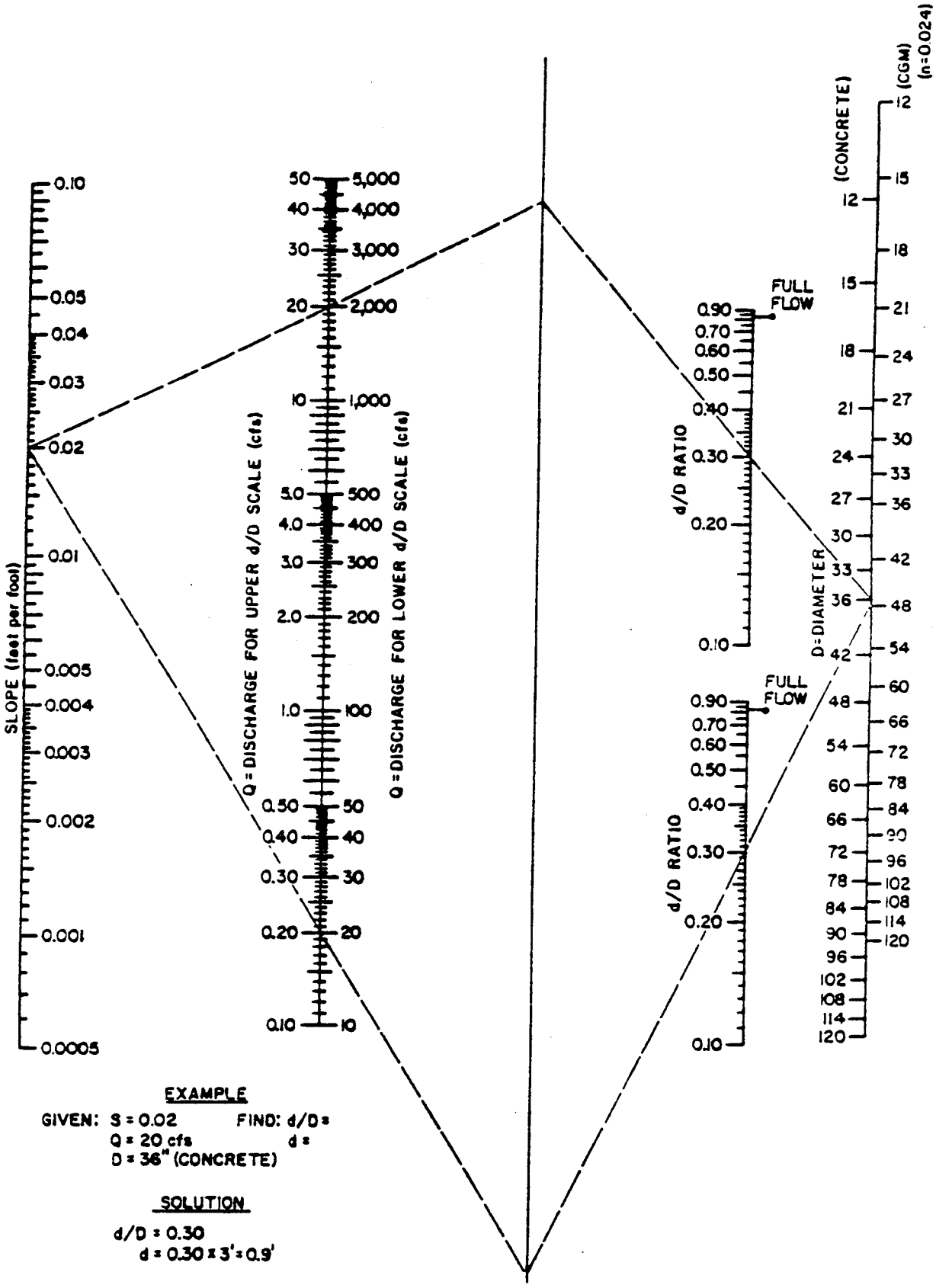
H. Compute hydraulic gradient at upstream end of junction as if junction were not there.

I. Add head loss to hydraulic gradient elevation determined to obtain hydraulic gradient elevation at upstream end of junction.

All information shall be recorded on the plans or in tabular form convenient for review.

3. Major Storm Design Adequacy Check

First, check the proposed street, storm drain and overflow system for the 10-year major storm event. Modify the proposed systems or provide additional flow capacity as required to accommodate the 100-year major storm water runoff according to the requirements stated in Section I and Chapters 6, 7 and 8.



EXAMPLE

GIVEN: S = 0.02 FIND: d/D =
Q = 20 cfs d =
D = 36" (CONCRETE)

SOLUTION

d/D = 0.30
d = 0.30 x 3' = 0.9'

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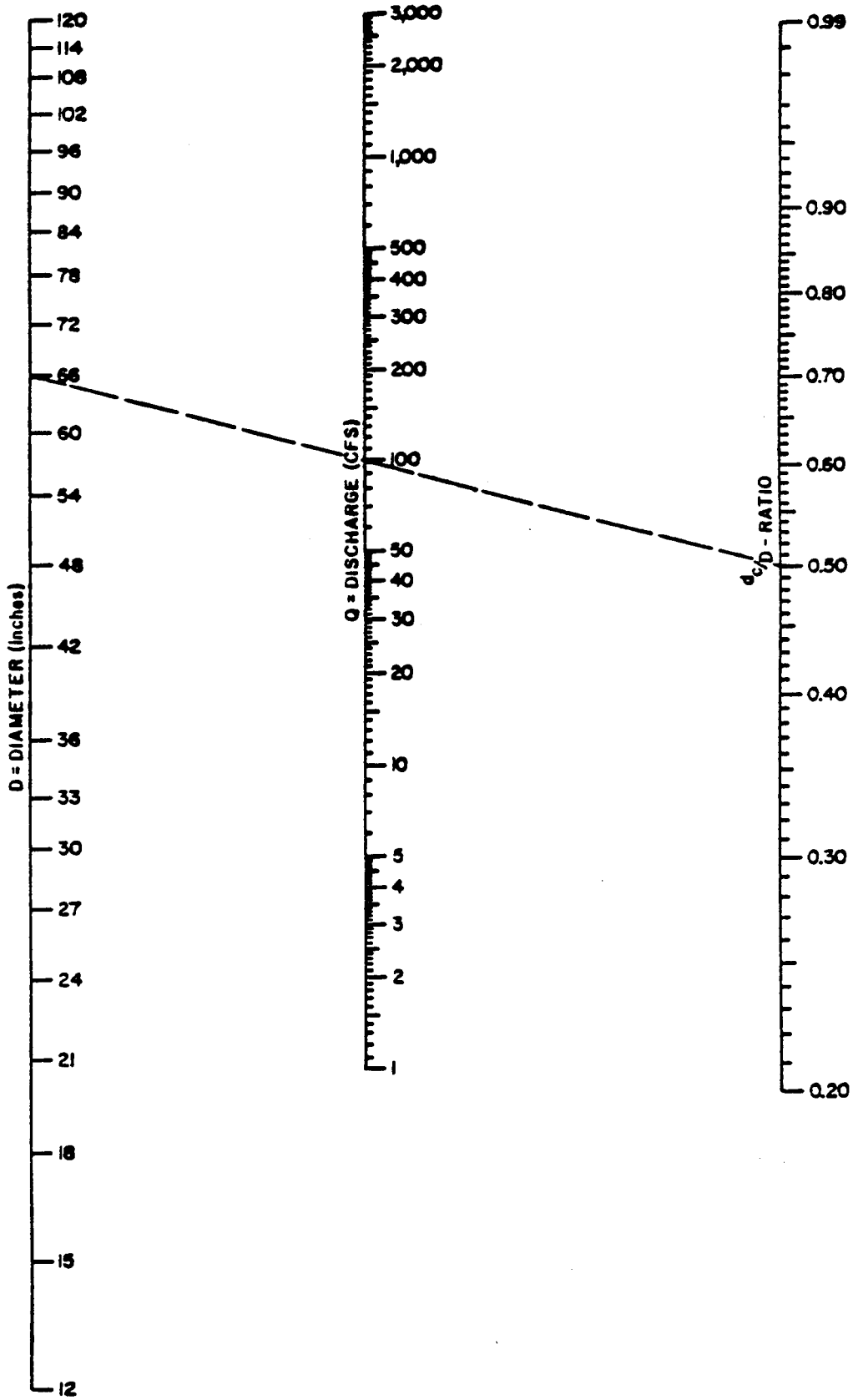
Uniform Flow for Pipe Culverts

Figure
8 - 1



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Critical Depth of Flow for Circular Conduits

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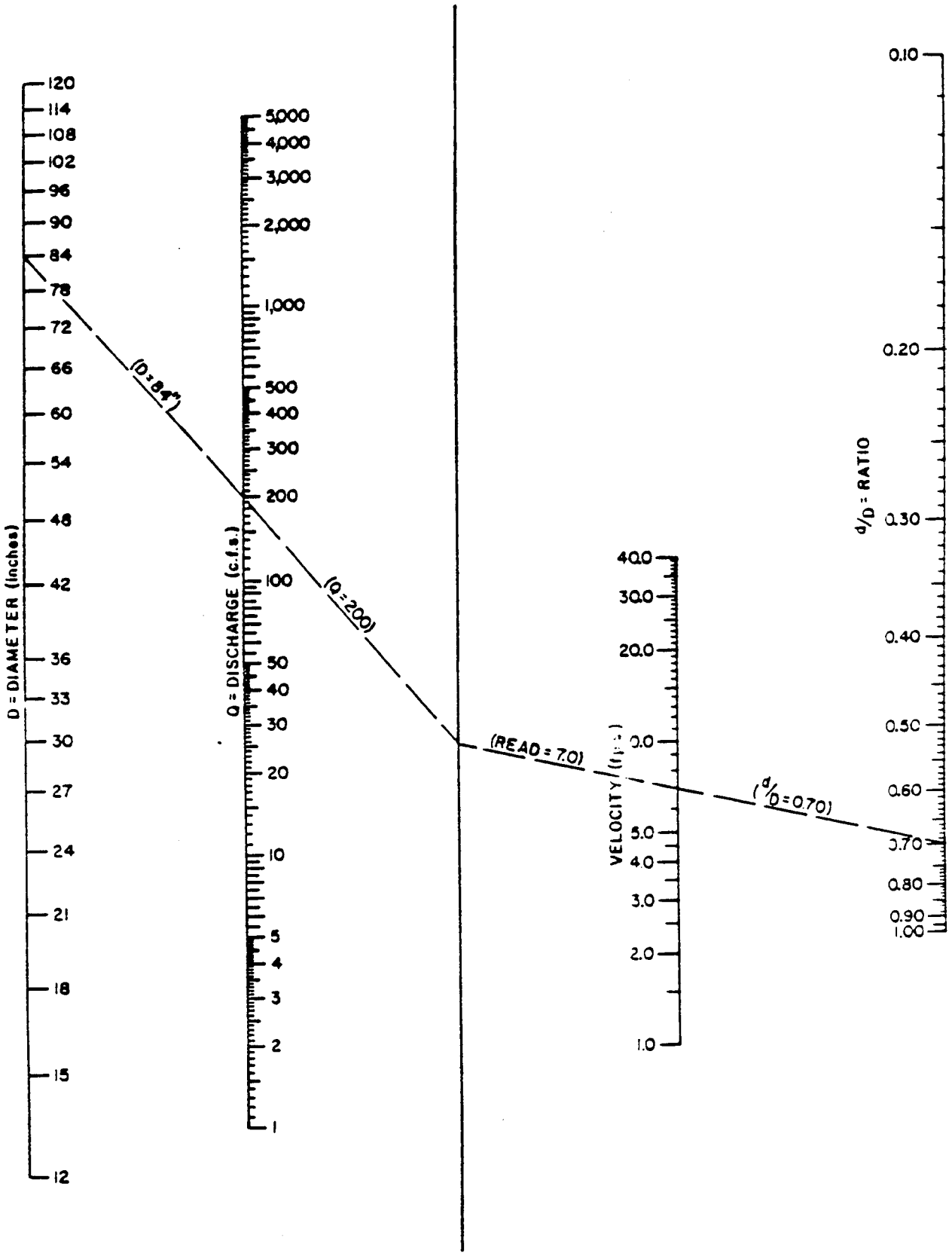
Figure
8 - 2



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Velocity in Pipe Conduits

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Figure
8 - 3

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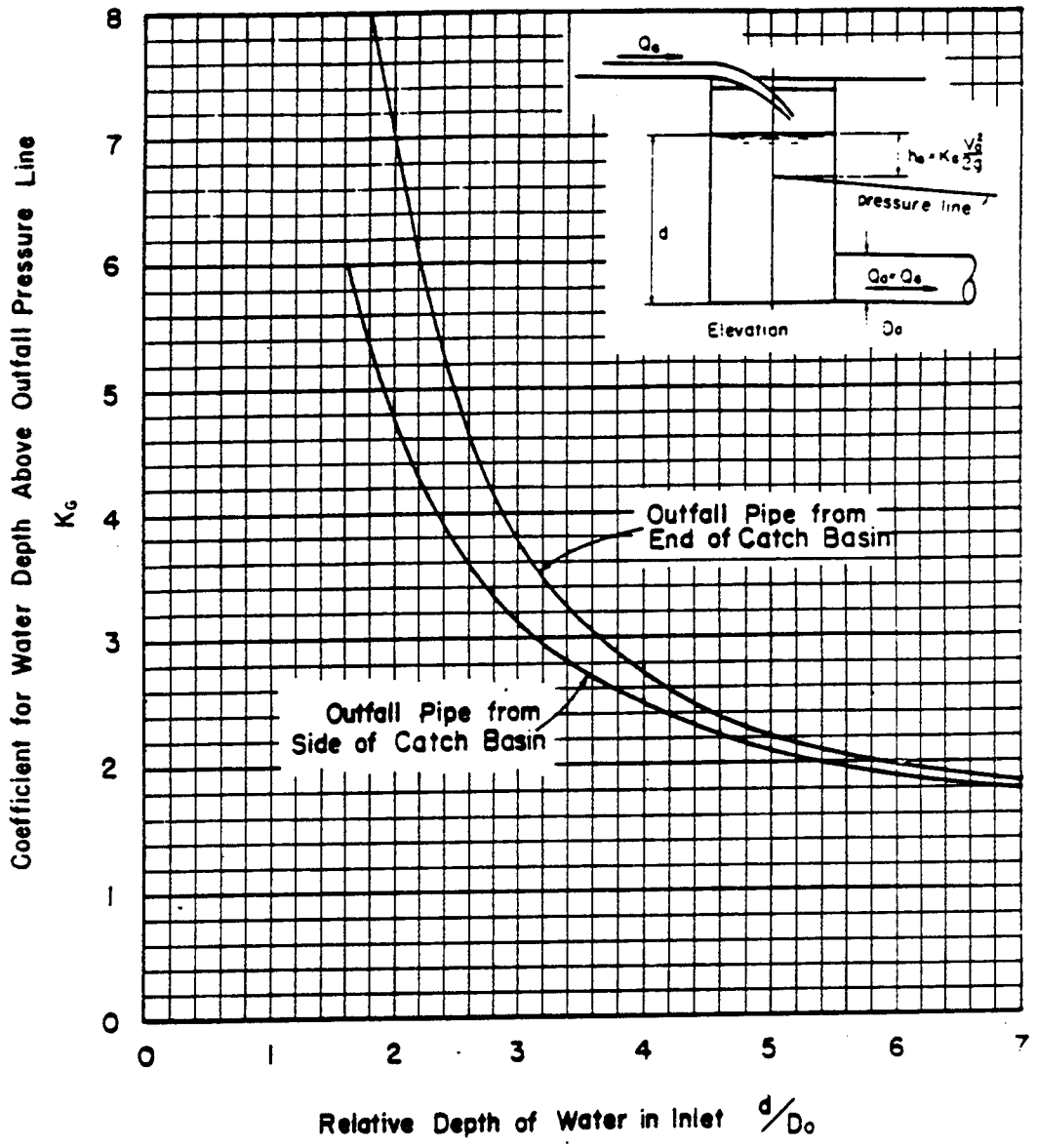
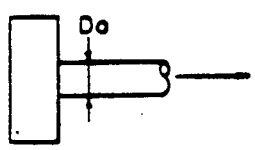
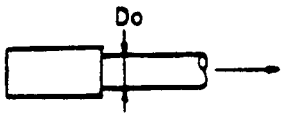




Box Side Flow



Box End Flow

CATCH BASIN WITH INLET FLOW ONLY

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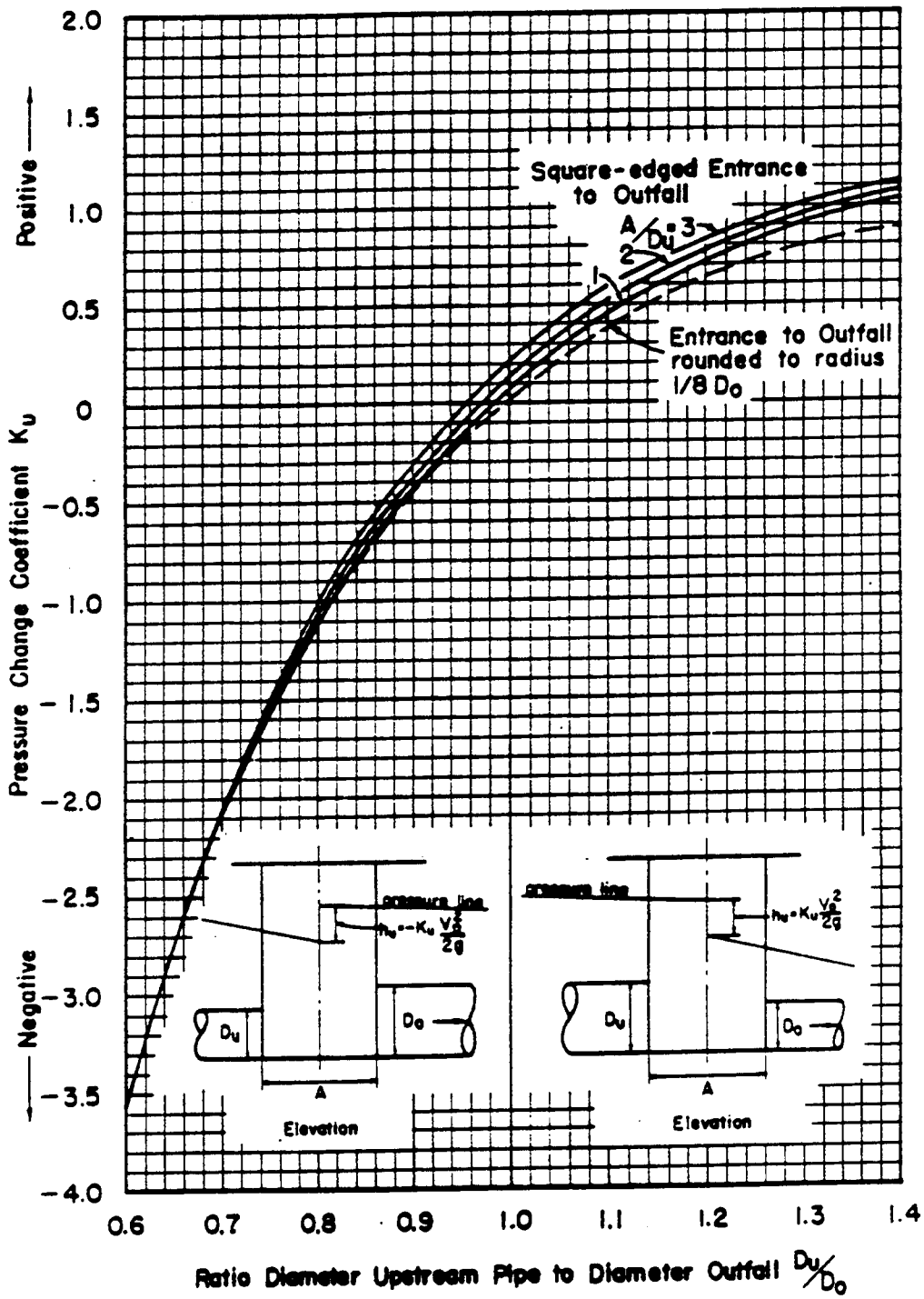
Figure
8 – 4



FLOW STRAIGHT THROUGH ANY MANHOLE

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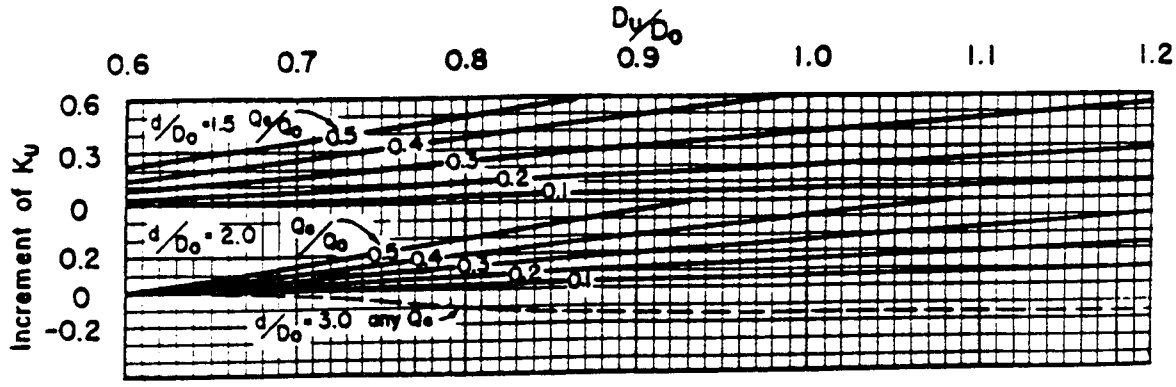
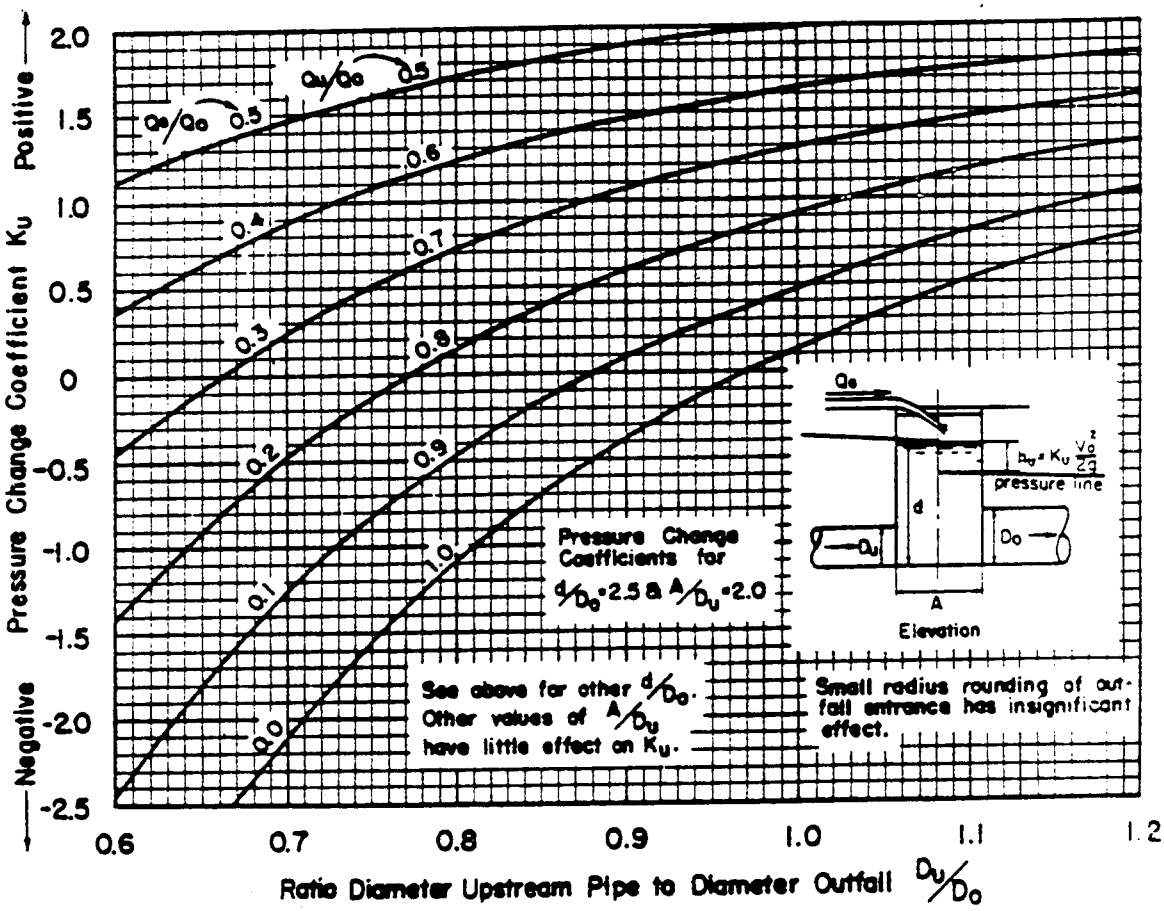
Figure
8 - 5



Supplementary Chart for Modification of K_u for Depth in Inlet other than 2.5 D_o



RECTANGULAR MANHOLE WITH THROUGH PIPELINE AND INLET FLOW

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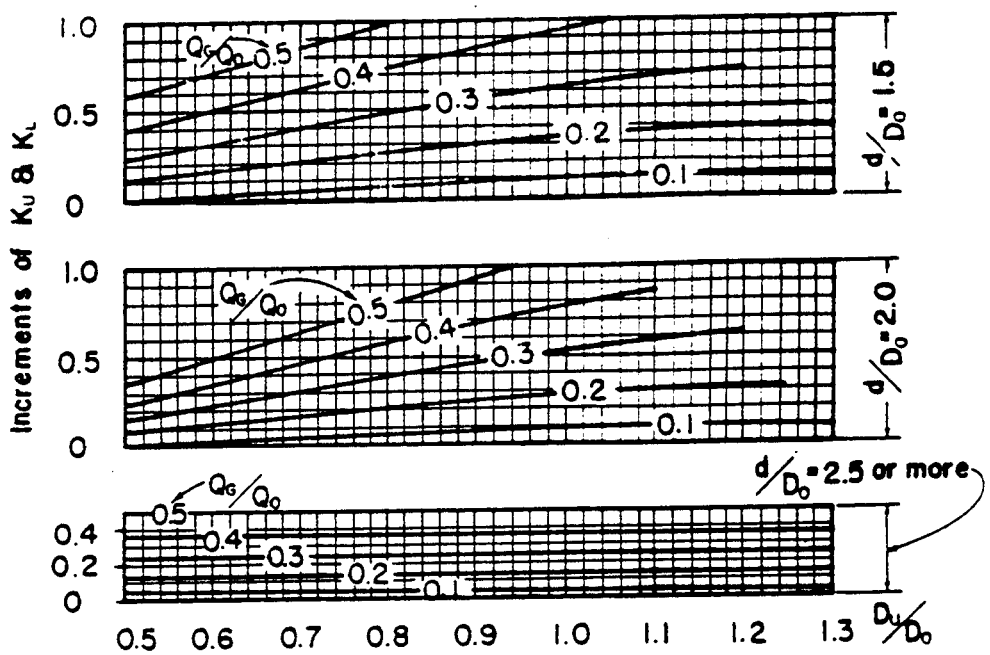
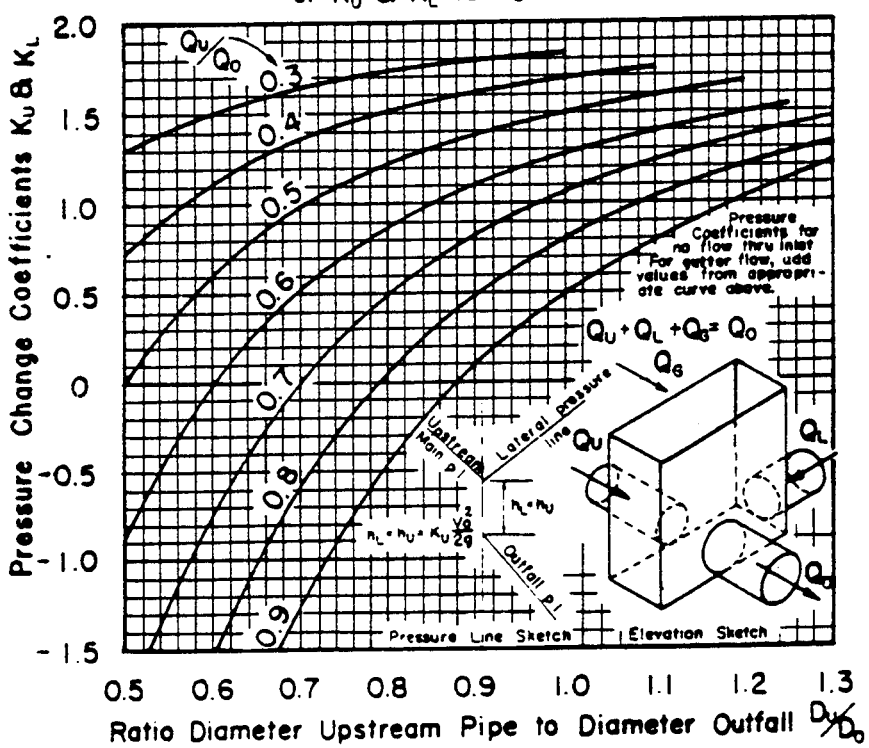
Figure
8 - 6



Supplementary Chart for Modification of K_U & K_L for Grate Flow



RECTANGULAR MANHOLE WITH IN-LINE UPSTREAM MAIN & 90° LATERAL PIPE
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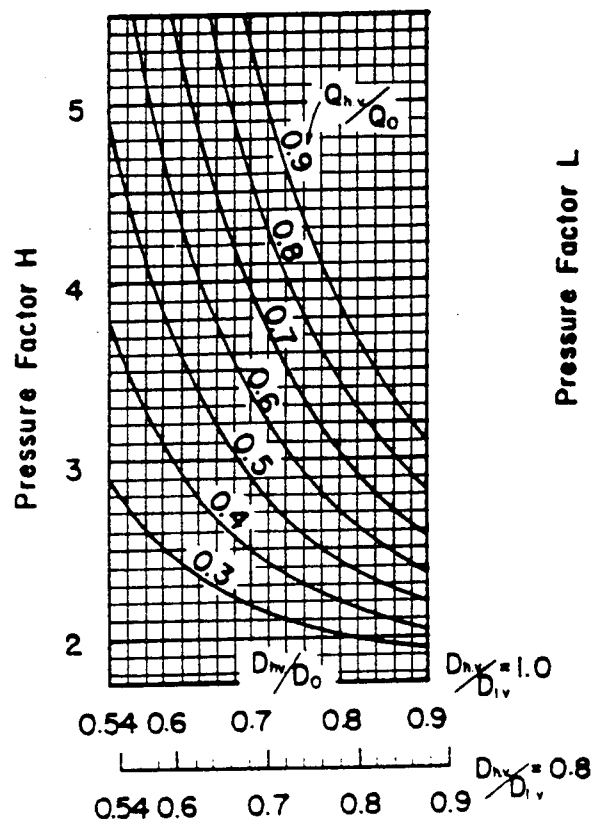
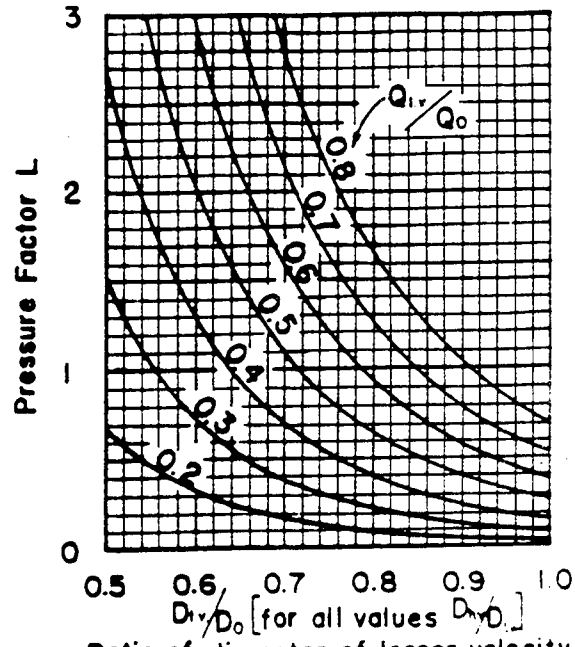
Figure
8 - 7



Ratio of diameter of higher velocity lateral to diameter of outfall



Ratio of diameter of lesser velocity lateral to diameter of outfall

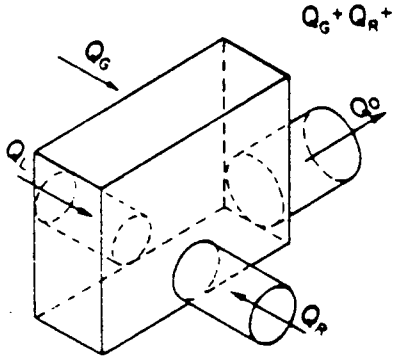
D_{hv} = diameter of lateral with higher-velocity flow.

Q_{hv} = rate of flow in lateral with higher-velocity flow.

D_{lv} = diameter of lateral with lower-velocity flow.

Q_{lv} = 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

$$h_L = K_L \frac{V_o^2}{2g} \qquad h_R = K_R \frac{V_o^2}{2g}$$

RECTANGULAR MANHOLE WITH IN-LINE OPPOSED LATERAL PIPES EACH AT 90° TO OUTFALL (WITH OR WITHOUT INLET FLOW)

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Figure
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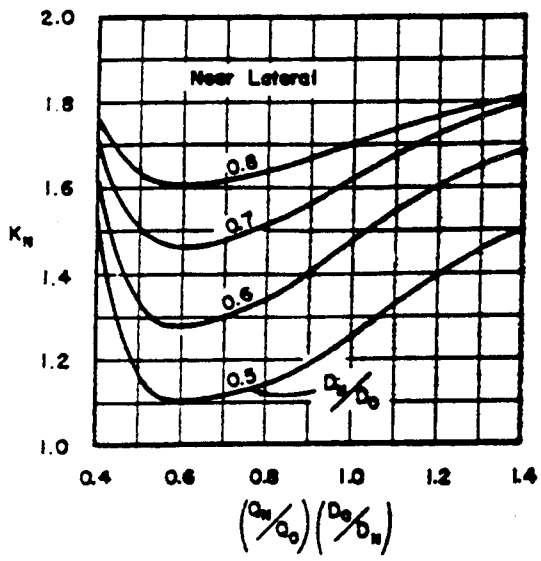
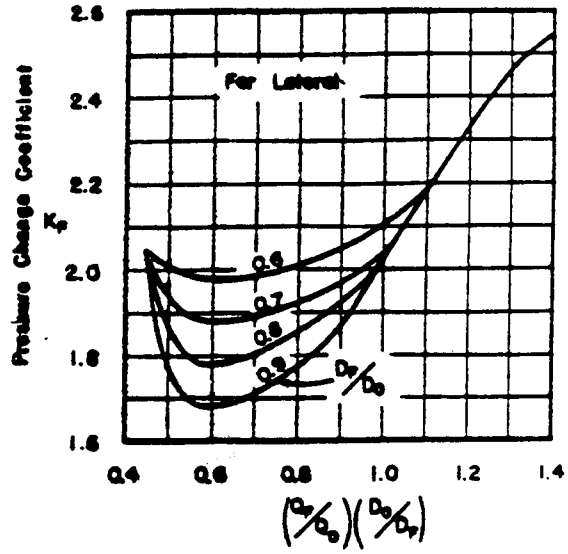
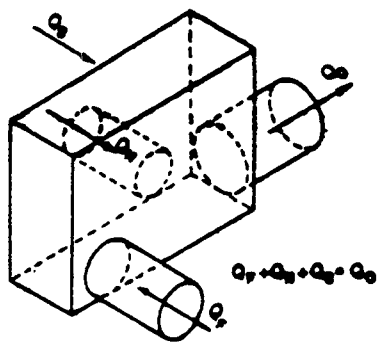




$h_N = K_N \frac{V_O^2}{2g}$

$h_F = K_F \frac{V_O^2}{2g}$



Elevation Sketch

RECTANGULAR MANHOLE WITH OFFSET OPPOSED LATERAL PIPES
EACH AT 90° TO OUTFALL
(WITH OR WITHOUT INLET FLOW)

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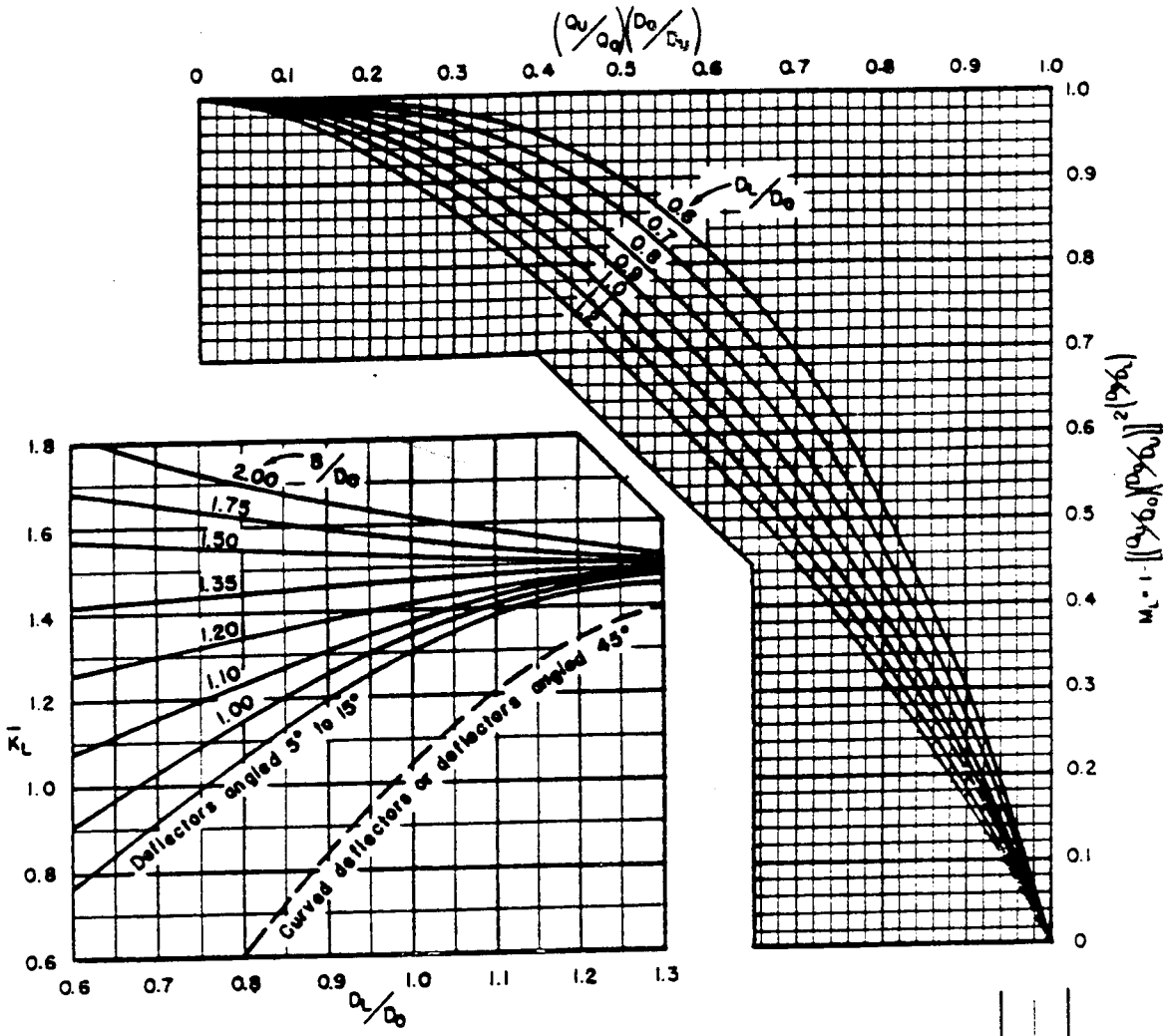
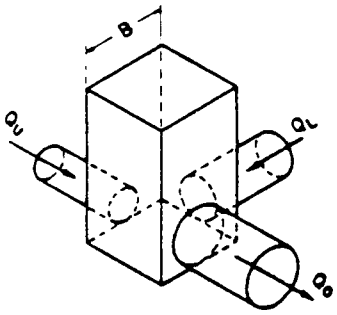
Figure
8 - 9





Elevation Sketch

To find K_L for the lateral pipe, first read $\bar{K}_L$ from the lower graph. Next determine M_L Then

$$K_L = \bar{K}_L \times M_L$$

Dashed curve for curved or 45° angle deflectors applies only to manholes without upstream in-line pipe.

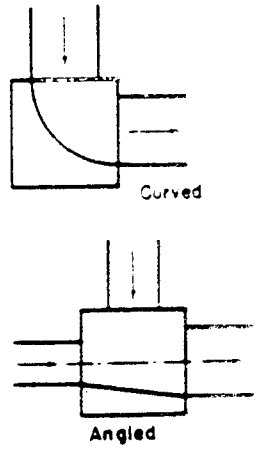
Use this chart for round manholes also.

For rounded entrance to outfall pipe, reduce chart values of $\bar{K}_L$ by 0.2 for combining flow.

For $Q_u/Q_o \times D_o/D_u > 1$ use Figure 8-12

For $D_L/D_o < 0.6$ use Figure 8-12

$$h_L = K_L \frac{V_o^2}{2g}$$



Plan of Deflectors

MANHOLE AT 90° DEFLECTION OR ON THROUGH PIPELINE AT JUNCTION OF 90° LATERAL PIPE. (LATERAL COEFFICIENT).

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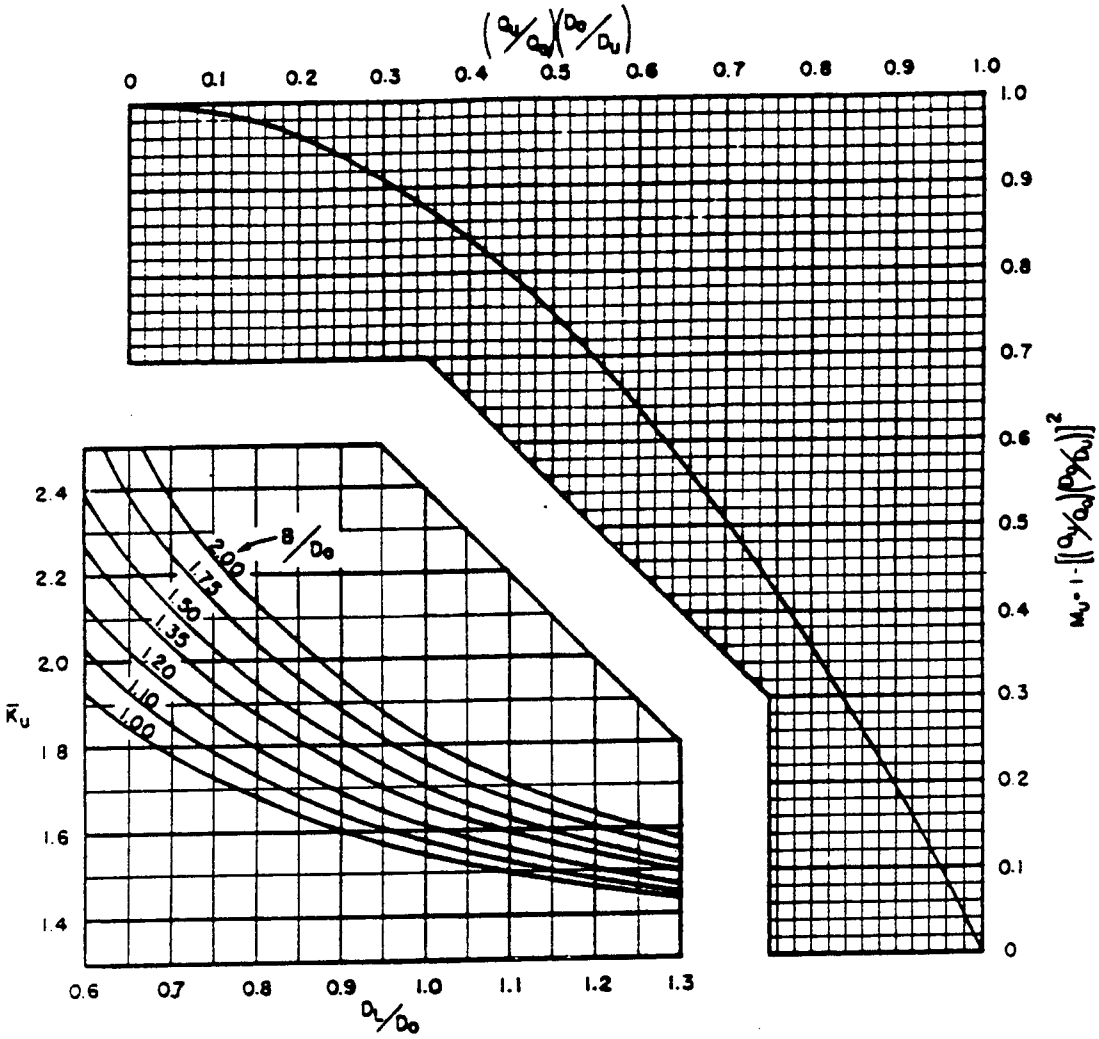
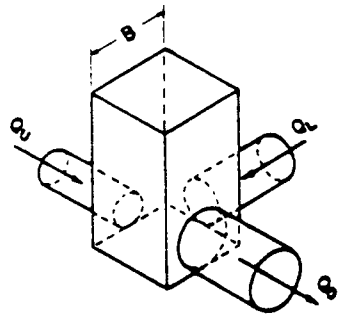
Figure
8 - 10





Elevation Sketch

To find K_U for the upstream main, first read $\bar{K}_U$ from the lower graph. Next determine M_U. Then

$$K_U = \bar{K}_U \times M_U$$

For manholes with deflectors at 0° to 15°, read $\bar{K}_U$ 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 $\bar{K}_U$ by 0.2 for combining flow.

For deflectors refer to sketches on Figure 8-10

For $Q_U/Q_o \times D_o/D_U > 1$ use Figure 8-12

For $D_L/D_o < 0.6$ use Figure 8-12

$$h_U = K_U \frac{V_o^2}{2g}$$

MANHOLE ON THROUGH PIPELINE AT JUNCTION OF A 90° LATERAL PIPE (IN-LINE PIPE COEFFICIENT)

1-15-69
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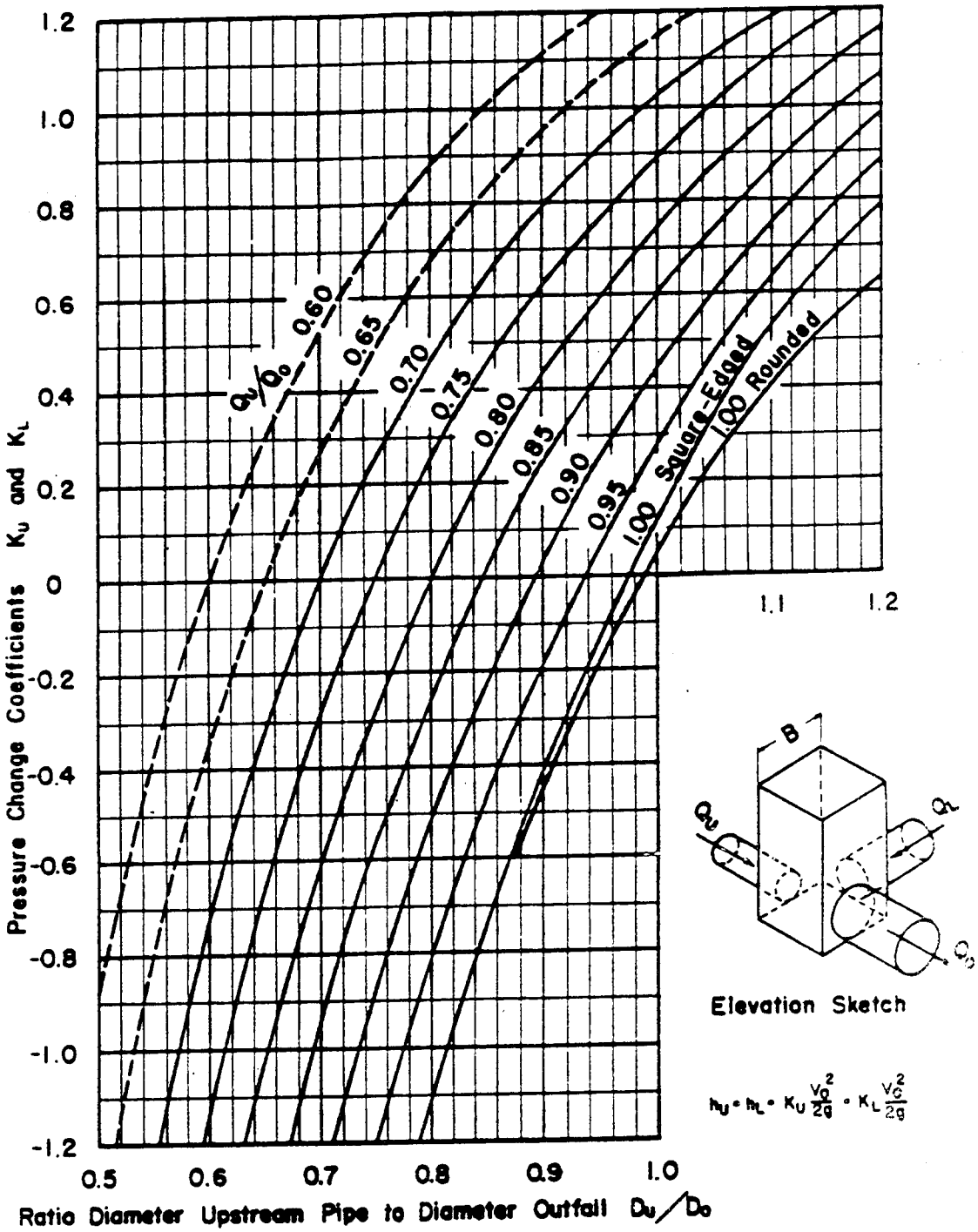
Figure
8 - 11



MANHOLE ON THROUGH PIPELINE AT JUNCTION OF A 90° LATERAL PIPE (FOR CONDITIONS OUTSIDE RANGE OF FIGURES 8-10 & 8-11

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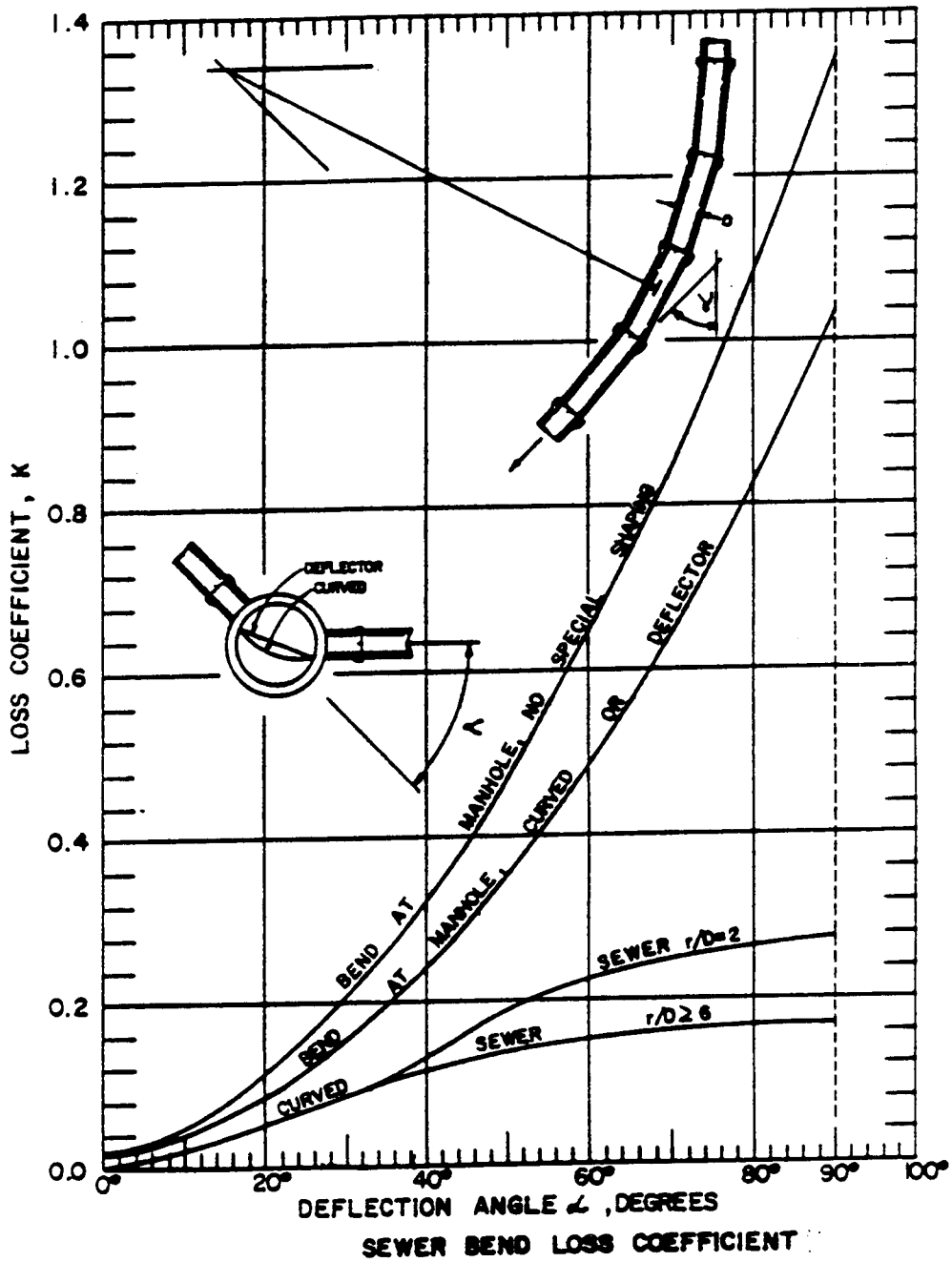
Figure
8 - 12



SEWER BEND LOSS COEFFICIENT

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Figure
8 - 13

Q = ACi

A - acres
i - inches/hr.
Q - cfs
t - min
V - ft/sec

COMPUTATION FORM

DRAINAGE & STORM SEWER DESIGN
RATIONAL METHOD

Project _____
Comp. by _____ date _____
Ck by _____ date _____

LOCATION			A		C	TIME OF CONCENTRATION		i	Q	DESIGN					PROFILE				
STREET	FROM	TO	INCRE-MENT	TOTAL		TO INLET	PIPE OR CHANNEL			SIZE	SLOPE %	n	CAP. FULL	V	LENGTH FT.	FALL FT.	OTHER LOSSES	UPPER INV. EL.	LOWER INV. EL.

TABLE 8-1
UNIFORM FLOW IN CIRCULAR SECTIONS FLOWING PARTLY FULL

d = depth of flow
D = diameter of pipe
A = area of flow
R = hydraulic radius
Q = discharge in cubic feet per second by Manning's formula
n = Manning's coefficient
S = slope of the channel bottom and of the water surface

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$
0.01	0.0013	0.0066	0.00007	15.04	0.51	0.4027	0.2531	0.239	1.442
0.02	0.0037	0.0132	0.00031	10.57	0.52	0.4127	0.2562	0.247	1.415
0.03	0.0069	0.0197	0.00074	8.56	0.53	0.4227	0.2592	0.255	1.388
0.04	0.0105	0.0262	0.00138	7.38	0.54	0.4327	0.2621	0.263	1.362
0.05	0.0147	0.0325	0.00222	6.55	0.55	0.4426	0.2649	0.271	1.336
0.06	0.0192	0.0389	0.00328	5.95	0.56	0.4526	0.2676	0.279	1.311
0.07	0.0242	0.0451	0.00455	5.47	0.57	0.4625	0.2703	0.287	1.286
0.08	0.0294	0.0513	0.00604	5.09	0.58	0.4724	0.2728	0.295	1.262
0.09	0.0350	0.0575	0.00775	4.76	0.59	0.4822	0.2753	0.303	1.238
0.10	0.0409	0.0635	0.00967	4.49	0.60	0.4920	0.2776	0.311	1.215
0.11	0.0470	0.0695	0.01181	4.25	0.61	0.5018	0.2799	0.319	1.192
0.12	0.0534	0.0755	0.01417	4.04	0.62	0.5115	0.2821	0.327	1.170
0.13	0.0600	0.0813	0.01674	3.86	0.63	0.5212	0.2842	0.335	1.148
0.14	0.0668	0.0871	0.01952	3.69	0.64	0.5308	0.2862	0.343	1.126
0.15	0.0739	0.0929	0.0225	3.54	0.65	0.5404	0.2882	0.350	1.105
0.16	0.0811	0.0985	0.0257	3.41	0.66	0.5499	0.2900	0.358	1.084
0.17	0.0885	0.1042	0.0291	3.28	0.67	0.5594	0.2917	0.366	1.064
0.18	0.0961	0.1097	0.0327	3.17	0.68	0.5687	0.2933	0.373	1.044
0.19	0.1039	0.1152	0.0365	3.06	0.69	0.5780	0.2948	0.380	1.024
0.20	0.1118	0.1206	0.0406	2.96	0.70	0.5872	0.2962	0.388	1.004
0.21	0.1199	0.1259	0.0448	2.87	0.71	0.5964	0.2975	0.395	0.985
0.22	0.1281	0.1312	0.0492	2.79	0.72	0.6054	0.2987	0.402	0.965
0.23	0.1365	0.1364	0.0537	2.71	0.73	0.6143	0.2998	0.409	0.947
0.24	0.1449	0.1416	0.0585	2.63	0.74	0.6231	0.3008	0.416	0.928
0.25	0.1535	0.1466	0.0634	2.56	0.75	0.6319	0.3017	0.422	0.910
0.26	0.1623	0.1516	0.0686	2.49	0.76	0.6405	0.3024	0.429	0.891
0.27	0.1711	0.1566	0.0739	2.42	0.77	0.6489	0.3031	0.435	0.873
0.28	0.1800	0.1614	0.0793	2.36	0.78	0.6573	0.3036	0.441	0.856
0.29	0.1890	0.1662	0.0849	2.30	0.79	0.6655	0.3039	0.447	0.838
0.30	0.1982	0.1709	0.0907	2.25	0.80	0.6736	0.3042	0.453	0.821
0.31	0.2074	0.1756	0.0966	2.20	0.81	0.6815	0.3043	0.458	0.804
0.32	0.2167	0.1802	0.1027	2.14	0.82	0.6893	0.3043	0.463	0.787
0.33	0.2260	0.1847	0.1089	2.09	0.83	0.6969	0.3041	0.468	0.770
0.34	0.2355	0.1891	0.1153	2.05	0.84	0.7043	0.3038	0.473	0.753
0.35	0.2450	0.1935	0.1218	2.00	0.85	0.7115	0.3033	0.477	0.736
0.36	0.2546	0.1978	0.1284	1.958	0.86	0.7186	0.3026	0.481	0.720
0.37	0.2642	0.2020	0.1351	1.915	0.87	0.7254	0.3018	0.485	0.703
0.38	0.2739	0.2062	0.1420	1.875	0.88	0.7320	0.3007	0.488	0.687
0.39	0.2836	0.2102	0.1490	1.835	0.89	0.7384	0.2995	0.491	0.670
0.40	0.2934	0.2142	0.1561	1.797	0.90	0.7445	0.2980	0.494	0.654
0.41	0.3032	0.2182	0.1633	1.760	0.91	0.7504	0.2963	0.496	0.637
0.42	0.3130	0.2220	0.1705	1.724	0.92	0.7560	0.2944	0.497	0.621
0.43	0.3229	0.2258	0.1779	1.689	0.93	0.7612	0.2921	0.498	0.604
0.44	0.3328	0.2295	0.1854	1.655	0.94	0.7662	0.2895	0.498	0.588
0.45	0.3428	0.2331	0.1929	1.622	0.95	0.7707	0.2865	0.498	0.571
0.46	0.3527	0.2366	0.201	1.590	0.96	0.7749	0.2829	0.496	0.553
0.47	0.3627	0.2401	0.208	1.559	0.97	0.7785	0.2787	0.494	0.535
0.48	0.3727	0.2435	0.216	1.530	0.98	0.7817	0.2735	0.489	0.517
0.49	0.3827	0.2468	0.224	1.500	0.99	0.7841	0.2666	0.483	0.496
0.50	0.3927	0.2500	0.232	1.471	1.00	0.7854	0.2500	0.463	0.463

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CHAPTER 9 CULVERT DESIGN

9.1 General Statement

The function of a drainage culvert is to pass the design storm water runoff without causing excessive backwater and without creating excessive downstream flow velocities.

9.2 Culvert Types

Culverts shall be selected based on hydraulic principles, economy of size and shape, and with a resulting headwater depth which will not cause damage to adjacent property for the 100-year storm. It is essential to the proper design of a culvert that the conditions under which the culvert will operate are known. Five types of operating conditions are discussed below.

Type I Flowing Part Full with Outlet Control and Tailwater Depth Below Critical Depth.

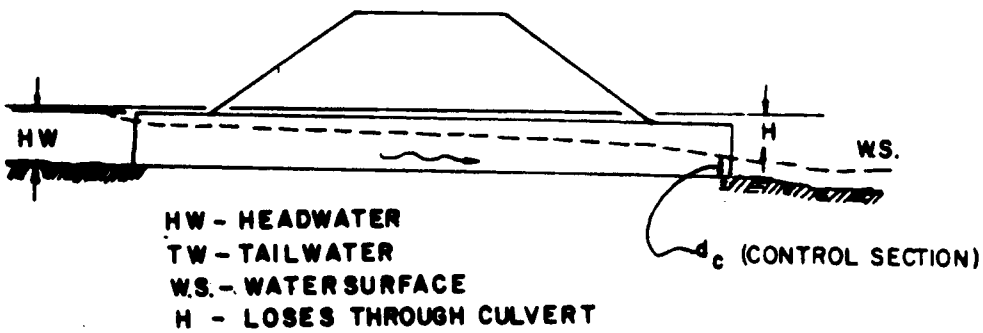

Figure 9-1

Conditions

The entrance is unsubmerged (HW<1.2D), the slope at design discharge is subcritical ($S_o < S_c$), and the tailwater is equal to or below critical depth ($TW \leq d_c$).

The above condition is a common occurrence where the natural channels are on flat grades and have wide, flat floodplains. The control is critical depth at the outlet.

In culvert design, it is generally considered that the headwater pool maintains a constant level during the design storm. If this level does not submerge the culvert inlet, the culvert flows part full.

If critical flow occurs at the outlet, the culvert is said to have "Outlet Control". A culvert flowing part full with outlet control will require a depth of flow in the barrel of the culvert greater than critical depth while passing through critical depth at the outlet.

The capacity of a culvert flowing part full with outlet control and tailwater depth below critical depth is governed by the following equation when the approach velocity is considered zero.

$$HW = d_c + \frac{V_c^2}{2g} + h_e + h_f - S_o L \qquad (9\text{-}1)$$

where:

d_c = critical depth of flow, in feet (Figure 9-25 or 9-26);

D = diameter of pipe or height of box, in feet;

V_c = critical velocity, in fps, occurring at critical depth;

h_e = entrance head loss, in feet, where;

$$h_e = k_e \frac{V_c^2}{2g} \qquad (9\text{-}2)$$

k_e = entrance loss coefficient (Table 9-1).

h_f = friction head loss, in feet = $S_f L$;

S_f = friction slope or slope that will produce uniform flow. For Type I operation, the friction slope is based upon $1.1 * d_c$;

S_o = slope of culvert, in feet per foot;

L = length of culvert, in feet; and

HW = headwater depth above the invert of the upstream end of the culvert in feet. Headwater must be equal to or less than 1.2D, or entrance is submerged and Type IV operation will result.

Type II Flowing Part Full with Outlet Control and Tailwater Depth Above Critical Depth

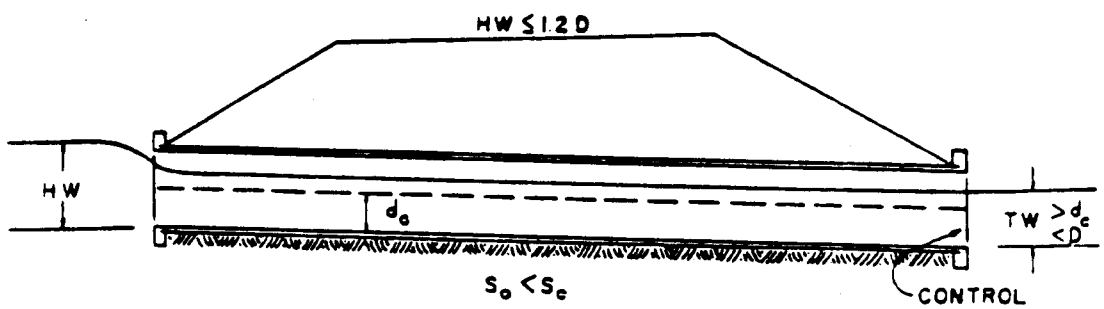

Figure 9-2

Conditions

The entrance is unsubmerged (HW < 1.2D), the slope at design discharge is subcritical ($S_o < S_c$), and the tailwater is above critical depth (TW > d_c).

The above condition is common occurrence where the channel is deep, narrow and well defined.

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If the headwater pool elevation does not submerge the culvert inlet, the slope at design discharge is subcritical, the tailwater depth is above critical depth and the control occurs at the outlet. The capacity of the culvert is governed by the following equation when the approach velocity is considered zero.

$$HW = TW + \frac{V_{TW}^2}{2g} + h_e + h_f - S_o L \qquad (9\text{-}3)$$

where:

TW = tailwater depth above the invert of the downstream end of the culvert, in feet;

V_{TW} = culvert discharge velocity, in fps, at tailwater depth;

h_e = entrance head loss, in feet, where;

$$h_e = k_e \frac{V_{TW}^2}{2g} \qquad (9\text{-}4)$$

k_e = entrance loss coefficient (Table 9-1)

h_f = friction head loss, in feet = $S_f L$

S_f = friction slope or slope that will produce uniform flow, in feet per foot. For Type II operation, the friction slope is based upon TW depth;

S_o = slope of culvert, in feet per foot;

L = length of culvert, in feet; and

HW = headwater depth above the invert of the upstream end of the culvert in feet. Headwater depth must be equal to or less than 1.2D, or entrance is submerged and Type IV operation will result.

Type III Flowing Part Full with Inlet Control.

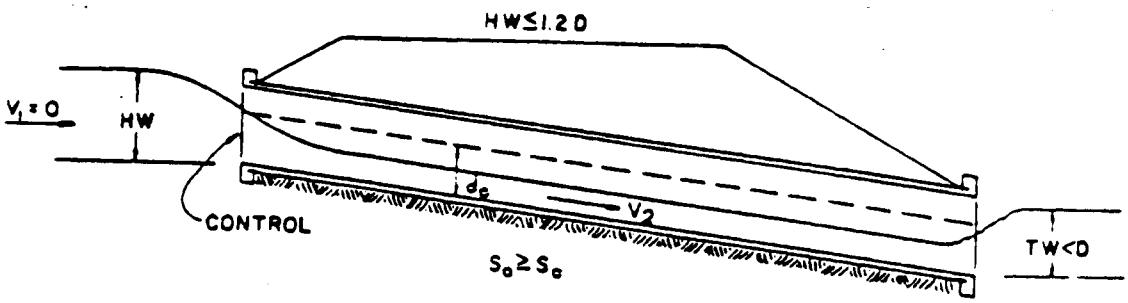

Figure 9-3

Conditions

The entrance is unsubmerged (HW < 1.2D) and the slope at design discharge is equal to or greater than critical slope (super-critical) ($S_o \geq S_c$).

This condition is a common occurrence for culverts in rolling or mountainous country where the flow does not submerge the entrance. The control is critical depth at the entrance.

If critical flow occurs near the inlet, the culvert is operating under "Inlet Control". The maximum discharge through a culvert flowing part full occurs when flow is at critical depth for a given energy head. To assure that flow passes through critical depth near the inlet, the culvert must be laid on a slope equal to or greater than critical slope for the design discharge. Placing culverts which are to flow part full on slopes greater than critical slope will increase the outlet velocities but will not increase the discharge. The discharge is limited by the section near the inlet at which critical flow occurs.

The capacity of a culvert flowing part full with control at the inlet is governed by the following equation when the approach velocity is considered zero.

$$HW = d_c + \frac{V_2^2}{2g} + ke \frac{V_2^2}{2g} \qquad (9\text{-}5)$$

where:

d_c = critical depth of flow, in feet (Figure 9-26 or 9-27);

V_2 = critical velocity at entrance of culvert, in fps;

The velocity of flow varies from critical velocity at entrance to uniform velocity at the outlet provided the culvert is sufficiently long. Therefore, the outlet velocity is the discharge divided by the area of flow in the culvert;

k_e = entrance loss coefficient (Table 9-2); and

HW = headwater depth above the invert of the upstream end of the culvert in feet. Headwater depth must be equal to or less than 1.2D, or entrance is submerged and Type IV operation will result.

Type IVA Flowing Full with Submerged Outlet

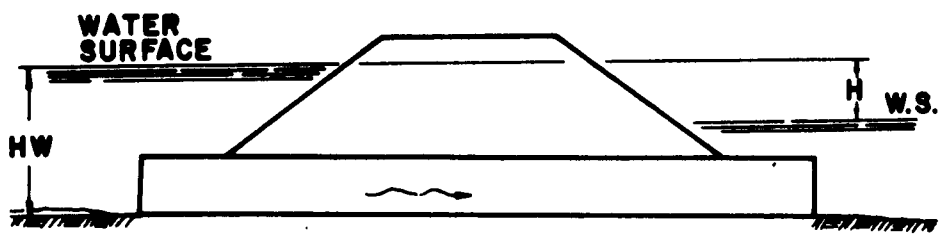

Figure 9-4

Conditions

(Submerged Outlet)

The entrance is submerged (HW $\geq$ 1.2D). The tailwater completely submerges the outlet.

Most culverts flow with free outlet, but depending on topography, a tailwater pool of a depth sufficient to submerge the outlet may form at some installations. For an outlet to be submerged, the depth at the outlet must be equal to or greater than the diameter of pipe or height of box. The capacity of a culvert flowing full with a submerged outlet is governed by the following equation when the approach velocity is considered zero. Outlet velocity is based on full-pipe flow at the outlet.

$$HW = H + TW - S_oL \qquad (9\text{-}6)$$

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where:

H = head for culvert flowing full, in feet;

TW = tailwater depth, in feet;

S_o = slope of culvert, in feet per foot;

L = length of culvert, in feet; and

HW = headwater depth, in feet, above the invert of the upstream end of the culvert. Headwater depth must be greater than 1.2D for entrance to be submerged.

Type IVB Flowing Full with Partially Submerged Outlet.

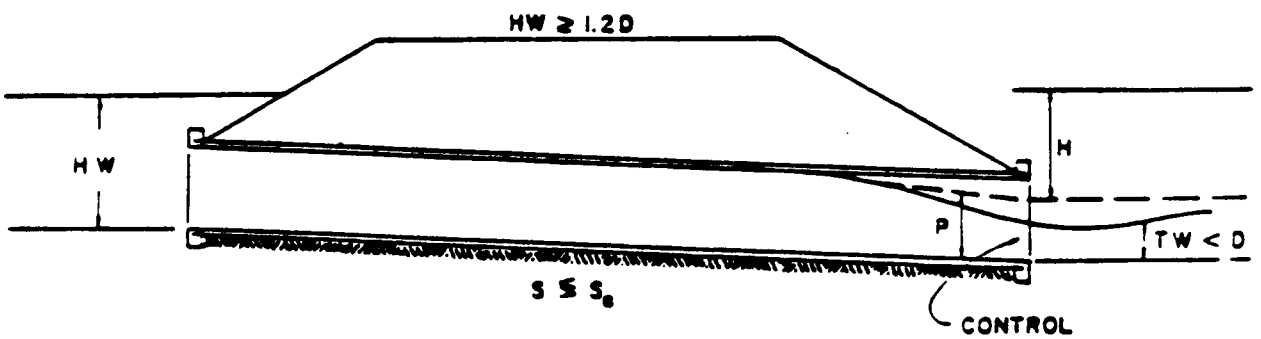

Figure 9-5

Conditions

(Partially Submerged Outlet)

The entrance is submerged (HW $\geq$ 1.2D). The tailwater depth is less than D (TW < D).

The capacity of a culvert flowing with a partially submerged outlet is governed by the following equation when the approach velocity is considered zero. Outlet velocity is based on critical depth if TW depth is less than critical depth. If TW depth is greater than critical depth, outlet velocity is based on TW depth.

$$HW = H + P - S_o L \qquad (9\text{-}7)$$

where:

H = head for culverts flowing full, in feet;

P = pressure line height = $\frac{d_c + D}{2}$, in feet

d_c = critical depth, in feet;

D = diameter or height of structure, in feet;

S_o = slope of culvert, in feet per foot;

L = length of culvert, in feet; and

HW = headwater depth, in feet, above the invert of the upstream end of the culvert. Headwater depth must be greater than 1.2D for entrance to be submerged.

9.3 End Treatments

The normal functions of properly designed headwalls and endwalls are to anchor the culvert to prevent movement, to control erosion and scour resulting from excessive flow velocities and turbulence, and to prevent adjacent soil from sloughing into the waterway opening. All headwalls shall be constructed of reinforced concrete and may be either straight parallel headwalls, flared headwalls, or warped headwalls with or without aprons as may be required by local site conditions.

Manufactured flared end sections may be used as approved by the City/County.

9.3.1 Entrance Conditions

It is important to recognize that the operating characteristics of a culvert may be completely changed by the shape or condition at the inlet or entrance. Design of culverts must involve consideration of energy losses that may occur at the entrance. The entrance head losses may be determined by the following equation.

$$h_e = k_e \frac{V^2}{2g} \qquad (9\text{-}8)$$

where:

k_e = entrance loss coefficient (Table 9-1)

V = velocity of flow in culvert, in fps; and

h_e = entrance head loss, in feet.

9.3.2 Headwall/Endwall Treatment

In general, the following guidelines should be used in the selection of the type of headwall or endwalls;

Parallel Headwall and Endwall

1. Approach velocities are low (below 6 fps).
2. Backwater pools may be permitted.
3. Approach channel is undefined.
4. Sufficient right-of-way or easement is available.
5. Downstream channel protection is not required.

Flared Headwall and Endwall

1. Channel is well defined.
2. Approach velocities are between 6 and 10 fps.
3. Medium amounts of debris exist.

Warped (twisted) Headwall and Endwall

1. Channel is well defined and concrete lined.
2. Approach velocities are between 8 and 20 fps.
3. Medium amounts of debris exist.

These headwalls are effective with drop-down aprons to accelerate flow through culvert, and they are effective endwalls for transitioning flow from closed conduit flow to open-channel flow.

9.3.3 Improved Inlets

Several types of improved inlets have been developed. The use of these inlets may provide substantial savings by a reduction in the barrel size of the proposed structure. The use of these inlets is optional and should be based on an economic analysis by the designer. For box culverts, reinforced concrete structures, and structures using headwalls the use of beveled inlets or tapered inlets is strongly recommended. For more information, the designer is referred to Hydraulic Design of Improvements for Culverts, Hydraulic Engineering Circular No. 13, August 1973, by Federal Highway Administration (FHA) and Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, September 1985 by FHA.

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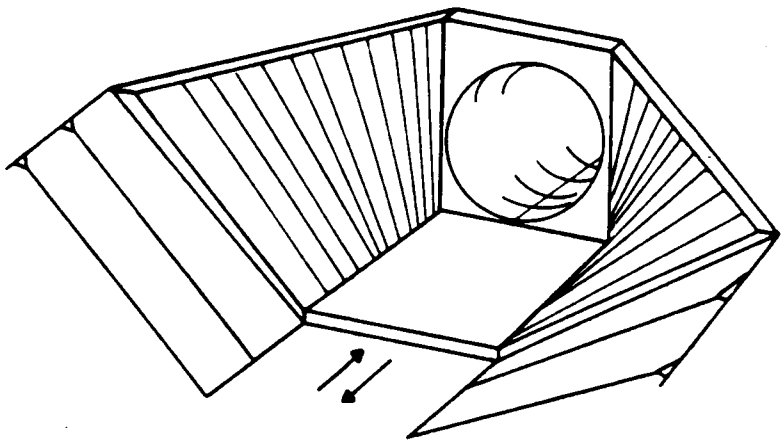


Figure 9-6

Warped (twisted) Headwall

9.4 Culvert Design with Standard Inlets

The information and publications necessary to design culverts according to the procedure presented in this section can be found in Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, September 1985. This publication is by the FHA. A few of the charts and nomographs, from this publication, covering the more common requirements, are located in the Design Charts at the end of this chapter. For special cases and larger structure sizes, the FHA publication should be used. In addition, an IBM PC compatible computer program is available to perform these calculations from the FHA.

9.4.1 Culvert Sizing

Each chart, Figures 9-13 through 9-20, contains a series of curves which show the discharge capacity per barrel in cfs for each of several sizes of similar type culverts for various headwater depths in feet above the culvert invert at the inlet. The invert of the culvert is defined as the low point of its cross-section.

Each culvert size is described by two lines, one solid and one dashed. The numbers associated with each line are the ratio of the length, L, in feet, to the slope, 100 S_o, in percent. The dashed lines represent the maximum $L/(100S_o)$ ratio for which the curves may be used without modification. The solid line represents the divisions between outlet and inlet control. For values of $L/(100S_o)$ less than that shown on the solid line, the culvert is operating under inlet control and the headwater depth is determined from the $L/(100S_o)$ value given on the solid line. The solid-line inlet-control curves are plotted from model test data. The dashed-line outlet-control curves were computed for culverts of various lengths with relatively flat slopes. Free outfall at the outlet was assumed, therefore, tailwater depth is assumed to not influence the culvert performance.

For culverts flowing under outlet control, the head loss at the entrance was computed, and the hydraulic roughness of the various materials used in culvert construction was taken into account in computing resistance loss for full or part-full flow. The Manning's "n" values used for each culvert type ranged from 0.012 to 0.032.

Except for large pipe sizes, headwater depths on the charts extend to three times the culvert height. Pipe arches and oval pipe show headwater up to 2.5 times their height since they are generally used in areas of low fill. The dotted line, stepped across the charts, shows headwater depths of about twice the barrel height and indicates the upper limit of restricted use of the charts. Above this line, the headwater elevation should be checked with the nomographs found in Figures 9-21 through 9-27.

The headwater depth given by the charts is actually the difference in elevation between the culvert invert at the entrance and the total head, that is depth plus velocity head, for flow in the approach channel. In most cases, the water surface upstream from the inlet is close to this level and the chart determination may be used as headwater depth for practical design purposes. Where the approach velocity is in excess of 3.0 feet per second, the velocity head must be subtracted from the curve determination of headwater to obtain the actual headwater depth.

Proper use of the capacity charts (Figures 9-13 through 9-20) will minimize problems of scour or sedimentation. The procedure for sizing the culvert is summarized below. Data can be tabulated in the Design Computation Form, Figure 9-44.

1. List design data: Q(cfs), L(ft), allowable HW (ft), S_o(ft/ft), type of culvert barrel and entrance.

2. Compute $L/(100S_o)$

3. Enter the appropriate capacity chart with the design discharge, Q.

4. Find the $L/(100S_o)$ value for the smallest pipe that will pass the design discharge. If this value is above the dotted line in Figures 9-13 through 9-20, use the nomographs to check headwater conditions.

5. If the computed $L/(100S_o)$ is less than the value of $L/(100S_o)$ given for the solid line, then the value of HW is the value obtained from the solid line curve. If the computed L/(100S) is larger than the value for the dashed outlet-control curve, then special measures must be taken, and the reader is referred to the FHA publications.

   Check the HW value obtained with the allowable HW. If the indicated HW is greater than the allowable HW, then try the next larger pipe size.

The use of the nomographs on Figures 9-21 through 9-27 is limited to cases where tailwater depth is higher than the critical depth in the culvert. The advantage of the capacity charts over the nomographs is that the capacity charts are direct where the nomographs are trial and error. The capacity charts can be used only when the flow passes through critical at the outlet. When the critical depth at the outlet is less than the tailwater depth, the nomographs must be used; however, both will provide the same results where either of the two methods are applicable. The procedure for design requires the use of both nomographs and is as follows:

1. List design data: Q(cfs), L(ft), invert elevations at inlet and outlet (ft), allowable HW(ft), mean and maximum flood velocities in the natural stream (ft/sec), type of culvert, and entrance type for first selection.

2. Determine a trial size by assuming a maximum average velocity based on channel considerations to compute the area, A = Q/V.

3. Find HW for trial size culvert for inlet control and outlet control. For inlet control, use Figures 9-30 through 9-38 connect a straight line through D and Q to Scale 1 of HW/D scales and project horizontally to the proper Scale (2 or 3). Compute HW and, if too large or too small, try another size before computing HW for outlet control.

   Next, compute the HW for outlet control with Figures 9-21 through 9-27. Enter the graph with the length, the entrance coefficient for the entrance type, and the trial size. Connect the length scale and the culvert-size scale with a straight line, pivot on the turning line, and draw a straight line from the design

discharge on the discharge scale through the turning point to the head scale (head loss, H). Compute HW from the head scale (head loss, H). Compute HW from the equation:

$$HW = H + h_o - LS_o$$

For TW greater than or equal to the top of the culvert, ho = TW, and for TW less than the top of the culvert:

$$h_o = \frac{(d_c + D)}{2} \text{ or } TW$$

whichever is greater. If TW is less than d_c, the nomographs cannot be used. See Figures 9-28 and 9-29 for critical-depth charts.

4. Compare the computed headwaters and use the higher HW to determine if the culvert is under inlet or outlet control. If outlet control governs and HW is unacceptable, select a larger trial size and find another HW with the outlet-control nomographs. Since the smaller size of culvert had been selected for allowable HW by the inlet-control nomographs, the inlet control for the larger pipe need not be checked.

9.4.2 Design Procedure

Due to problems arising from topography and other considerations, the actual design of a culvert installation is more difficult than the simple process of sizing culverts. The procedure is a guide to design since the problems encountered are too varied and too numerous to be generalized. However, the actual process presented should be followed to ensure that some special problem is not overlooked.

Design Computation Form - The use of a design computation form is a convenient method to use to obtain consistent designs with a minimum of culvert cost. An example of such a form is Figure 9-44.

Invert Elevations - After determining the allowable headwater elevation, the tailwater elevation, and the approximate length, invert elevations must be assumed. When considering ponded and non-ponded inlets, either for the design discharge or for some lesser storm which will not cause ponding, scour is not likely to occur in an artificial channel, such as a roadside ditch or a major drainage channel, when the culvert has the same slope as the channel. To reduce the chance of failure due to scour, invert elevations corresponding to the natural grade should be used as a first trial. The flow velocity in the channel downstream from the culvert should be investigated to determine if scour will occur.

Culvert Diameter - After the invert elevations have been assumed and using the design computation forms, the capacity charts, and the nomographs, the diameter of pipe that will meet the headwater requirements should be determined. The smallest diameter that appears in the nomographs and capacity charts is 12 inches. Since smaller diameter pipes are often closed by silt, pipes smaller than 18 inches may not be used for any culverts where this manual applies. Since the pipe roughness influences the culvert diameter, both smoothwall and corrugated pipe could be considered in design, if both will satisfy the headwater requirements.

Limited Headwater - If there is insufficient headwater elevation to obtain the required discharge, it is necessary to oversize the culvert barrel, lower the inlet invert, use an irregular cross section, or use any combination of the above.

If the inlet invert is lowered, special consideration must be given to scour. The use of concrete drop structures, rip rap, and headwalls with aprons and toe walls should be investigated and compared to obtain the proper design.

Culvert Outlet - The outlet velocity must be checked to determine if excessive scour will occur downstream. If scour is indicated, then rip rap, an expanding end section, or a more sophisticated energy dissipating structure should be used as discussed in Chapter 10 of this manual.

Minimum Slope - To prevent sediment from plugging the culvert, the culvert slope must be equal to or greater than the slope required to maintain a minimum velocity of 2.5 feet per second for design flow. The slope should be checked for each design, and if the proper minimum velocity is not obtained, the pipe diameter may be decreased, the slope steepened, a smoother pipe used, or a combination of these may be used.

9.5 Culvert Design with Improved Inlets

The objective of this design procedure is the hydraulic design of culverts, using improved inlets. This design procedure hinges on the selection of a culvert barrel based on its outlet-control performance curve, which is unique when based on elevation. The culvert inlet is then manipulated using edge improvements and adjustment of its elevation in order to achieve inlet-control performance compatible with the outlet-control performance. The resultant culvert design will best satisfy the criteria set by the designer and make optimum use of the barrel selected for the site.

9.5.1 Design Procedure

The flow chart shown on Figure 9-7 outlines the steps of the design procedure, and each step is discussed in detail below. Design calculation forms and design charts and tables that are included are located at the end of this chapter.

Step 1. Determine and Analyze Site Characteristics.

Step 2. Perform Hydrologic Analysis.

Step 3 Perform Outlet-Control Calculations and Select Culvert.

Outlet-control calculations are performed before inlet-control calculations in order to select the smallest feasible barrel which can be used without the required headwater elevation in outlet control (HW_o) exceeding the allowable headwater elevation (AHW EL.). For use in this procedure, the equation is in terms of elevation.

The full-flow outlet-control performance curve for a given culvert (size, inlet edge, shape, or material) defines its maximum performance. Therefore, inlet improvements beyond the beveled edge or changes in inlet invert elevation will not reduce the required outlet control headwater elevation. This makes the outlet-control performance curve an ideal limit for improved inlet design.

When the barrel size is increased, the outlet-control curve is shifted to the right, indicating a higher capacity for a given head. (See Figure 9-8) Also, it may be generally stated that increased barrel size will flatten the slope of the outlet-control curve.

The outlet-control curve passing closest to and below the AHW EL., for the design Q on the performance-curve graph defines the smallest possible barrel which will meet the hydraulic design criteria. However, that curve may be very steep (rapidly increasing headwater requirements for discharges higher than design), or the use of such a small barrel may not be practical. The proper culvert selection procedure is:

a. Calculate HW_o at design discharge for trial culvert sizes, entrance conditions, shapes and materials.

b. Calculate headwater elevations at two additional discharge values in the vicinity of the design Q in order to define outlet-control performance.

c. Plot outlet-control performance curves for trial culvert sizes.

d. Select barrel size, shape and material.

This selection should be based solely on calculations which indicate that the required headwater at the design discharge is near the AHW EL., but should also be based on outlet velocity as affected by material selection, the designer's evaluation of site characteristics, and the possible consequences of a flood occurrence in excess of the estimated design flood. A sharply rising outlet-control performance curve may be sufficient reason to select a culvert of different size, shape or material.

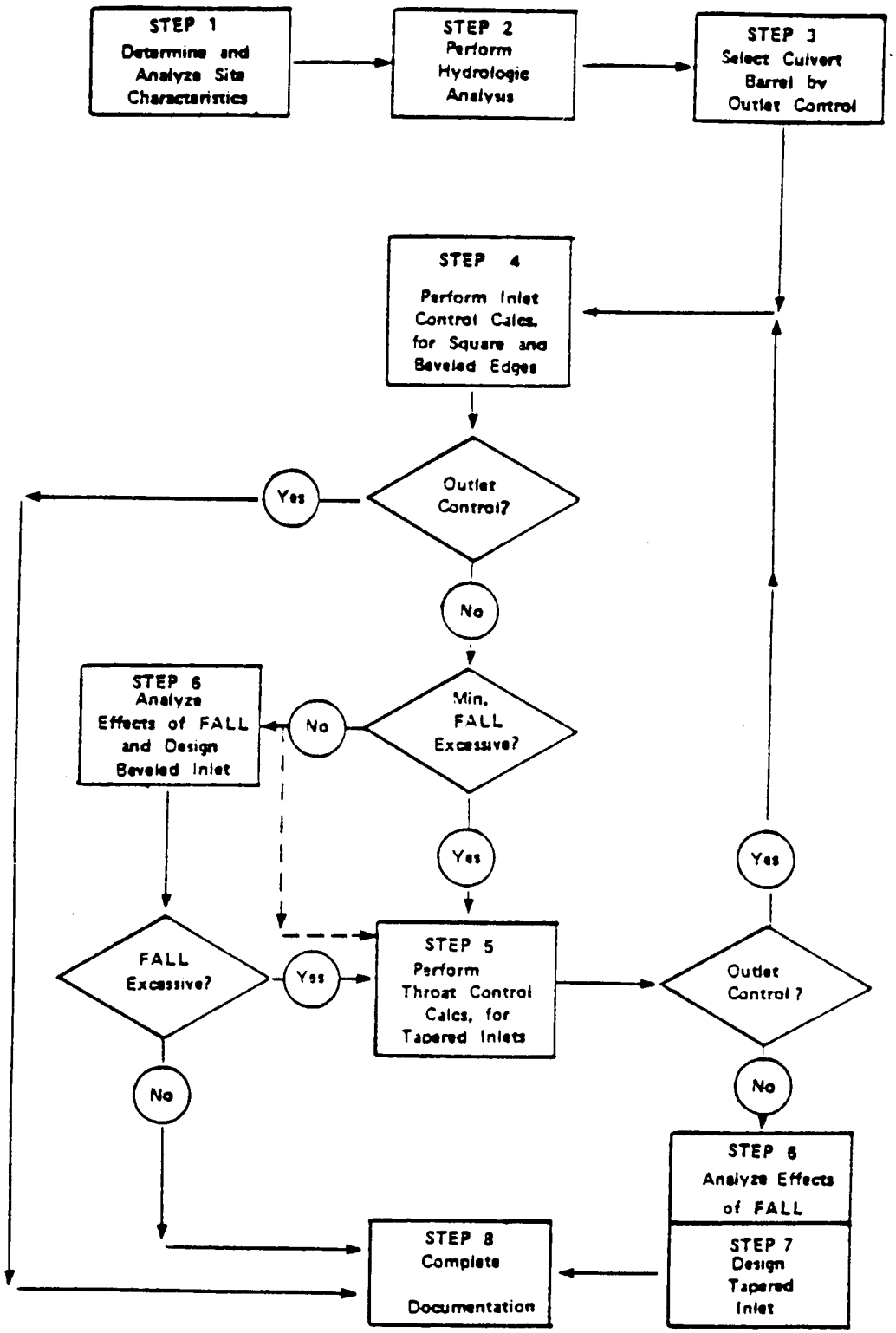


Figure 9-7 Culvert Design Flow Chart

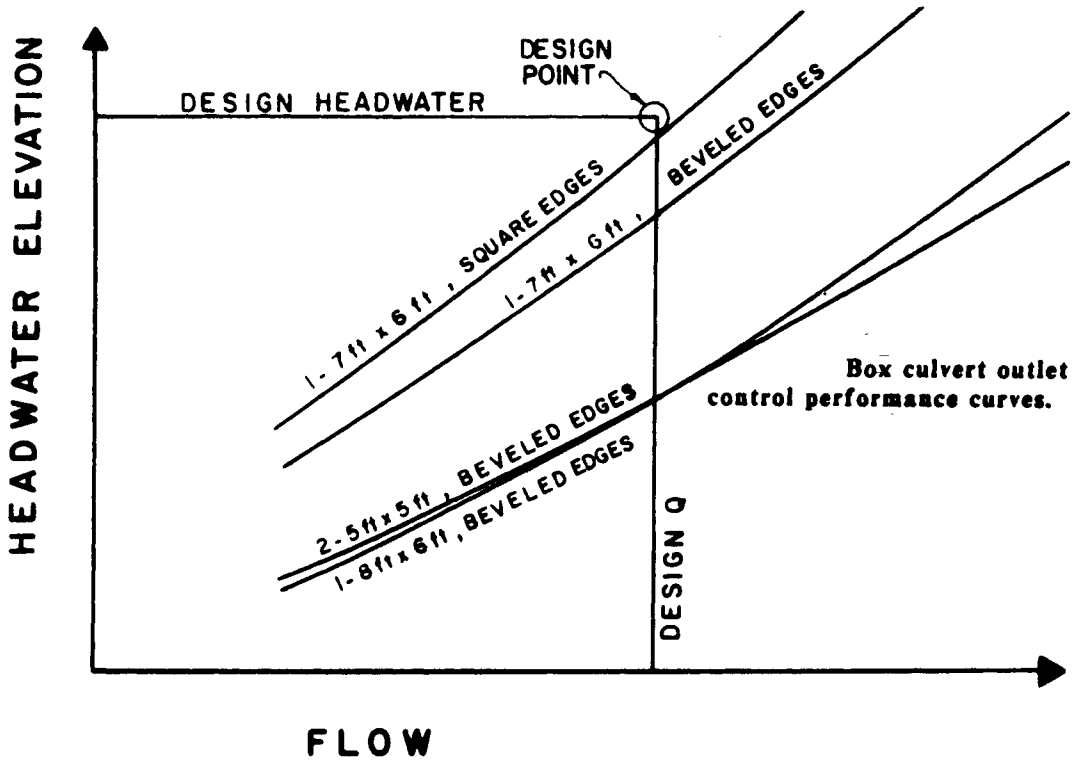


Figure 9-8

In order to calculate the barrel size required in outlet control, the applicable outlet-control nomograph may be used as follows:

a. Intersect the "Turning Line" with a line drawn between Discharge and Head, H. To estimate H, use the following equation:

$$H = \text{AHW EL.} - \text{EL. Outlet Invert} - h_o$$

where h_o may be selected as culvert height. Accuracy is not critical at this point.

b. Using the point on the "Turning Line", k_e, and the barrel length, draw a line defining the barrel size.

This size gives the designer a good first estimate of the barrel size and more precise sizing will follow rapidly.

Step 4. Perform Inlet-Control Calculations for Conventional and Beveled-Edge Culvert Inlets

The calculation procedure is similar to that used in Hydraulic Engineering Circular No. 5, except that headwater is defined as an elevation rather than a depth, a FALL may be incorporated upstream of the culvert face, and performance curves are an essential part of the procedure. The depression or FALL should have dimensions as described for side-tapered inlets.

a. Calculate the required headwater depth, H_f, at the culvert face at design discharge for the culvert selected.

b. Determine required face invert elevation to pass design discharge by subtracting H_f from AHW EL.

c. If this invert elevation is above the steam bed elevation at the face, the invert would generally be placed on the stream bed and the culvert will then have a capacity greater than the design Q with headwater at the AWH EL.

d. If this invert elevation is below the stream bed elevation at the face, the invert must be depressed, and the amount of depression is termed the FALL.

e. Add H_f to the invert elevation to determine HW_f. If HW_f is lower than HW_o, the barrel operates in outlet-control at the design Q. Proceed to Step 8, Figure 9-7.

f. If the FALL is excessive in the designer's judgment from the standpoint of aesthetics, economy and other engineering reasons, a need for inlet geometry refinements is indicated. If square edges were used in Steps 3 and 4 above, repeat with beveled edges. If beveled edges were used, proceed to Step 5.

g. If the FALL is within acceptable limits, determine the inlet-control performance by calculating required headwater elevation using the flow rates from Step 3 and the FALL determined above. $HW_f = H_f$ + EL. face invert.

h. Plot the inlet-control performance curve with the outlet-control performance curve plotted in Step 3.

i. Proceed to Step 6.

Step 5. Perform Throat-Control Calculations for Side-Tapered and Slope-Tapered Inlets

The same concept is involved here as with conventional or beveled-edge culvert design.

a. Calculate required headwater depth on the throat (H_t) at design discharge, the Q for the culvert selected in Step 3.

b. Determine required throat elevation to pass design discharge by subtracting Ht from the AHW EL.

c. If this throat invert elevation is above the stream bed elevation, invert would probably be placed on the stream bed and the culvert throat will have a capacity greater than the design Q with headwater at the AHW EL.

d. If this throat invert elevation is below the stream bed elevation, the invert must be depressed, and the elevation difference between the stream bed at the face and the throat invert is termed the FALL. If the FALL is determined to be excessive, a larger barrel must be selected. Return to Step 5(a).

e. Add H_t to the invert elevation to determine HW_t. If HW_t is lower than HW_o, the culvert operates in outlet control at he design Q. In this case, adequate performance can probably be achieved by the use of beveled edges with a FALL. Return to Step 4.

f. Define and plot the throat-control performance curve.

Step 6. Analyze the effect of FALLS on Inlet-Control Section Performance

It is apparent from Figure 9-9 that either additional FALL or inlet improvements would increase the culvert capacity in inlet-control by moving the inlet-control performance curve to the right toward the outlet-control performance curve. If the outlet-control performance curve of the selected culvert passes below the point defined by the AHW EL. and the design Q, there is an opportunity to optimize the culvert design by selecting the inlet so as to either increase its capacity to the maximum at the AHW EL. or to pass the design discharge at the lowest possible headwater elevation.

Some possibilities are illustrated on Figure 9-10. The minimum inlet control performance which will meet the selected design criteria is illustrated by Curve A. This design has merit in that minimum expense for inlet improvements and/or FALL is incurred and the inlet will pass a flood in excess of the design Q before performance is governed by outlet control. This performance is adequate in many locations, including those locations where headwaters in excess of the AHW EL. would be tolerable on the rare occasion of floods in excess of the design Q.

Curve B illustrates the performance of a design which takes full advantage of the potential capacity of the selected culvert and the site to pass the maximum possible flow at the AHW EL. A safety factor in capacity is thereby incorporated in the design.

This can be accomplished by the use of a FALL, by geometry improvements at the inlet, or by a combination of the two. Additional inlet improvement and/or FALL will not increase the capacity at or above the AHW EL.

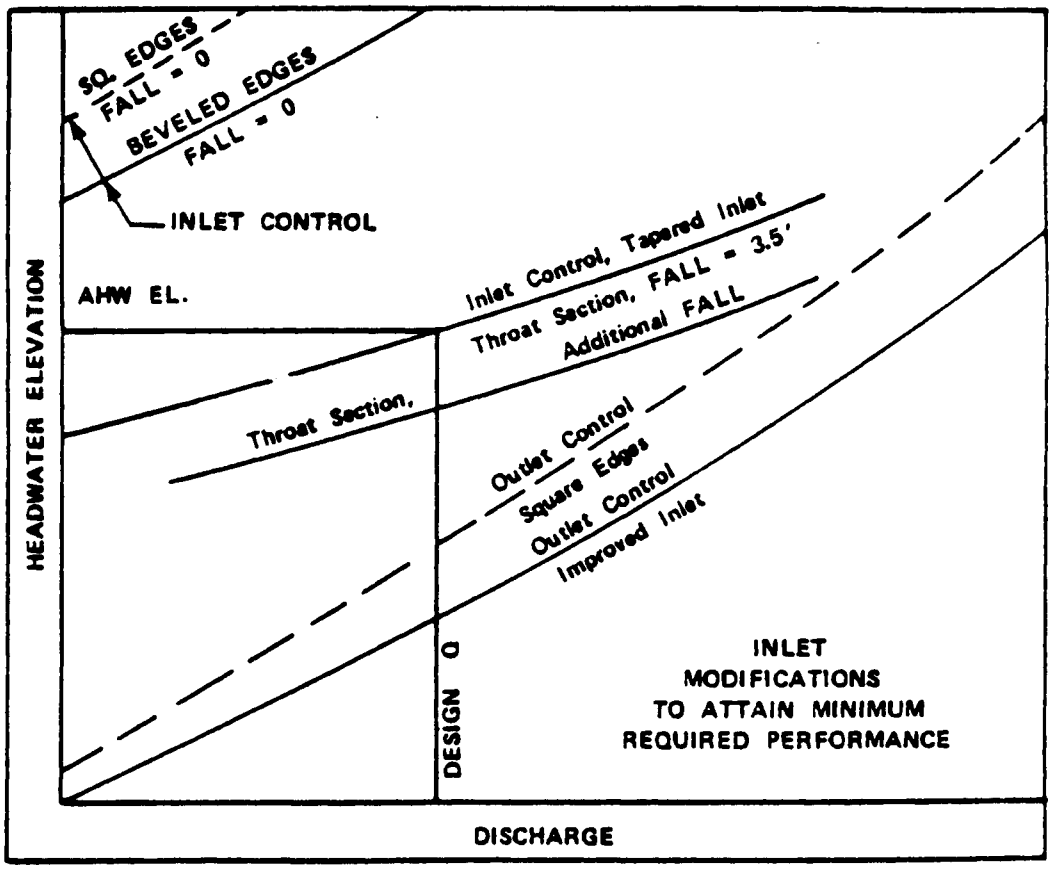
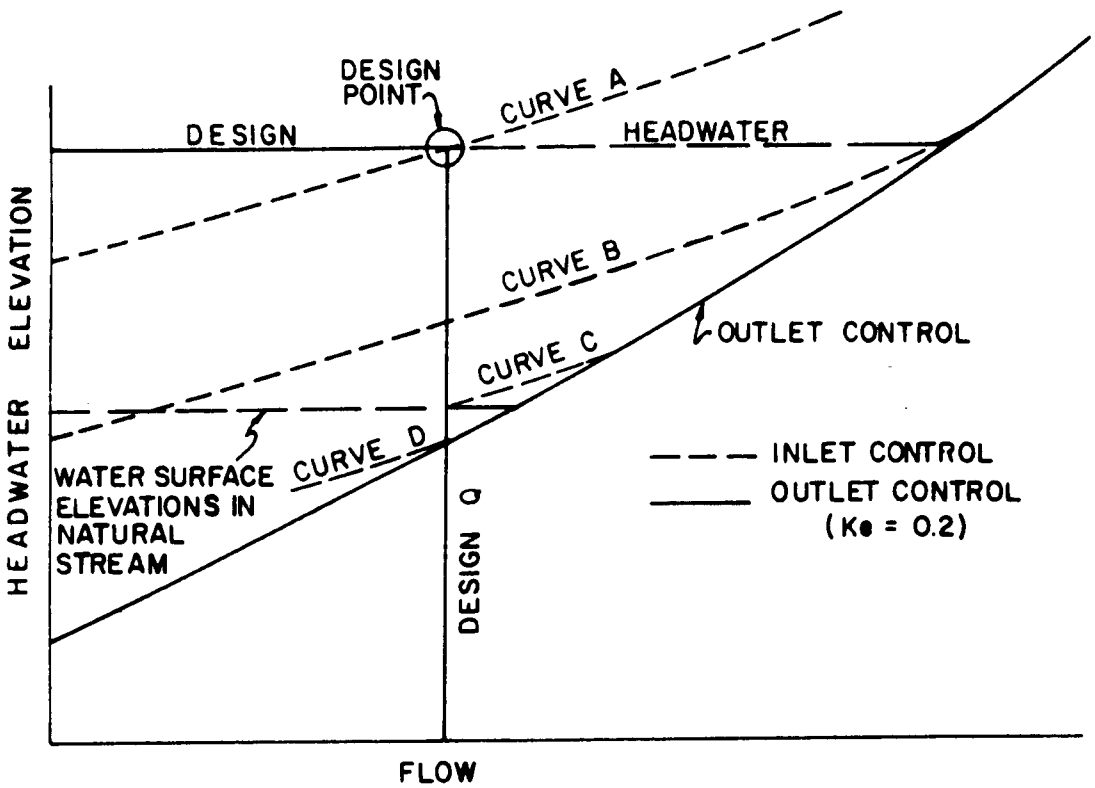


Figure 9-9

There may be reason to pass the design flow at the lowest possible headwater elevation even though the reasons are insufficient to cause the AHW EL. to be set at a lower elevation. The maximum possible reduction in headwater at the design Q is illustrated by Curve C. Additional inlet improvement and/or FALL will not reduce the required headwater elevation at the design Q.



Optimization of Performance in Inlet Control

Figure 9-10

The water-surface elevation in the natural stream may be a limiting factor in design, i.e., it is not productive to design for headwater at a lower elevation than natural streamflow elevations. The reduction in headwater elevation illustrated by Curve C is limited by natural water surface elevations in the stream. If the water surface elevations in the natural stream had fallen below Curve D, this curve would illustrate the maximum reduction in headwater elevation at the design Q. Tailwater depths calculated by assuming normal depth in the stream channel may be used to estimate natural water-surface elevations in the stream at the culvert inlet. These may have been computed as a part of Step 3.

Curve A has been established in either Step 4 for conventional culverts or Step 5 for improved inlets. To define any other inlet-control performance curve such as B, C or D for the same control section:

a. Select a point on the outlet-control performance curve.

b. Measure the vertical distance from this point to Curve A. This is the difference in FALL between Curve A and the curve to be established, e.g., the FALL on the control section for Curve A plus the distance between Curves A and B is the FALL on the control section for Curve B.

For conventional culverts only:

c. Estimate and compare the costs incurred in FALLS (structural excavation and additional culvert length) to achieve various levels of inlet performance.

d. Select design with increment in cost warranted by increased capacity and improved performance.

e. If FALL required to achieve desired performance is excessive, proceed to Step 5.

f. If FALL is acceptable and performance achieves the design objective, proceed to Step 8.

Step 7. Design Side- and/or Slope-Tapered Inlet (Figures 9-39 through 9-43)

Side-tapered inlets have an enlarged face area with a transition to the culvert barrel accomplished by tapering the sidewalls. The inlet has equal barrel and face heights except for the addition of a top bevel at the face. Slope-tapered inlets provide a greater head at the control section. The capacity depends largely upon the amount of FALL between the invert at the face and the invert at the throat section.

The face is the entrance of the barrel perpendicular to the flow-line. The throat lies within the barrel section and is the intersection of the transition section and the barrel. The face or throat control is that section which governs the hydraulics of the structure.

Either a side-tapered or slope-tapered inlet may be used if a FALL is required on the throat by use of a depression (FALL) upstream of the face of a side-tapered inlet or a FALL in the inlet of a slope-tapered inlet.

The face of the side-tapered or the slope-tapered inlet should be designed to be compatible with the throat performance defined in Step 6. The basic principles of selecting the face design are illustrated on Figure 9-11.

The minimum face design is one whose performance curve does not exceed the AHW EL. at the design Q. However, a "balanced design requires that full advantage be taken of the increased capacity and/or lower headwater requirement gained through use of various FALLS. This suggests a face-control curve which intersects the throat-control curve: (1) at the AHW EL., (2) at the design Q, (3) at its intersection with the outlet-control curve, or (4) other. These options are illustrated on Figure 9-11 by points a through e representing the intersections of face-control curves with the throat-control curves. The options are explained as follows: (1) Intersection of face-control and throat-control curves at the AHW EL. (Point a or b): For the minimum acceptable throat-control performance (Curve A), this is the minimum face size that can be used without the required headwater elevation (HW_f) exceeding the AHW EL. at the design Q (Point a). For throat-control performance greater than minimum but equal to or less than Curve B, this is the minimum face design which makes full use of the FALL placed on the throat to increase culvert capacity at the AHW EL. (Point b). (2) Intersection of face-control and throat-control at the design Q (Points a, c or d): This face design results in throat control at discharges equal to or greater than the design Q. It makes full use of the FALL to increase capacity and reduce headwater requirements at flows equal to or greater than the design Q. (3) Intersection of the face-control curve with throat-control curve at its intersection with the outlet-control curve (Points b or e): This option is the minimum face design which can be used to make full use of the increased capacity available from the FALL placed on the throat. It cannot be used where HW_f would exceed AHW EL. at the design Q; e.g., with the minimum acceptable throat-control curve. (4) Variations in the above options available to the designer. The culvert face can be designed so that culvert performance will change from face-control to throat-control at any discharge at which inlet-control governs. Options (1) through (3), however, fulfill design objectives of minimum face size to achieve the maximum increase in capacity possible for a given FALL, or the maximum possible decrease in the required headwater for a given FALL for any discharge equal to or greater than the design Q.

Figure 9-12 illustrates the optional tapered-inlet designs possible. Note that the inlet dimensions for the side-tapered inlet are the same for all options. This is because performance of the side-tapered inlet nearly parallels the performance of the throat, and an increase in headwater on the throat by virtue of an increased FALL results in an almost equal increase in headwater on the face. Each foot of FALL on the throat of a culvert with a side-tapered inlet requires additional barrel length equal to the fill slope; e.g., if the fill slope is 3:1, use of 4 feet of FALL rather than 3 feet results in a culvert barrel 3 feet longer as well as increased culvert capacity and/or reduced headwater requirements.

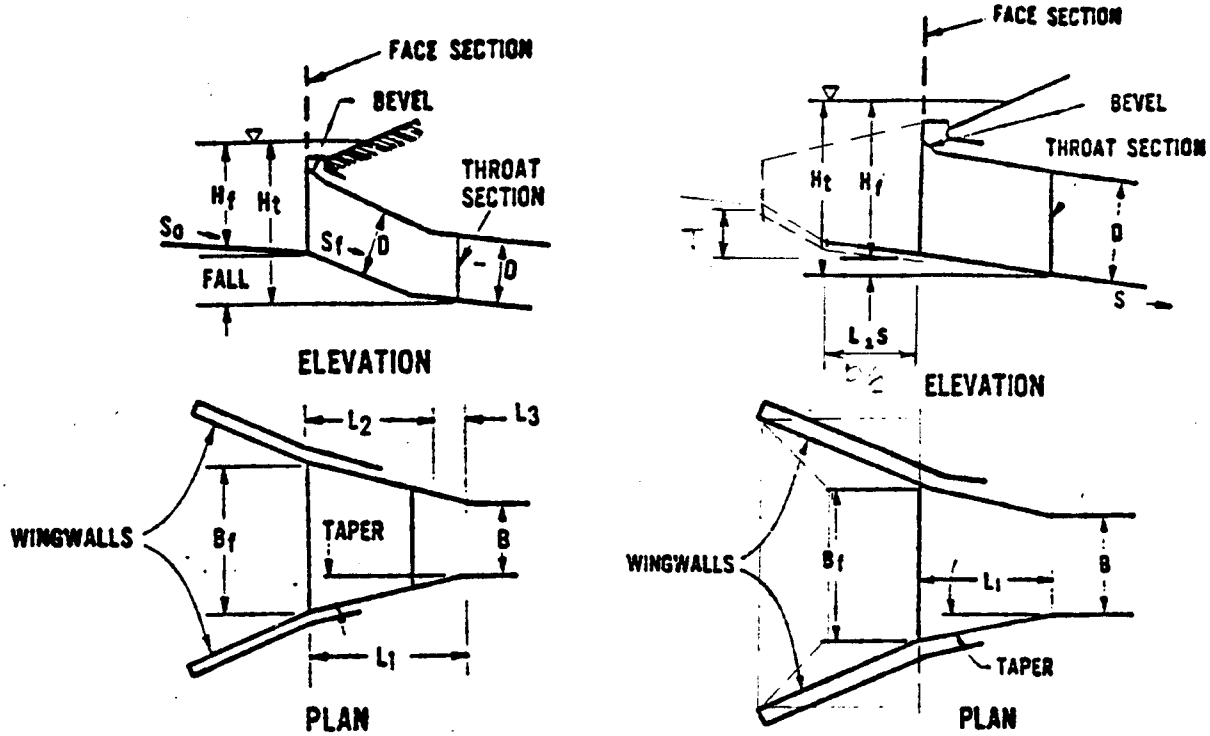


Figure 9-10a
Slope and Side Tapered Inlets

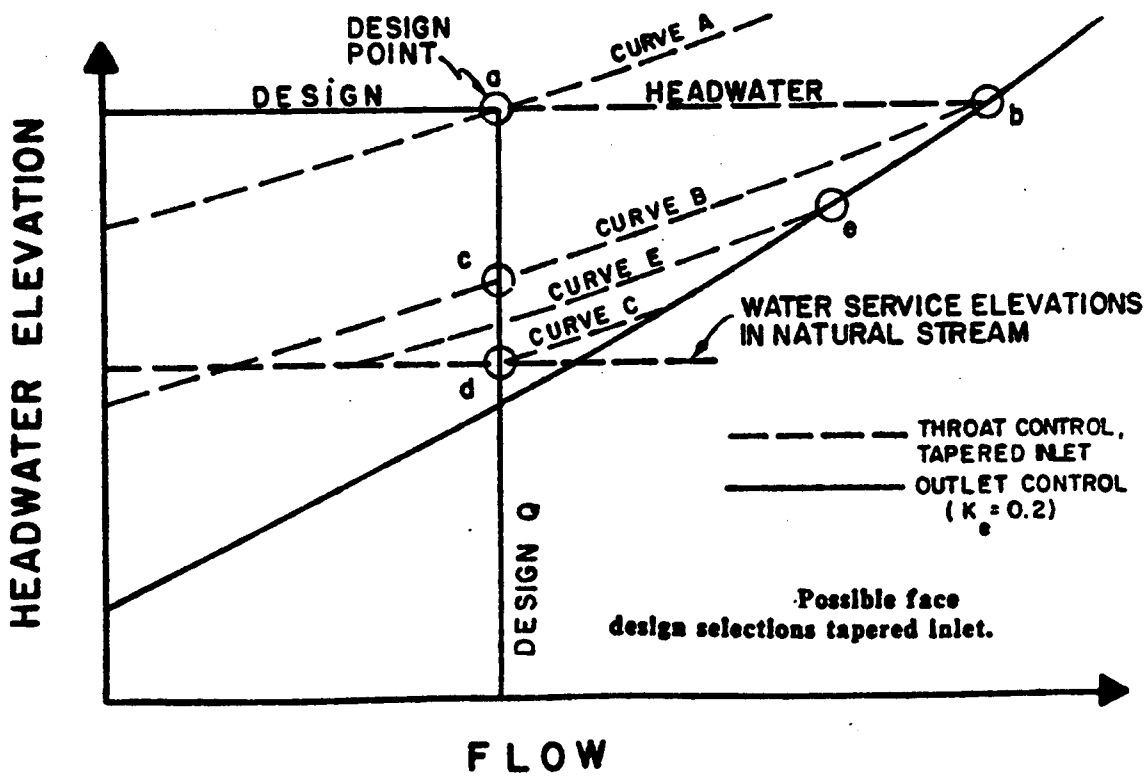
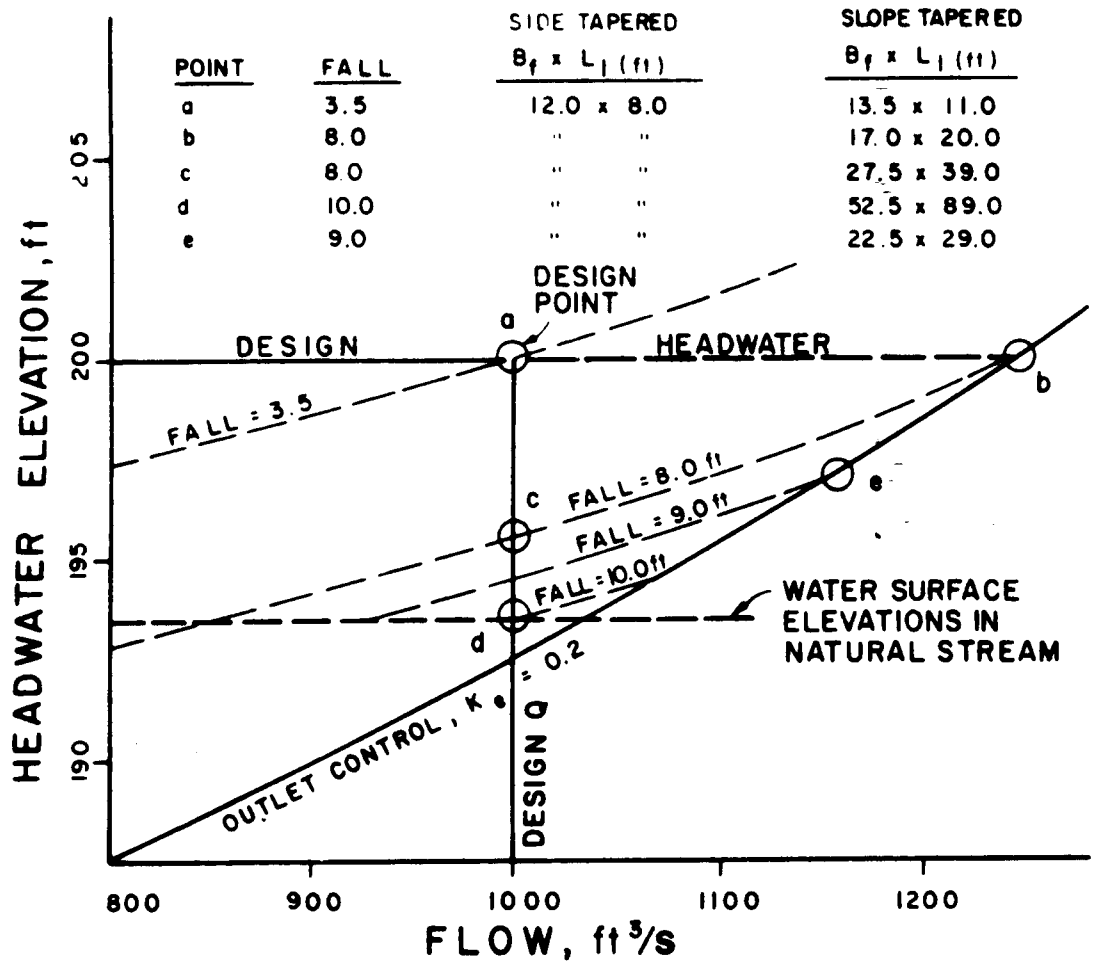


Figure 9-11



Tapered inlet design options for 8 ft by 6 ft reinforced concrete box culvert.

Figure 9-12

Face dimensions and inlet length increase for the slope-tapered inlet as the capacity of the culvert is increased by additional FALL on the throat. No additional head is created for the face by placing additional FALL on the throat. On the other hand use of a greater FALL at the throat of a culvert with a slope-tapered inlet does not increase culvert length.

The steps followed in the tapered-inlet designs are:

a. Compute H_f for side-tapered and slope-tapered inlets for various FALLS at the design Q and other discharges. Side-Tapered Inlet: $H_f = H_t - 1.0$ (approximate).

   Slope-Tapered Inlet: H_f = HW EL. - Streambed EL. at face.

b. Determine dimensions of side-tapered and slope-tapered inlets for trial options.

9/30/90

c. For slope-tapered inlets with mitered face, check for crest control.

d. Compare construction costs for various options, including the cost of FALL on the throat.

e. Select design with incremental cost warranted by increased capacity and improved performance.

From the above, it is apparent that in order to optimize culvert design, performance curves are an integral part of the design procedure. At many culvert sites, designers have valid reasons for providing a safety factor in designs. These reasons include uncertainty in the design discharge estimate, potentially disastrous results in property damage or damage to the highway from headwater elevations which exceed the allowable, the potential for development upstream of the culvert, and the chance that the design flood will be exceeded during the life of the installation.

9.5.2 Dimensional Limitations

Side-Tapered Inlets (See List of Symbols)

1. $6:1 \geq \text{Taper} \geq 4:1$

   Tapers greater than 6:1 may be used but performance will be underestimated.

2. Wingwall flare angle from 15 degrees to 26 degrees with top edge beveled or from 26 degrees to 90 degrees with or without bevels.

3. If FALL is used upstream of face, extend barrel slope upstream from face a distance of D/2 before sloping upward more steeply.

4. For pipe culverts, these additional requirements apply:

   a. $D \leq E \leq 1.1D$

   b. Length of square to round transition $\geq 0.5D$

   c. $FALL \leq D/4$

   d. $P \geq 3T$

   e. $W_p = B_f + T$ or $4T$, whichever is larger.

Slope-Tapered Inlet (See List of Symbols)

1. $6:1 \geq \text{Taper} \geq 4:1$

   Tapers > 6:1 may be used, but performance will be underestimated.

2. $3:1 \geq S_f \geq 2:1$

   If $S_f > 3:1$, use side-tapered design

3. Minimum $L_3 = 0.5B$

4. $1.5D \geq FALL \geq D/4$

   For FALL $< D/4$, use side-tapered design

   For FALL $> 1.5D$, estimate friction losses between face and throat.

5. Wingwall flare angle from 15 degrees to 26 degrees with top edge beveled or from 26 degrees to 90 degrees with or without bevels.

6. For pipe culvert, these additional requirements apply:

   a. Square to circular transition length $\geq$ 0.5D.

   b. Square-throat dimension equal to barrel diameter. It is not necessary to check square-throat

EXAMPLE 1

Box Culvert

Given: Design Discharge, Q = 1,000 cfs, for a 10-year recurrence interval

Slope of stream bed, S_o = 0.05 ft/ft

Allowable Headwater Elevation = 200.0 ft msl

Elevation Outlet Invert = 172.5 ft msl

Culvert Length, L_a = 350 ft

Downstream channel approximates an 8 ft wide trapezoidal channel with 2:1 side slopes and a Manning's "n" of 0.03

Requirements: This box culvert will be located in a rural area where the Allowable Headwater Elevation is not too critical; that is, the damages are low due to exceeding that elevation at infrequent times. Thus, the culvert should have the smallest possible barrel to pass the design Q without exceeding the AHW EL. Use a reinforced concrete box with n = 0.012.

EXAMPLE 1

Conclusion

Since the requirements called for the smallest possible reinforced concrete box culvert, the barrel should be a single 7 ft x 6 ft.

Selection of the inlet would be based on cost. The additional 1.3 ft of FALL gains 62 cfs at AHW EL. = 200.0 but this is not significant at this site. It appears that a side-tapered or slope-tapered design meeting the Q and HW requirements of point 1 would be adequate and the least expensive.

Examination of the outlet control curve shows that a discharge of 1,200 cfs (20% above design) results in an AHW EL. 5.5 ft above design. At this site, no serious flooding of upstream property or the roadway will be caused by such a headwater, and no larger barrel is required.

The dimensions of several alternate inlet structure designs are presented, based on points 1, 2, and 3 on the culvert performance curves. Note that the side-tapered inlets remain about the same size for all FALL values, while the slope-tapered inlets increase in size as FALL increases. However, the side-tapered inlets require an increasingly larger upstream sump as FALL increases. Which design will be more favorable will be a matter of economics and site considerations.

PROJECT: Example No. 1

OUTLET CONTROL
DESIGN CALCULATIONS

DESIGNER: JMN

STATION:

DATE: 12-10-

INITIAL DATA:

Q 10 = 1000 cfs

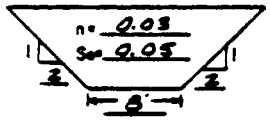
AHW El. = 200 ft.

S_o = 0.05

L_o = 350 ft.

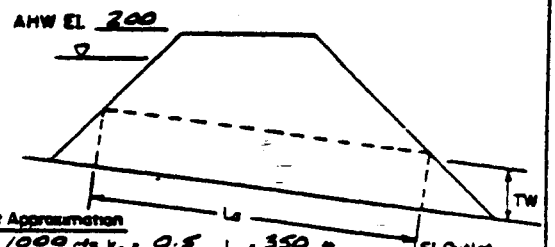
El. Outlet Invert 172.5 ft.

Stream Data:



Barrel Shape and Material Rect. Conc. Box Barrel n = 0.012

SKETCH



First Approximation

Q = 1000 cfs, k_e = 0.5, L_o = 350 ft.

H = AHW El. − El. Outlet Invert − h_o

= 200 − 172.5 − 5 = 22.5

∴ A = 40 ft.² or D = ____ ft.; Try 7'x6'

Q	Q/N	* H	Q/NB	(1) d_c	$\frac{d_c+D}{2}$	Qn	(2) TW	(3) h_o	(4) HW_o	(5) V_o	COMMENTS
Trial No. 1, N = 1, B = 7, D = 6, k_e = 0.5											Square edges
1000	1000	21	143	7.6	6.0		3.5	6.0	199.5	23.8	Ok - Close to AHW El.
800	800	13.2	114	7.6	6.0		3.25	6.0	191.7		
1200	1200	30	172	7.6	6.0		3.8	6.0	208.5		
Trial No. 2, N = 1, B = 7, D = 6, k_e = 0.2											Beveled edges
1000	1000	19	143	7.6	6.0		3.5	6.0	197.5	23.8	Ok - Lowered HW_o 2'
800	800	12		Same as				6.0	190.5		Try 1-6'x6'
1200	1200	27		Sq. edge				6.0	205.5		
Trial No. 3, N = 1, B = 6, D = 6, k_e = 0.2											Beveled edges
1000	1000	26	167	7.6	6.0		3.5	6.0	204.5	27.8	No good - Does not meet
											design Criteria
											exceeds AHW El.

Notes and Equations:

(1) d_c cannot exceed D

(2) TW based on d_n in natural channel, or other downstream control.

(3) $h_o = \frac{d_c+D}{2}$ or TW, whichever is larger.

(4) $HW_o = H + h_o$ − El. Outlet Invert.

(5) Outlet Velocity (V_o) = Q/Area defined by d_c or TW, not greater than D. Do not compute until control section is known.

SELECTED DESIGN

N = 1

B = 7 ft.

D = 6 ft.

k_e = 0.2

At Design Q:

HW_o = 197.5 ft.

V_o = 23.8 ft/s

$$H = \left[1 + k_e - \frac{29n^2 \cdot L}{R^{1.33}}\right] \frac{V^2}{2g}$$

PROJECT: Example No. 1 | CULVERT INLET CONTROL SECTION DESIGN CALCULATIONS | DESIGNER: JMN

STATION: _____ | DATE: 12-10-73

INITIAL DATA:

Q 10 = 1000 cfs

AHW El. = 200 ft.

S_o = 0.05

L_e = 350 ft.

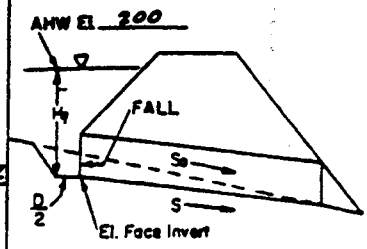
El. Stream Bed at Face 190 ft.

Barrel Shape and Material RCB Barrel n = 0.012

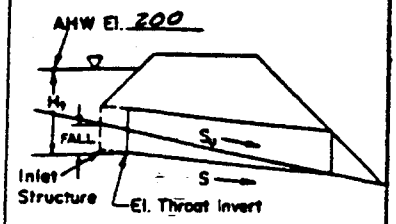
N = 1, B = 7

D = 6, $NBD^{3/2}$ = 102.9 (max. 5)

(Pipe) $ND^{3/2}$ = _____



CONVENTIONAL or BEVELED INLET: FACE CONTROL SECTION (Upper Headings)



TAPERED INLET THROAT CONTROL SECTION (Lower Headings)

DEFINITIONS OF INLET CONTROL SECTION

Q	$\frac{Q}{NB}$ / $\frac{Q}{NBD^{3/2}}$	$\frac{H_f}{D}$ / $\frac{H_t}{D}$	H_f / H_t	(1) El. Face Invert / El. Throat Invert	El. Stream Bed At Face	(2) FALL	(3) HW_f / HW_t	(4) S	(5) V_o	Note: Use Upper Headings for Coventional or Beveled Face; Lower Headings for Tapered Inlet Throat. / COMMENTS
Trial No. 1 Inlet and Edge Description Beveled-edged Inlet										
1000	143	3.9	23.4	176.6	192*	15.4	200			FALL too large, try tapered
* Adjusted upstream due to FALL.										inlet - Do not use bevel-
										edged inlet
Trial No. 2 Inlet and Edge Description Tapered inlet throat										
1000	9.72	2.65	15.9	184.1	190	5.9	200	0.033	34.2	Ok - Calc. Perf. Curves
800	7.79	2.05	12.3				196.9			From plot, Opportunity to [illegible] (FALL = 1.3 + 5.9 = 7.2')
1200	11.68	3.4	20.4				204.5			Max. Capacity at AHW = 200
Trial No. 3 Inlet and Edge Description Tapered inlet throat, FALL = 7.2'										
1000			15.9	182.8	190	7.2	198.7	0.029	33.3	Ok. Capacity at AHW = 200
800			12.3				195.1			1062 c.f.s.
1200			20.4				203.2			

Notes and Equations:

(1) El. Face (or throat) invert = AHW El. − H_f (or H_t)

(2) FALL = El. Stream Bed at Face − El. face (or throat) invert

(3) HW_f (or HW_t) = H_f (or H_t) + El. face (or throat) invert, where El. face (or throat) invert should not exceed El. stream bed.

(4) $S = S_o - FALL/L_e$

(5) Outlet Velocity = Q/Area defined by d_n at S

SELECTED DESIGN

Inlet Description:

FALL = 7.2 ft.

Invert El. = 182.8 ft.

Bevels:

Angle = _____

b = _____ in., d = _____ in.

PROJECT: Example No. 1

DESIGNER: JMN

SIDE-TAPERED INLET
DESIGN CALCULATIONS

STATION:

DATE: 12-10-73

INITIAL DATA

Q 10 = 1000 cfs S_o = 0.05

AHW El. = 200 ft. L_o = 350 ft.

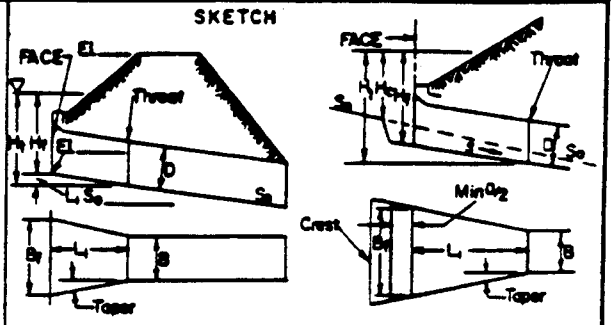
TAPER = 4 : 1

Barrel Shape and Material: RCB; n = 0.012

Face Edge Description: 45° Bevels

N = 1, B = 7 ft, D = 6 ft

SKETCH



Q	El. Throat Invert	(1) H_f/D ~~H_f/E~~	$Q/B_f D^{3/2}$ ~~$Q/A_f E^{1/2}$~~	$D^{3/2}$ ~~$E^{1/2}$~~	(2) Min. B_f ~~Min A_f~~	B_f	(3) L_1	(4) S	$L_1 S$	(5) El. Face Invert	Upper Headings for Box Culverts, Lower Headings for Pipes COMMENTS
Trial No. 1, Q = 1000, HW_f = 200 (Min. required) FALL = 5.9'											
1000	184.1	2.48	6.6	14.7	10.3	10.5	7.0	0.033	0.2	184.3	$B_f D^{3/2}$ [or $A_f E^{1/2}$] = 154
900		2.19	5.84								H_f 18.8, HW_f 197.1
1000		2.48	6.50								14.5, 198.8
1100		2.77	7.14								16.6, 200.9
Trial No. 2, Q = 1000, HW_f = 198.7 (FALL = 7.2')											
1000	182.8	2.48	6.6	14.7	10.3	10.5	7.0	0.029	0.2	183.0	$B_f D^{3/2}$ [or $A_f E^{1/2}$] =
Trial No. 3, Q = 1062, HW_f = 200 (FALL = 7.2')											
1062	182.8	2.70	7.05	14.7	10.3	10.5	7.0	0.029	0.2	183.0	$B_f D^{3/2}$ [or $A_f E^{1/2}$] =

Notes and Equations:

(1) H_f/D [or H_f/E] = (HW_f − El. Throat Invert − 1)/D [or E]

$0 \le E \le 1.1D$

(2) Min. $B_f = Q/\left[(D^{3/2})\ \frac{Q}{B_f D^{3/2}}\right]$

Min. $A_f = Q/\left[(E^{1/2})\ \frac{Q}{A_f E^{1/2}}\right]$

(3) $L_1 = \left[\frac{B_f - NB}{2}\right]$ TAPER

(4) From throat design

(5) El. Face Invert − El. Throat Invert > 1 ft., recompute.
Face and Throat may be lowered to better fit site, but do not raise.

SELECTED DESIGN

B_f = 10.5 ft.

L_1 = 7.0 ft.

Bevels: Angle 45 °

d = 3 in., b = 5.3 in.

Crest Check:

HW_c = 198.7 ft.

H_c = 7.7 ft.

Q/W = 59 (Chart 17)

Min. W = 17 ft.

PROJECT: Example No. 1 | SLOPE-TAPERED INLET DESIGN CALCULATIONS | DESIGNER: JMN

STATION: | DATE: 12-10-73

INITIAL DATA:

Q = 1000 cfs S_o = 0.05

AHW EL. 200 ft L_a = 350 ft

El. Stream bed at crest 191 ft.

El. stream bed at face 190 ft

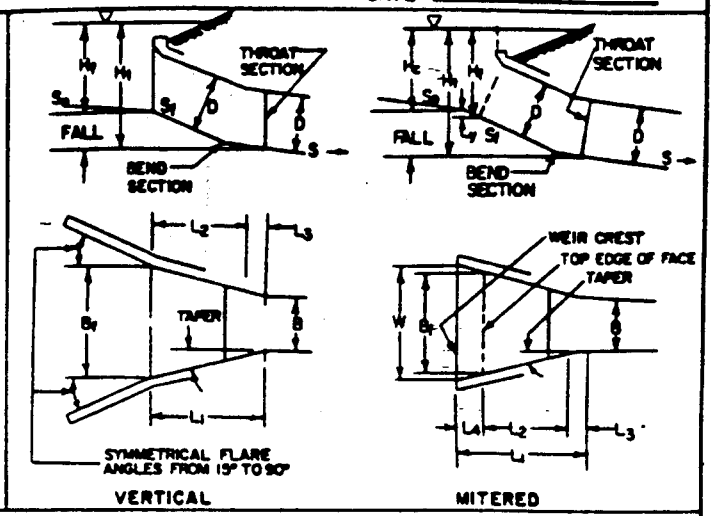
TAPER = 4 :1 (4:1 to 6:1)

S_f = 2 :1 (2:1 to 3:1)

Barrel Shape and Material RCB; n = 0.012

Inlet Edge Description 45° Bevels

N = 1, B = 7 ft, D = 6 ft



	Q	HWi	El. Throat Invert	(1.) El. Face Invert	(2.) Hf	$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	(3.) Min. Bf	Bf	S	Comments
Trial 1	1000	200	184.1	190	10	1.67	5.1	14.7	13.3	14	0.035	$B_f D^{3/2}$ =
												Vertical face point no. 1
Trial 2	1000	198.7	182.8	190	8.7	1.43	4.45	14.7	15.3	16	0.029	$B_f D^{3/2}$ =
												Vertical face point no. 2

Note: Use only throat designs with FALL > 0.25D

(1) El. face invert: Vertical = Approx. stream bed elevation
Mitered = El Crest − y, where y = 0.4D (Approx.), but higher than throat invert elevation.

(2) Hf = HWi − El face invert

(3) Min Bf = Q/((D^3/2) Q/B_f D^3/2)

(4) Min. L_3	(5) L_4	(6) L_2	(7) Check L_2	(8) Adj L_3	(9) Adj TAPER	(10) L_1	(11) W	$\frac{Q}{W}$	H_c	(12.) Max Crest El.
3.5	—	11.8	10.5	—	4.4:1	15.3				
3.5	—	14.4	14.4	3.6	—	18.0				

GEOMETRY
B_f = ___ ft. L_3 = ___ ft.
L_1 = ___ ft. L_4 = ___ ft.
L_2 = ___ ft. d = ___ in.
b = ___ in.
TAPER = ___ :1

(4.) Min. L_3 = 0.5NB

(5.) L_4 = S_{fy} + D/S_f

(6.) L_2 = (El. Face (Crest) Invert − El. Throat Invert) S_f − L_4

(7) Check L_2 = $\left[\frac{B_f - NB}{2}\right]$ TAPER − L_3

(8) If (7)>(6), Adj. L_3 = $\left[\frac{B_f - NB}{2}\right]$ TAPER − L_2

(9) If (6)>(7) Adj. TAPER = $(L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(10) L_1 = L_2 + L_3 + L_4

(11) Mitered: W = NB + 2 $\left[\frac{L_1}{TAPER}\right]$

(12.) Max. Crest El. = HWi − H_c

PROJECT: Example No. 1 — SLOPE-TAPERED INLET DESIGN CALCULATIONS — DESIGNER: JMN

STATION: _____ — DATE: 12-10-73

INITIAL DATA:

Q 10 = 1000 cfs Se = 0.05

AHW El. 200 ft. L_e = 350 ft.

El. Stream bed at crest 191 ft.

El. stream bed at face 190 ft.

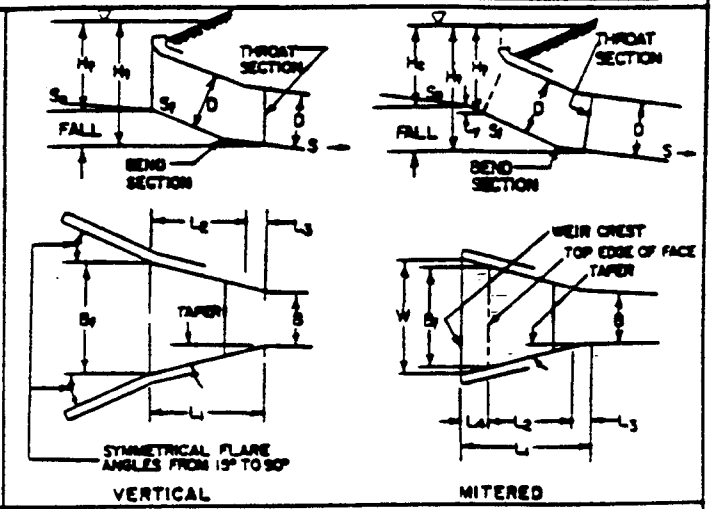
TAPER = 4 :1 (4:1 to 6:1)

S_f = 2 :1 (2:1 to 3:1)

Barrel Shape and Material RCB · n = 0.012

Inlet Edge Description 45° Bevels

N = 1, B = 7 ft, D = 6 ft



	Q	HWf	El. Throat Invert	(1) El. Face Invert	(2) Hf	$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	(3) Min. B_f	B_f	S	Comments
Trial 1	1062	200	182.8	190	10	1.67	5.1	14.7	14.2	15.0	0.029	$B_f D^{3/2}$ = _____
												Vertical face point no. 3
Trial 2	1000	200	184.1	188.6	11.4	1.90	5.65	14.7	12.0	12.0	0.033	$B_f D^{3/2}$ = _____
												Mitered face point no. 1

Note: Use only throat designs with FALL > 0.25D

(1) El. face invert: Vertical = Approx. stream bed elevation
Mitered = El. Crest − y, where y = 0.4D (Approx.), but higher than throat invert elevation.

(2) Hf = HWf − El. face invert

(3) Min B_f = Q/((D^3/2) Q/$B_f D^{3/2}$)

(4) Min. L_3	(5) L_4	(6) L_2	(7) Check L_2	(8) Adj L_3	(9) Adj TAPER	(10) L_1	(11) W	$\frac{Q}{W}$	H_c	(12) Max Crest El.
3.5	—	14.4	12.5	—	4.5:1	17.9				
3.5	7.8	6.0	6.5	4.0	—	17.8	15.9	63.0	8.0	192.0

GEOMETRY
B_f = ___ ft. L_3 = ___ ft.
L_1 = ___ ft. L_4 = ___ ft.
L_2 = ___ ft. d = ___ in.
b = ___ in.
TAPER = ___ :1

(4) Min. L_3 = 0.5 NB

(5) L_4 = S_f − D/S_f

(6) L_2 = (El. Face (Crest) Invert − El. Throat Invert) S_f − L_4

(7) Check L_2 = $\left[\frac{B_f - NB}{2}\right]$ TAPER − L_3

(8) If (7) > (6), Adj. L_3 = $\left[\frac{B_f - NB}{2}\right]$ TAPER − L_2

(9) If (6) > (7) Adj. TAPER = (L_2 + L_3) / $\left[\frac{B_f - NB}{2}\right]$

(10) L_1 = L_2 + L_3 + L_4

(11) Mitered: W = NB + 2 $\left[\frac{L_1}{TAPER}\right]$

(12) Max. Crest El. = HWf − H_c

PROJECT: Example No. 1 | SLOPE-TAPERED INLET DESIGN CALCULATIONS | DESIGNER: JMN

STATION: _____ | DATE: 12-10-75

INITIAL DATA:

Q 10 = 1000 cfs S_o = 0.05

AHW EL. 200 ft L_o = 350 ft

El. Stream bed at crest 191 ft.

El. stream bed at face 190 ft

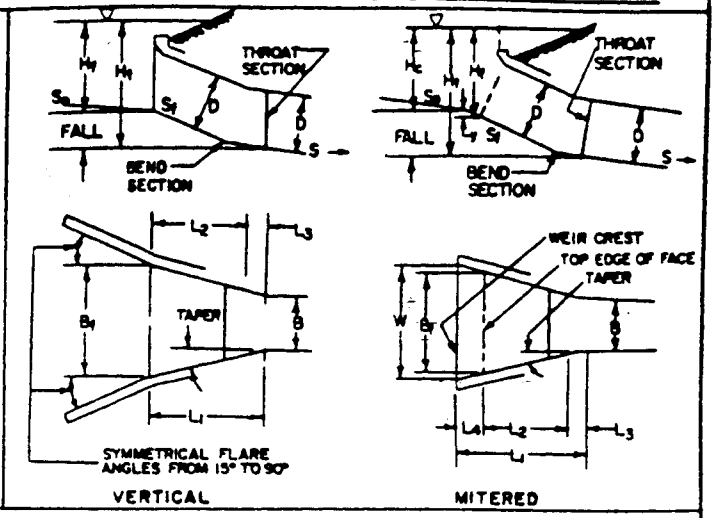
TAPER = 4 :1 (4:1 to 6:1)

S_f = 2 :1 (2:1 to 3:1)

Barrel Shape and Material RCB ; n = 0.012

Inlet Edge Description 45° Bevels

N = 1, B = 7 ft, D = 6 ft



	Q	HWf	El Throat Invert	(1) El Face Invert	(2.) Hf	Hf/D	Q/(Bf D^3/2)	D^3/2	(3.) Min. Bf	Bf	S	Comments
Trial 1	1000	198.7	182.8	188.6	10.1	1.68	5.15	14.7	13.5	14.0	0.029	Bf D^3/2 = _____
												Mitered face point no. 2
Trial 2	1062	200	182.8	188.6	11.4	1.90	5.65	14.7	12.8	13.0	0.029	Bf D^3/2 = _____
												Mitered face point no. 3

Note: Use only throat designs with FALL > 0.25D

(1) El. face invert: Vertical = Approx. stream bed elevation
Mitered = El Crest − y, where y = 0.4D (Approx.), but higher than throat invert elevation

(2) Hf = HWf − El face invert

(3) Min Bf = Q/((D^3/2)) Q/(Bf D^3/2))

(4) Min L3	(5) L4	(6) L2	(7) Check L2	(8) Adj L3	(9) Adj TAPER	(10) L1	(11) W	Q/W	Hc	(12) Max Crest El.
3.5	7.8	8.6	10.5	5.4	—	21.8	17.9	55.9	7.4	191.3
3.5	7.8	8.6	8.5	—	4.03:1	19.9	16.8	63.3	7.9	192.1

GEOMETRY
Bf = ___ ft. L3 = ___ ft.
L1 = ___ ft L4 = ___ ft
L2 = ___ ft d = ___ in.
b = ___ in.
TAPER = ___ :1

(4) Min. L_3 = 0.5NB

(5) L_4 = S_f + D/S_f

(6) L_2 = (El. Face (Crest) Invert − El. Throat Invert) S_f − L_4

(7) Check L_2 = $\left[\frac{B_f - NB}{2}\right]$ TAPER − L_3

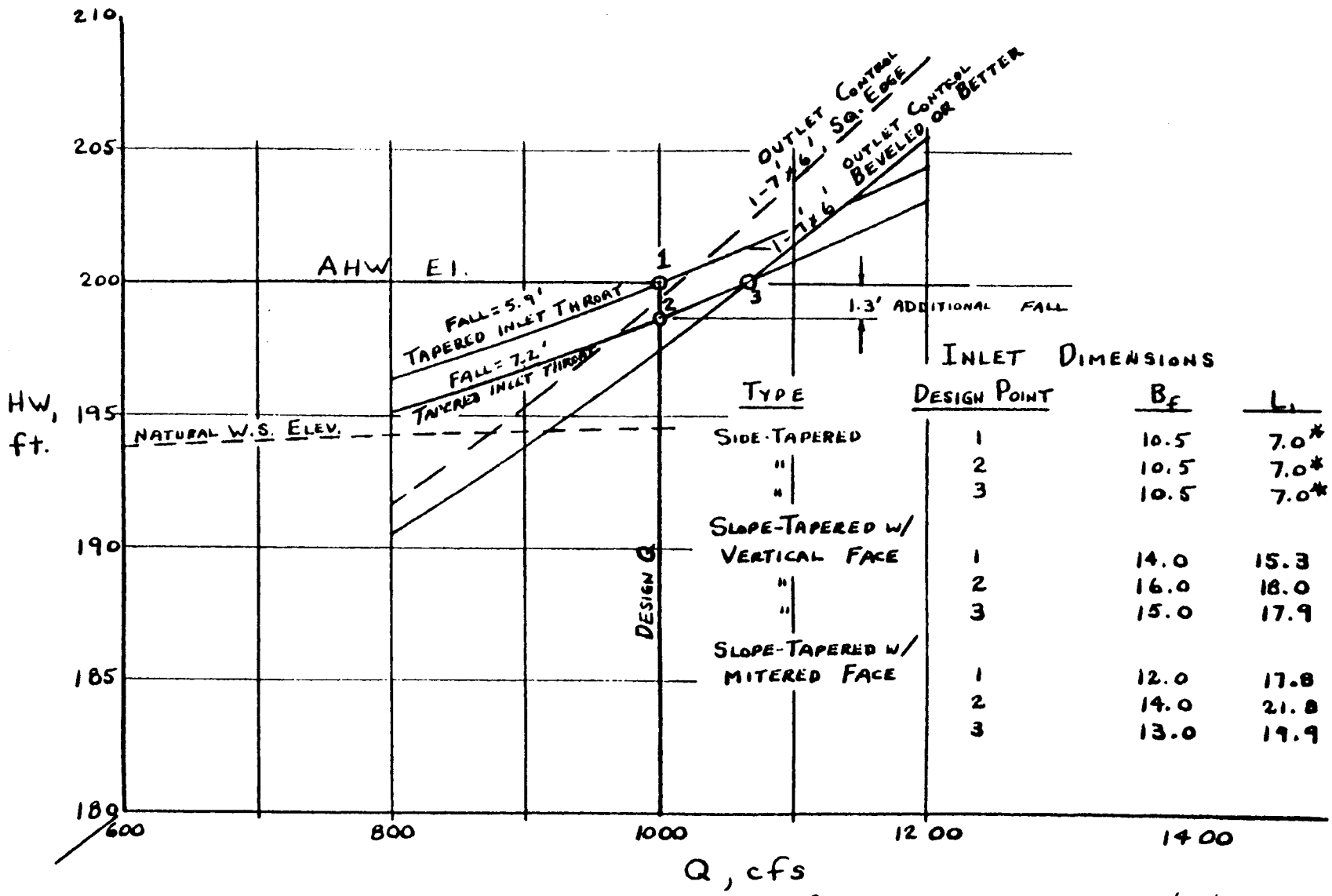
(8) If (7)>(6), Adj. L_3 = $\left[\frac{B_f - NB}{2}\right]$ TAPER − L_2

(9) If (6)>(7) Adj. TAPER = $(L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(10) L_1 = L_2 + L_3 + L_4

(11) Mitered W = NB + 2 $\left[\frac{L_1}{TAPER}\right]$

(12) Max Crest El. = HWf − Hc



INLET DIMENSIONS

TYPE	DESIGN POINT	B_f	L_1
SIDE-TAPERED	1	10.5	7.0*
"	2	10.5	7.0*
"	3	10.5	7.0*
SLOPE-TAPERED W/ VERTICAL FACE	1	14.0	15.3
"	2	16.0	18.0
"	3	15.0	17.9
SLOPE-TAPERED W/ MITERED FACE	1	12.0	17.8
	2	14.0	21.8
	3	13.0	19.9

DESIGN PERFORMANCE CURVES - 1-7'x6' RCB

EXAMPLE 2

Pipes Culverts

Given: Design Discharge, Q_{10} = 150 cfs

Allowable Headwater Elevation = 100.0 ft msl

Elevation Outlet Invert = 75.0 ft msl

Culvert Length, La = 350 ft

Downstream channel approximates a 5 ft wide trapezoidal channel with 2:1 side slopes, a Manning's "n" of 0.03, and S_o = 0.05.

Requirements: This pipe is located in a suburban area where the AHW EL. may be exceeded by 2 to 3 feet without extreme damage. However, headwater elevations greater than 103.0 feet should be avoided for flows significantly higher than the design Q of 150 cfs.

EXAMPLE 2

Conclusion

From the performance curves, beveled edges meet the AHW EL. of 100 ft and Q = 150 cfs, while the use of a side-tapered inlet would increase Q to 170 cfs at AHW EL. = 100 ft. In both cases, the FALL = 0. It appears that the beveled-edge inlet would be sufficient and the least costly in this case, since the culvert performance curve does not exceed 103.0 feet until Q is 186 cfs.

For additional examples, consult Appendix A of Reference 4.

9.6 Design Figures

Capacity charts and nomographs covering the range of applications commonly encountered in urban drainage are contained in this section. These charts are from Federal Highway Administration publications. For situations beyond the range covered by these charts, reference should be made to the original publications.

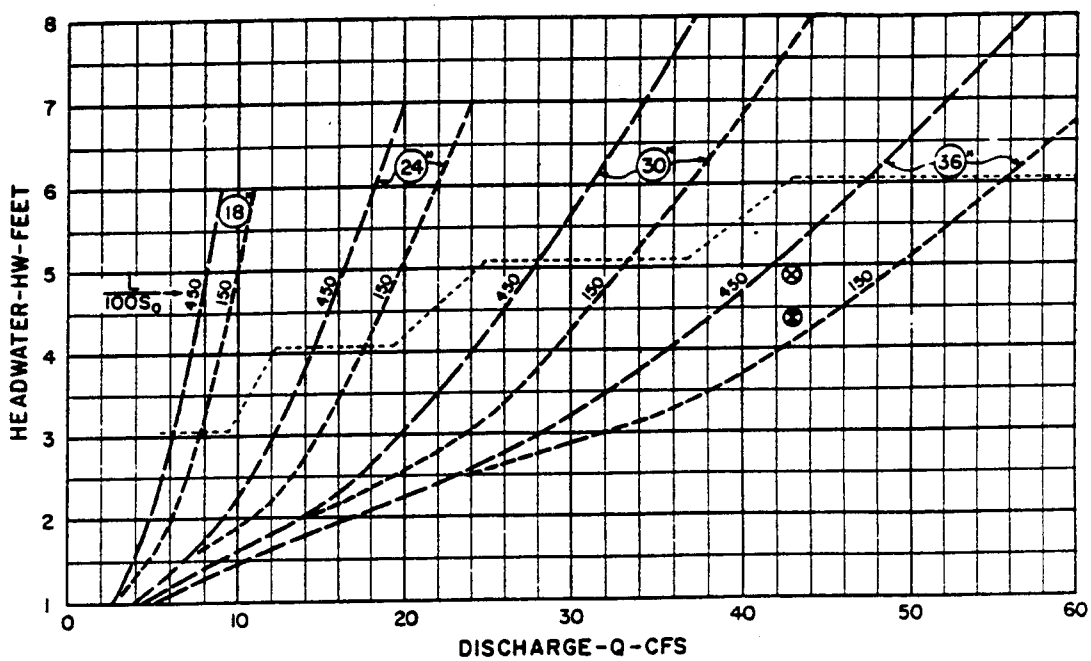
Table 9-1

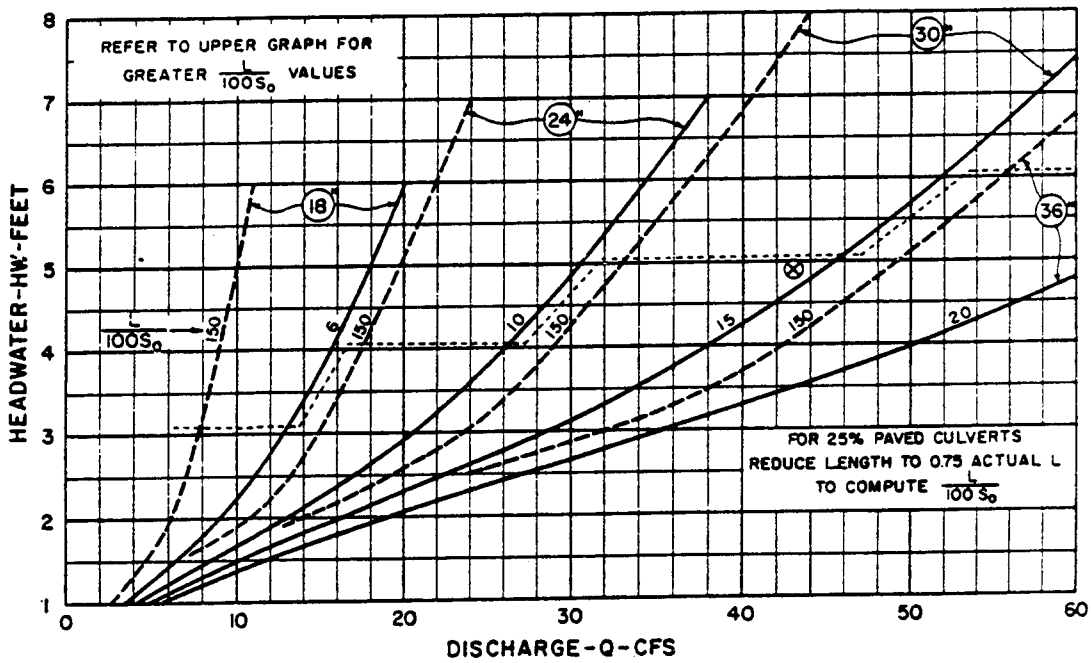
ENTRANCE LOSS COEFFICIENTS
Outlet Control, Full or Partly Full

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square-cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove end)	0.2
Square-edged	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End section conforming to fill slope	0.5
Beveled edges, 33.7-degree to 45-degree bevels	0.2
Side-tapered or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End section conforming to fill slope	0.5
Beveled edges, 33.7-degree to 45-degree bevels	0.2
Side-tapered or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30 degrees to 75 degrees to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10 degrees to 25 degrees to barrel	
Square-edged at crown	0.5
Wingwall parallel (extension of sides)	
Square-edged at crown	0.7
Side-tapered or slope-tapered inlet	0.2

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

Federal Highway Administration, Hec No. 13





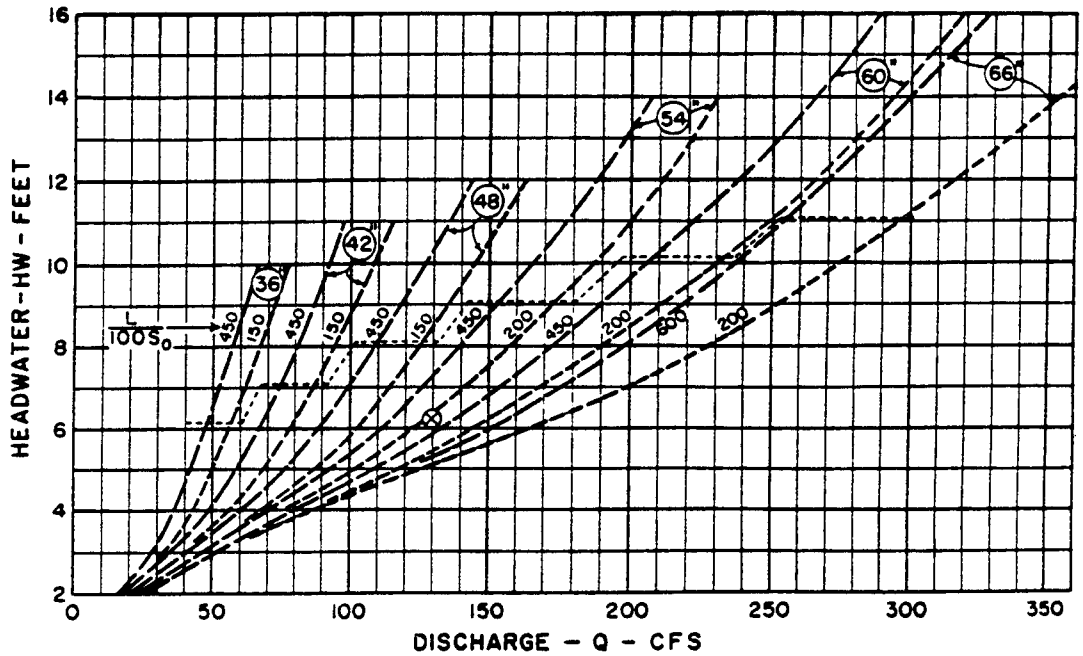
EXAMPLE

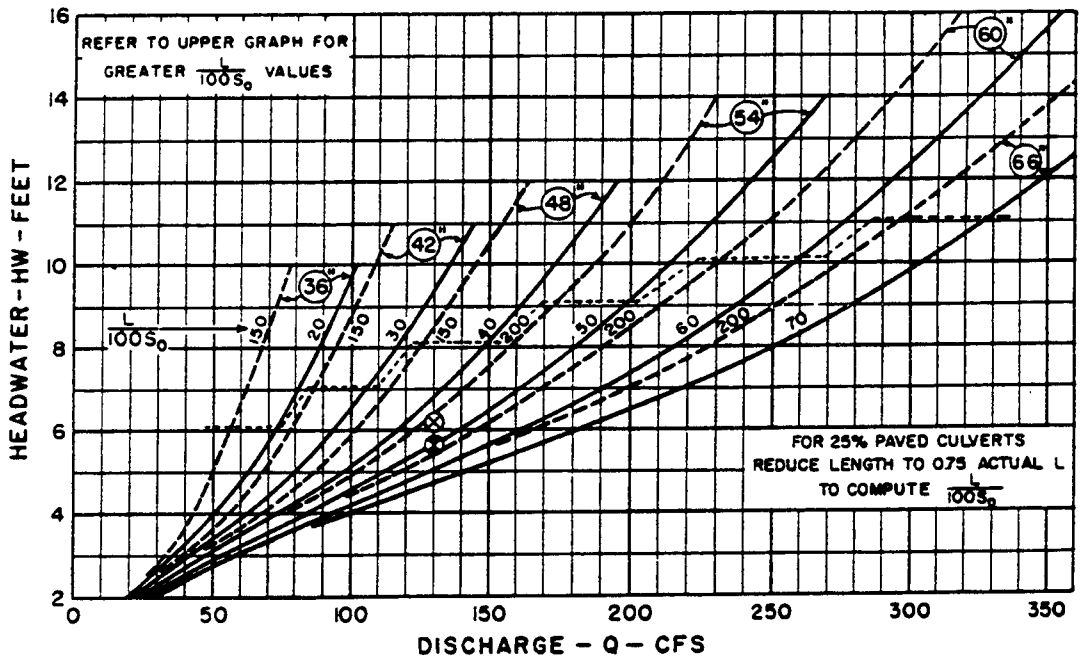
⊗ GIVEN:
43 CFS; AHW = 4.9 FT.
L = 72 FT.; S_0 = 0.003

⊗ SELECT 36" UNPAVED
HW = 4.4 FT.

BUREAU OF PUBLIC ROADS JAN 1963

CULVERT CAPACITY STANDARD CIRCULAR CORR. METAL PIPE HEADWALL ENTRANCE 18" TO 36" ○





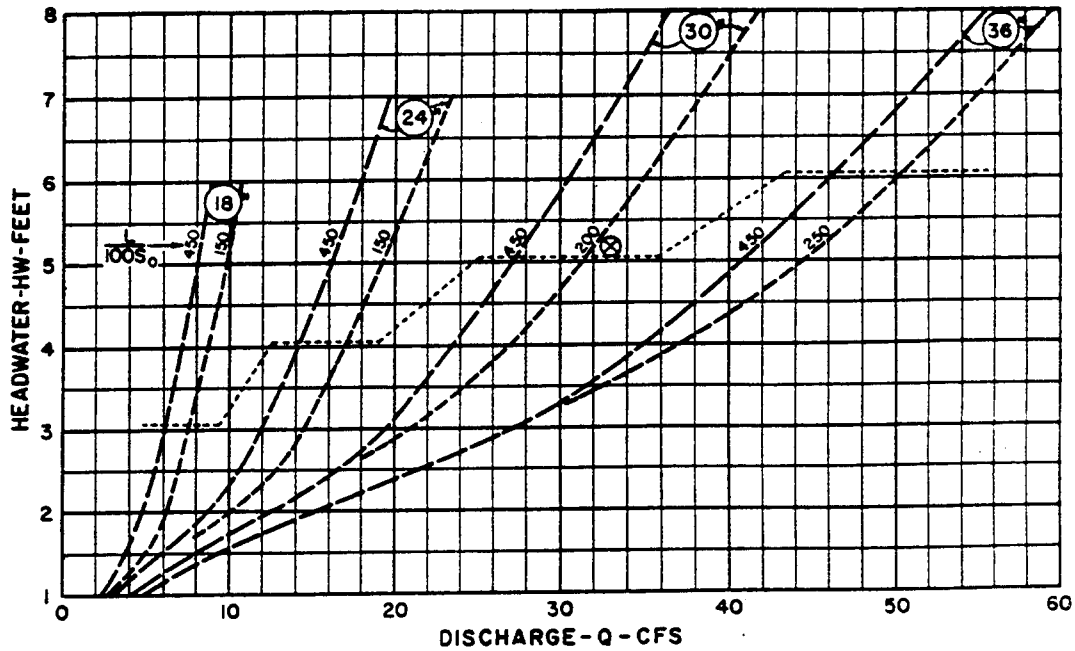
EXAMPLE

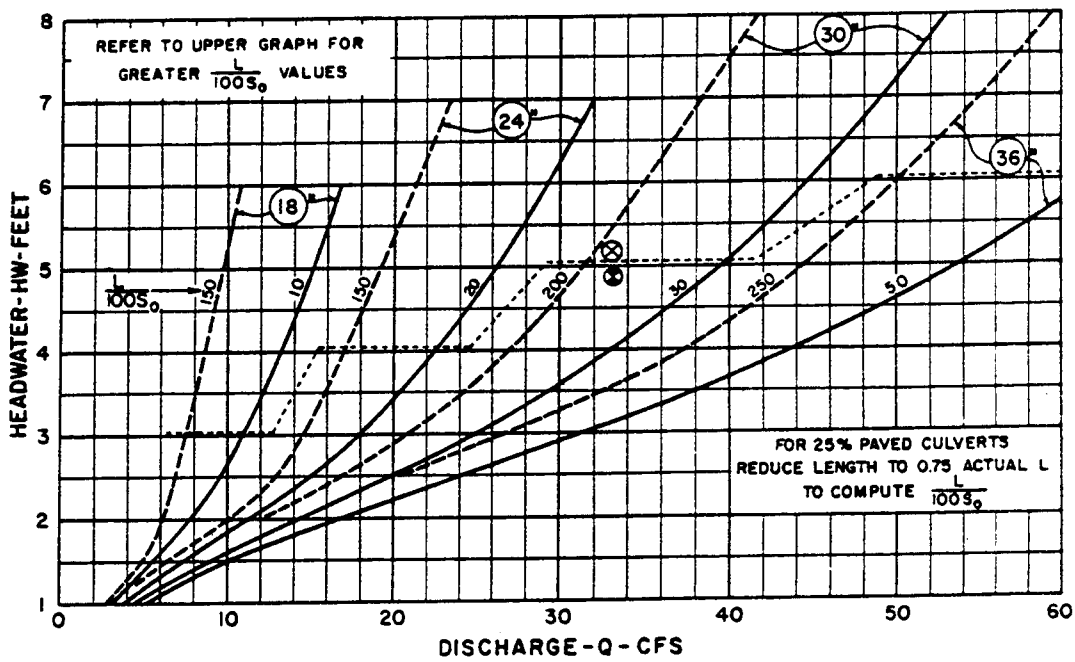
⊗ GIVEN:
130 CFS; AHW = 6.2 FT.
L = 120 FT.; S_0 = 0.025

⊗ SELECT 54" UNPAVED
HW = 5.6 FT.

BUREAU OF PUBLIC ROADS JAN. 1963

CULVERT CAPACITY STANDARD
CIRCULAR CORR. METAL PIPE HEADWALL ENTRANCE
36" TO 66" ○





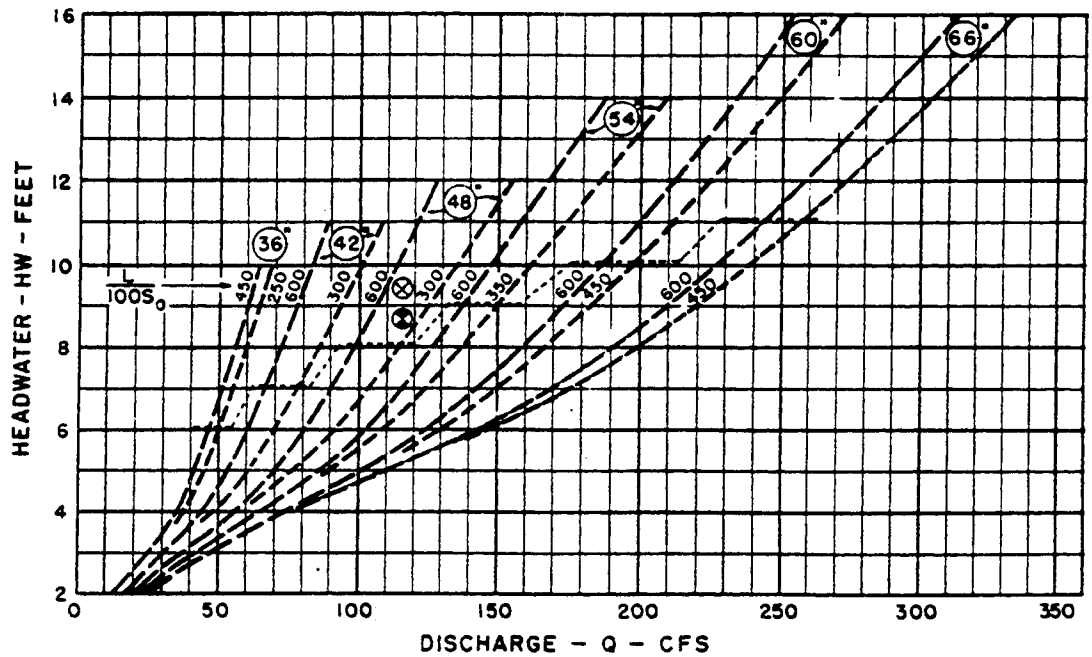
EXAMPLE

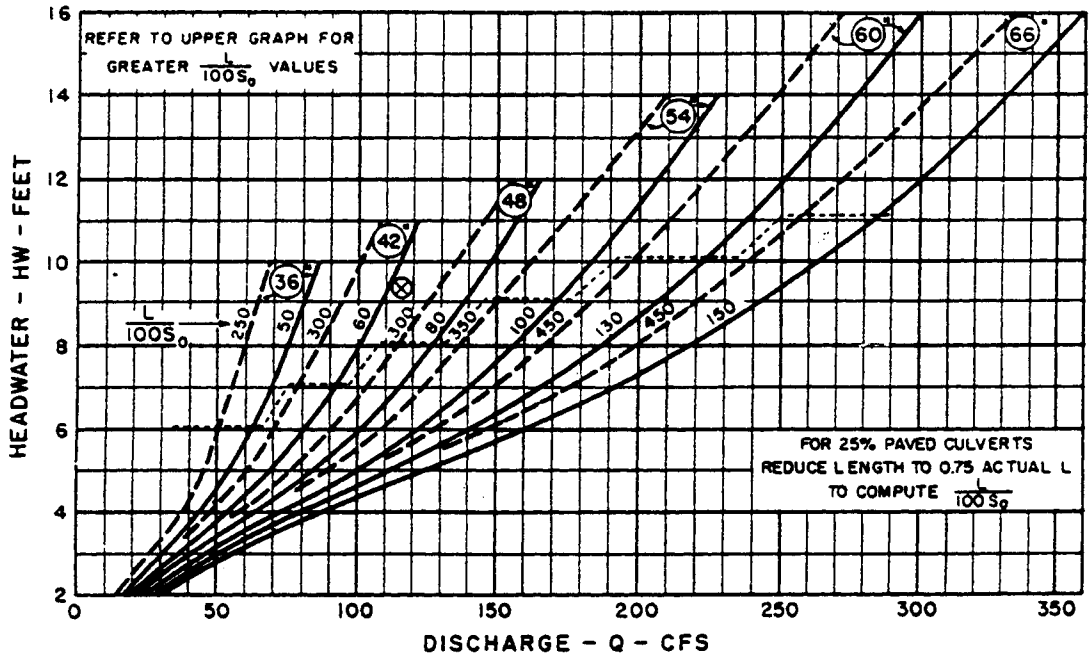
⊗ GIVEN:
33 CFS; AHW = 5.2 FT.
L = 70 FT.; S_0 = 0.005

⊗ SELECT 30" UNPAVED
HW = 4.9 FT.

CULVERT CAPACITY STANDARD CIRCULAR CORR. METAL PIPE PROJECTING ENTRANCE 18" TO 36" ○

BUREAU OF PUBLIC ROADS JAN. 1963





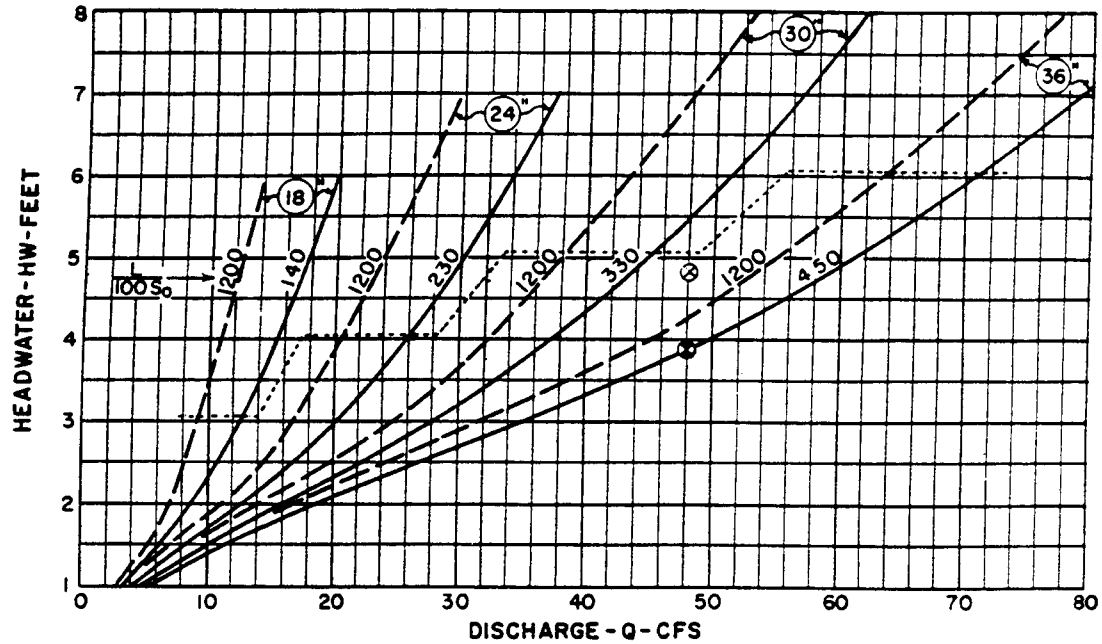
EXAMPLE

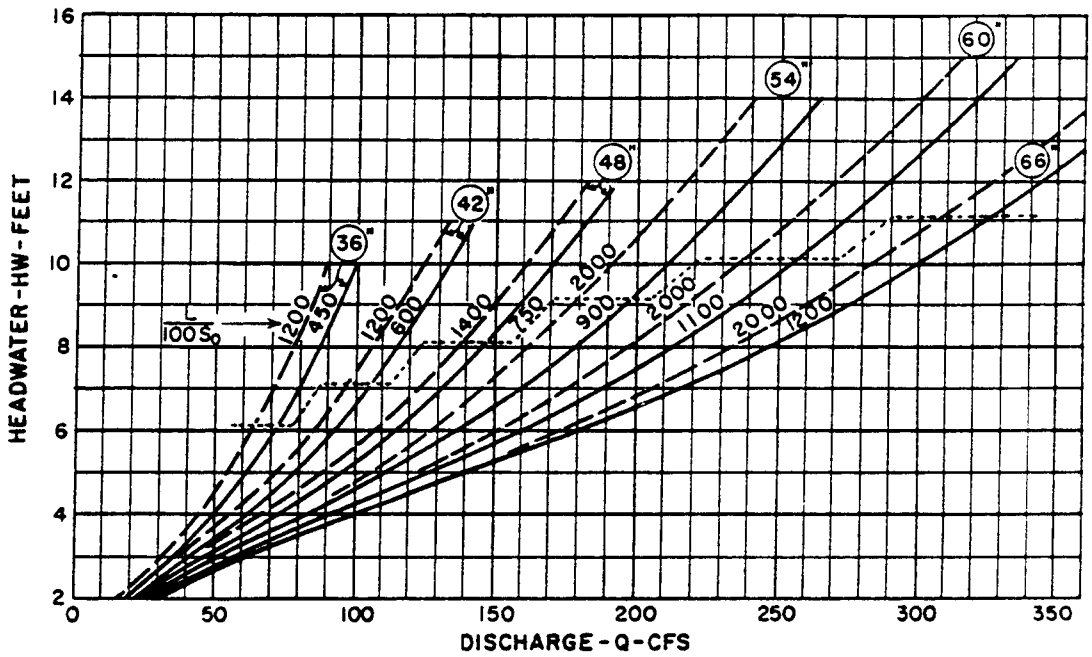
⊗ GIVEN:
115 CFS; AHW = 9.4 FT.
L = 135 FT.; S_0 = 0.0034

⊗ SELECT 48" UNPAVED
HW = 8.6 FT.

BUREAU OF PUBLIC ROADS JAN. 1963

CULVERT CAPACITY STANDARD
CIRCULAR CORR. METAL PIPE PROJECTING ENTRANCE 36" TO 66" ○





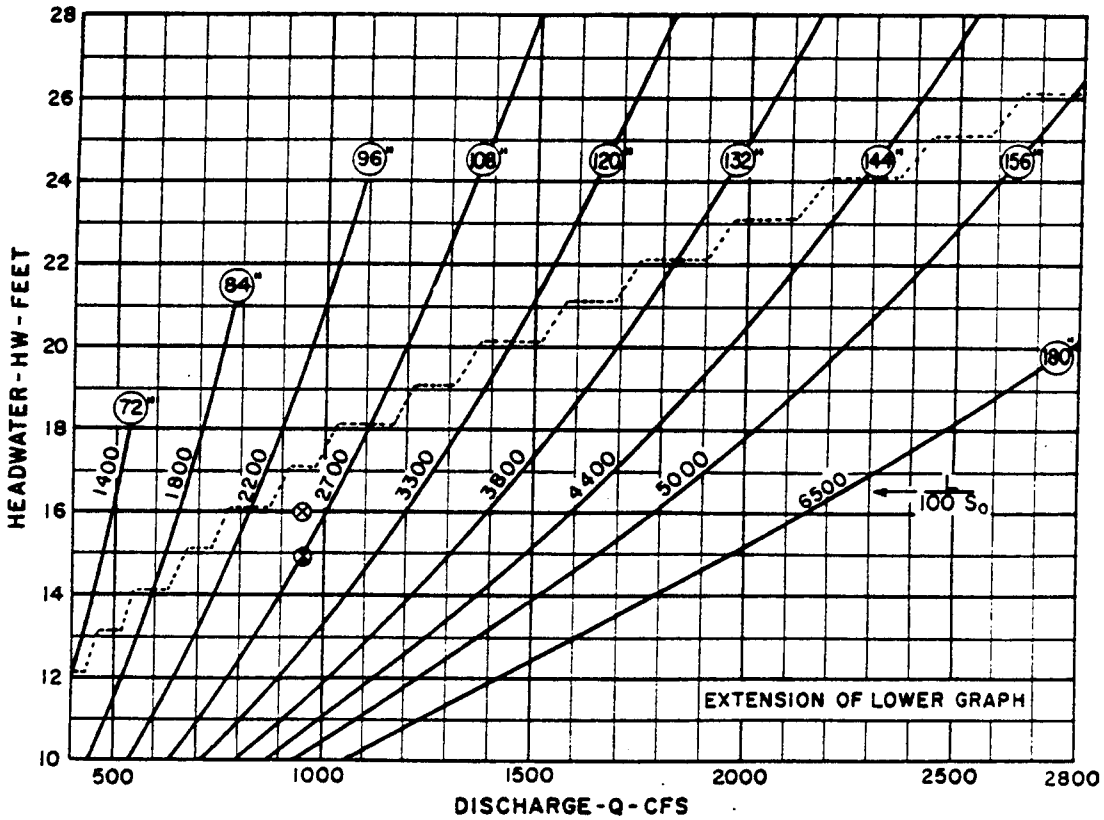
EXAMPLE

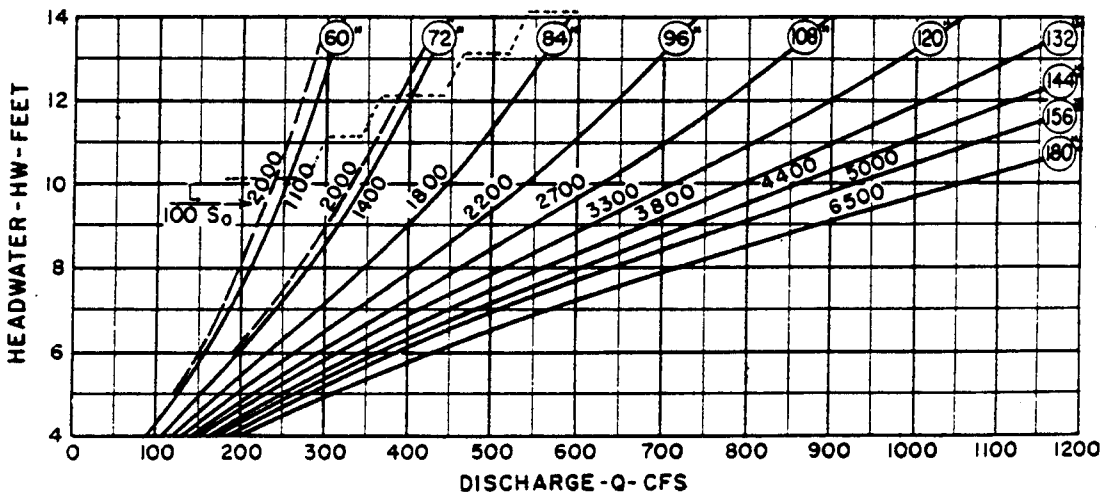
⊗ GIVEN:
48 CFS; AHW = 4.8 FT.
L = 60 FT.; S_o = 0.003

⊗ SELECT 36"
HW = 3.9 FT.

BUREAU OF PUBLIC ROADS JAN. 1963

CULVERT CAPACITY
CIRCULAR CONCRETE PIPE
SQUARE-EDGED ENTRANCE
18" TO 66"





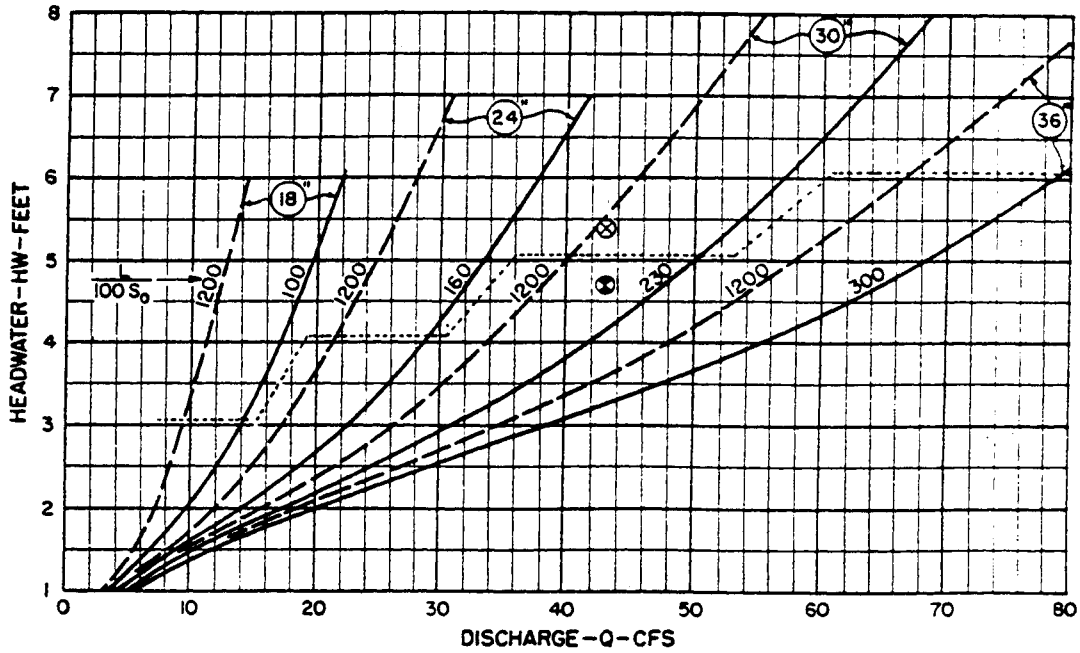
EXAMPLE

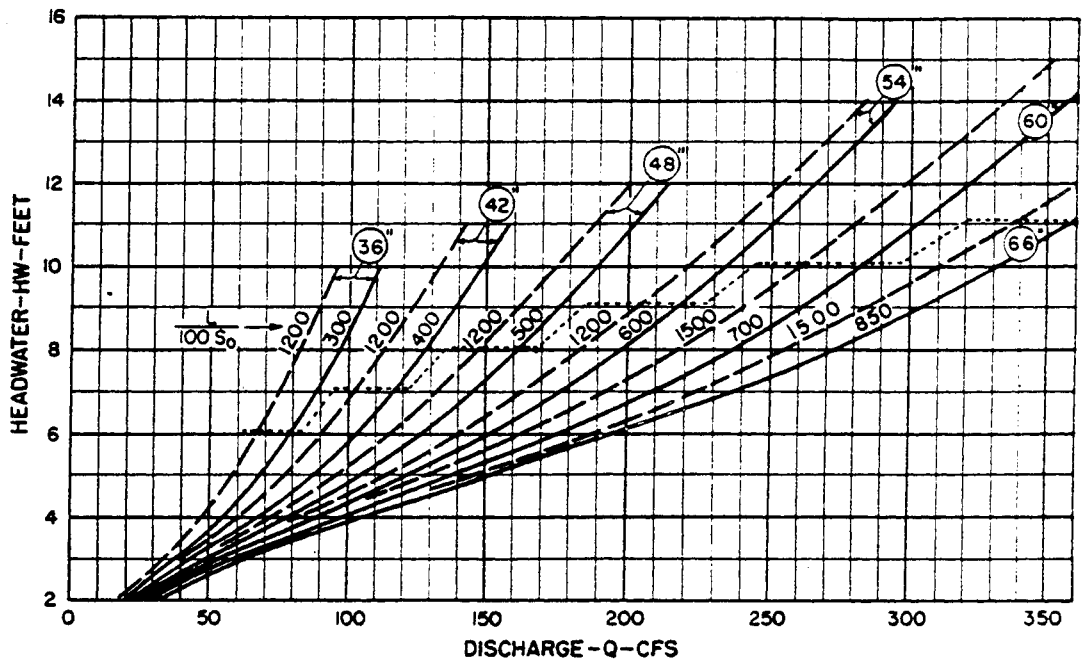
⊗ GIVEN:
950 CFS; AHW = 16 FT.
L = 480 FT.; S_0 = 0.040

⊗ SELECT 108"
HW = 15.0 FT.

BUREAU OF PUBLIC ROADS JAN. 1963

CULVERT CAPACITY
CIRCULAR CONCRETE PIPE
SQUARE-EDGED ENTRANCE
60" TO 180" ◎





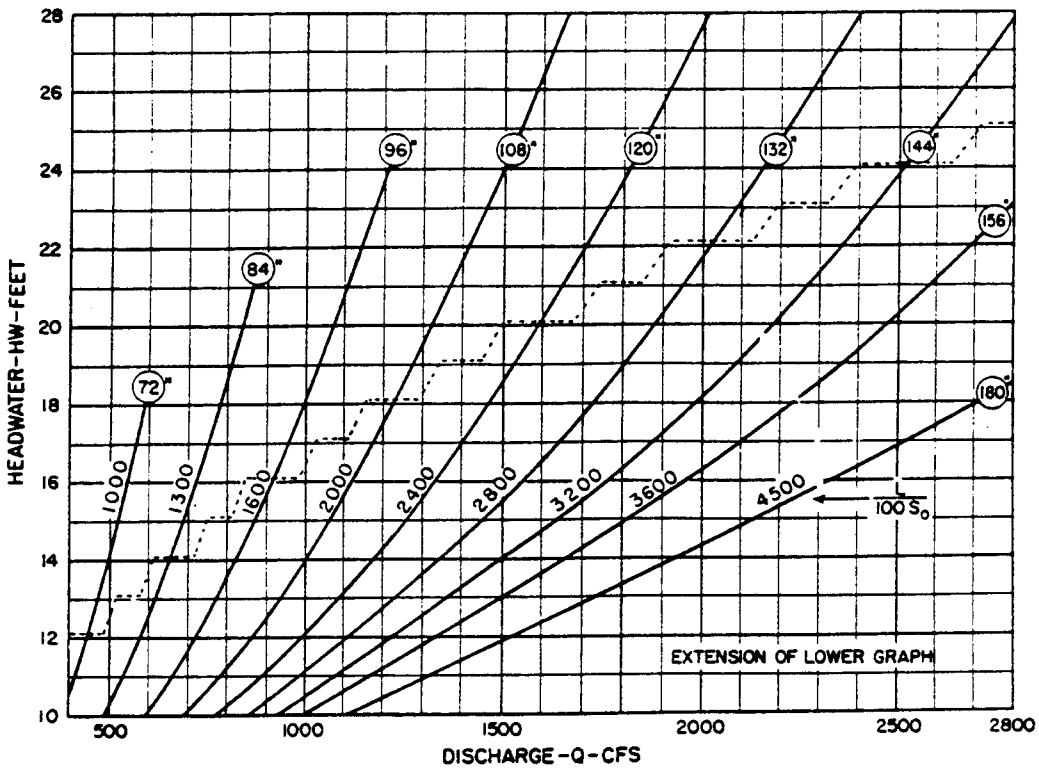
EXAMPLE

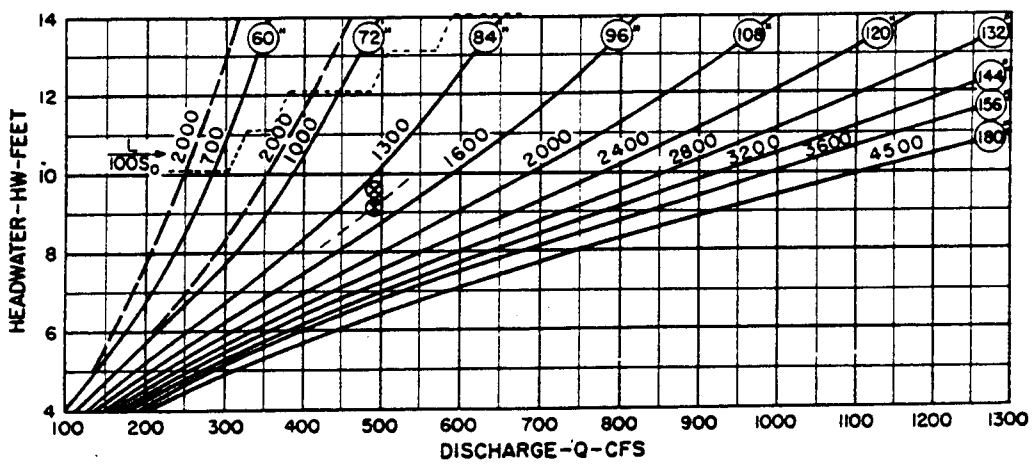
⊗ GIVEN:
43 CFS ; AHW = 5.4 FT.
L = 120 FT.; S_0 = 0.002

⊗ SELECT 30"
HW = 4.7 FT.

BUREAU OF PUBLIC ROADS JAN. 1963

CULVERT CAPACITY
CIRCULAR CONCRETE PIPE
GROOVE-EDGED ENTRANCE
18" TO 66" ○





EXAMPLE

⊗ GIVEN:
490 CFS ; AHW = 9.6 FT.
L = 60 FT. ; S_0 = 0.000

⊗ SELECT 90" ($\frac{L}{D}$ = 8)
HW = 9.2 FT.

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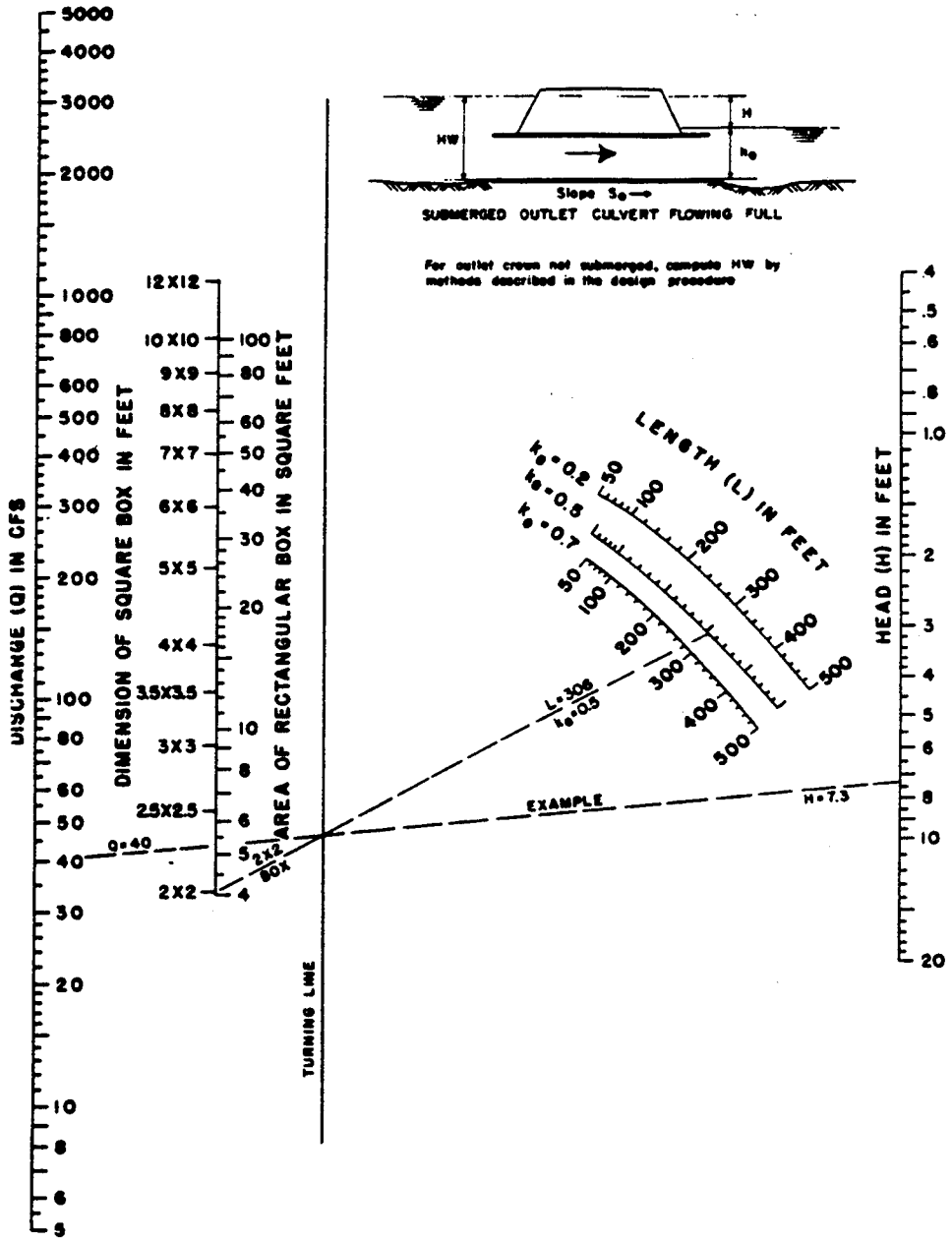
CULVERT CAPACITY
CIRCULAR CONCRETE PIPE
GROOVE-EDGED ENTRANCE
60" TO 180"



HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

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Drainage Criteria Manual

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Figure
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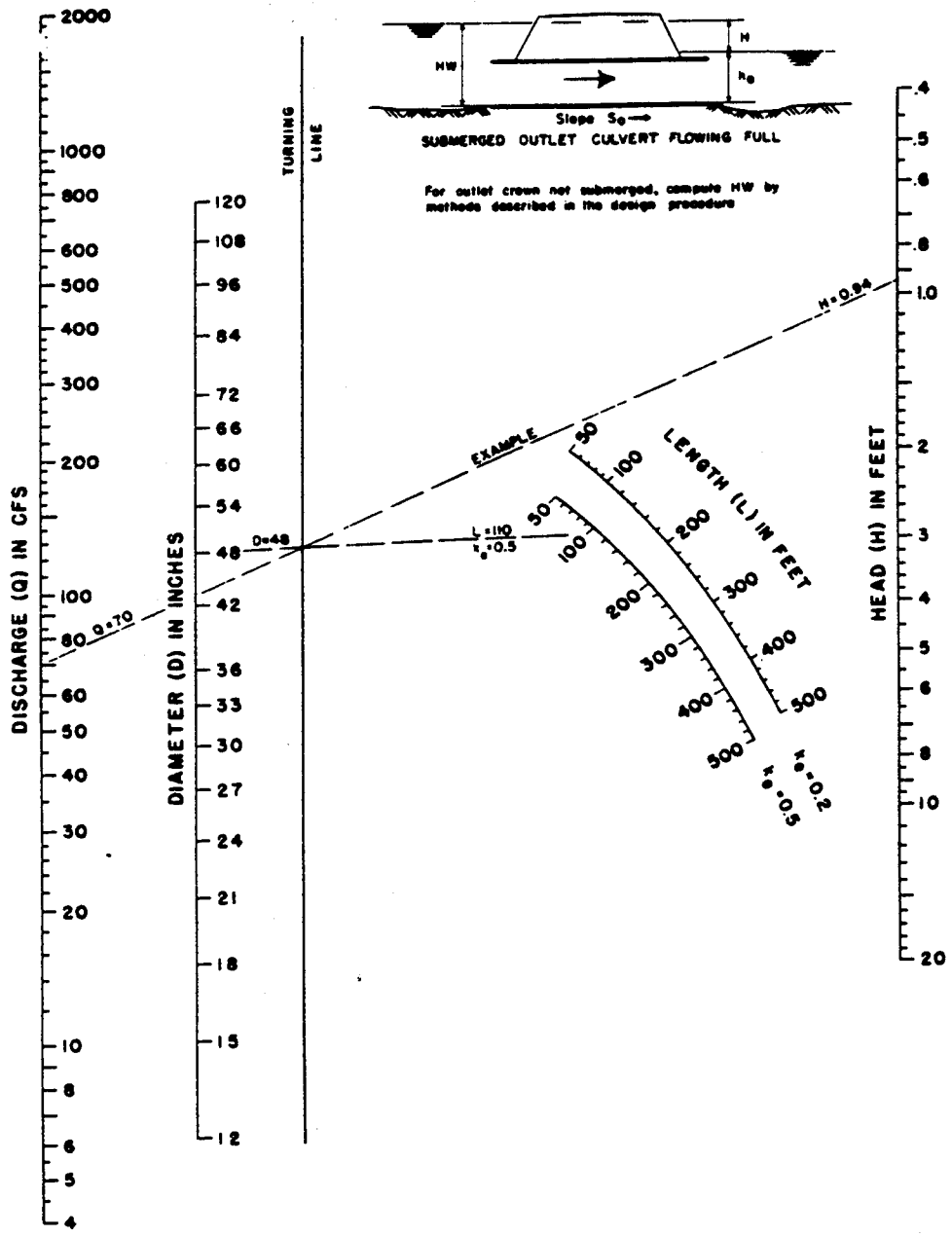
A Centerra Company



HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

HDR Infrastructure, Inc.
A Centerra Company

The City of Colorado Springs / El Paso County
Drainage Criteria Manual

Date
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Figure
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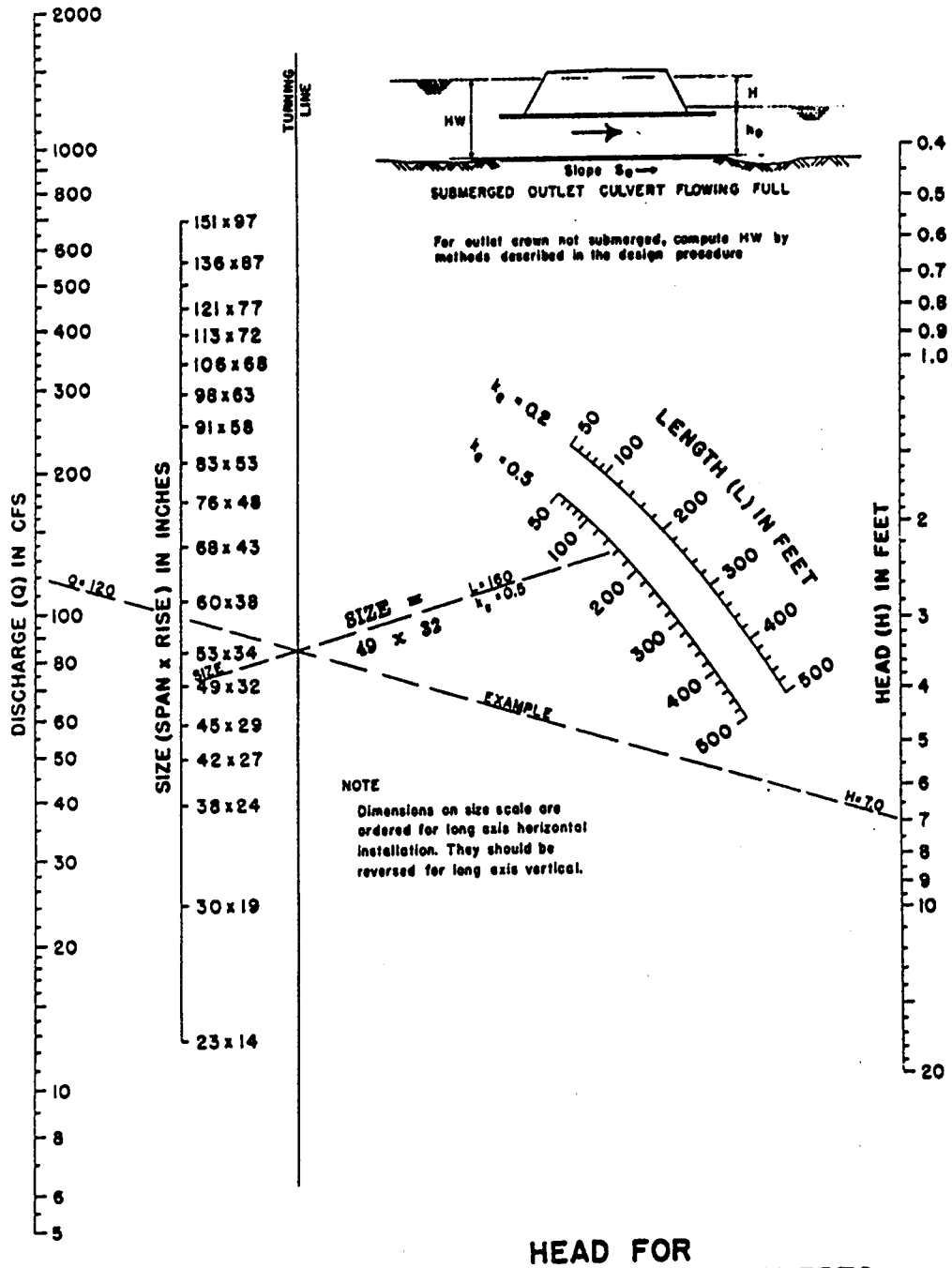
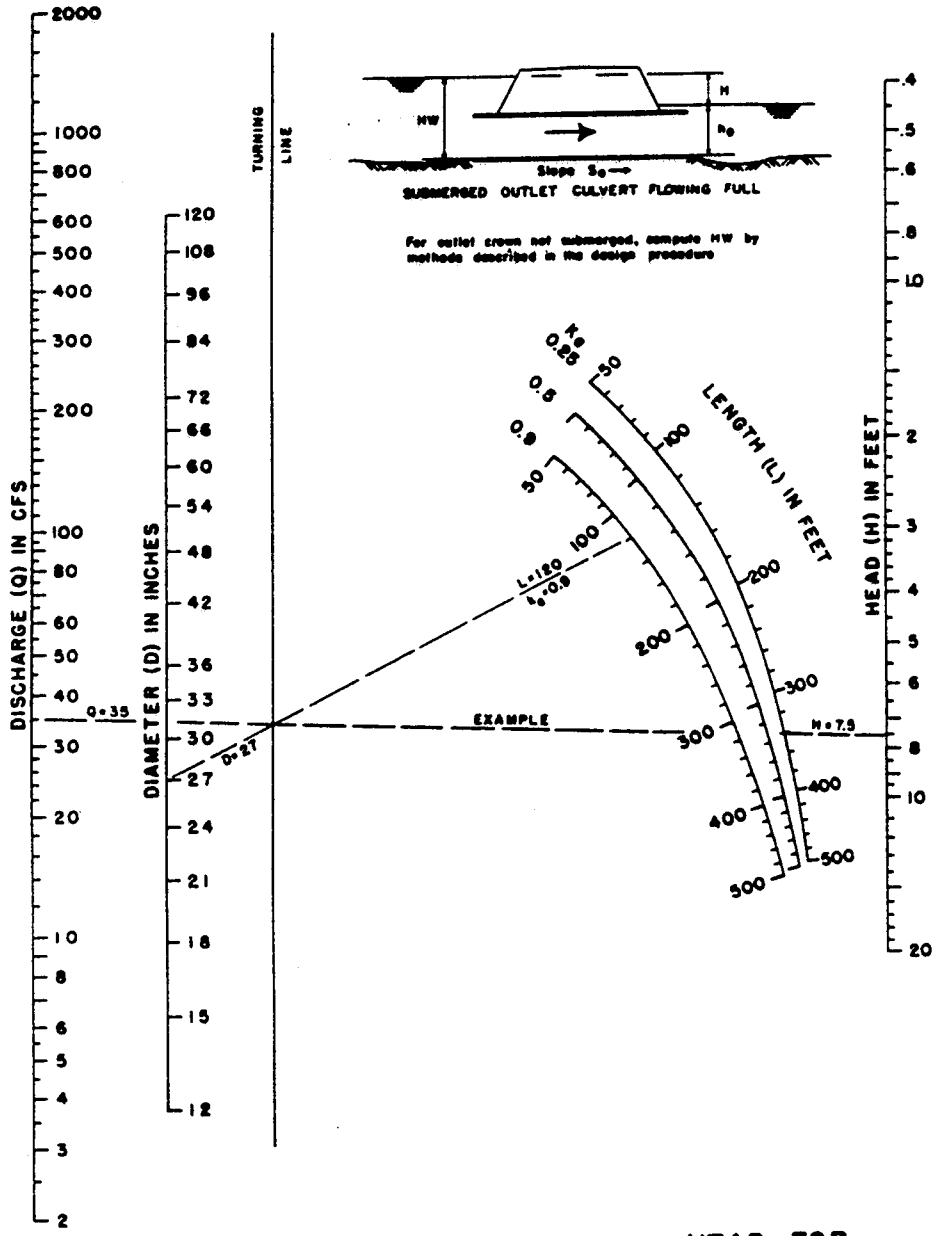


HEAD FOR OVAL CONCRETE PIPE CULVERTS LONG AXIS HORIZONTAL OR VERTICAL FLOWING FULL

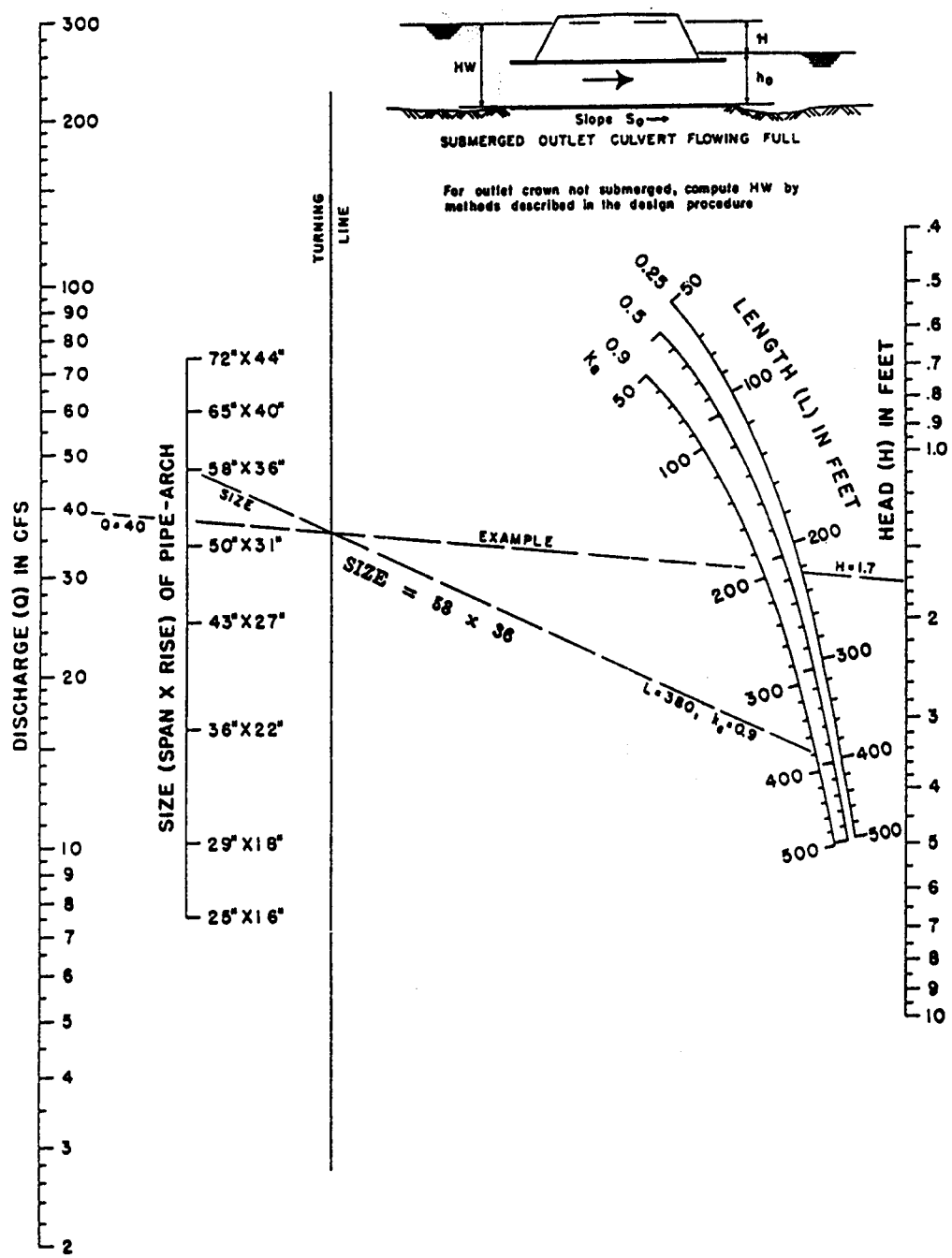
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963



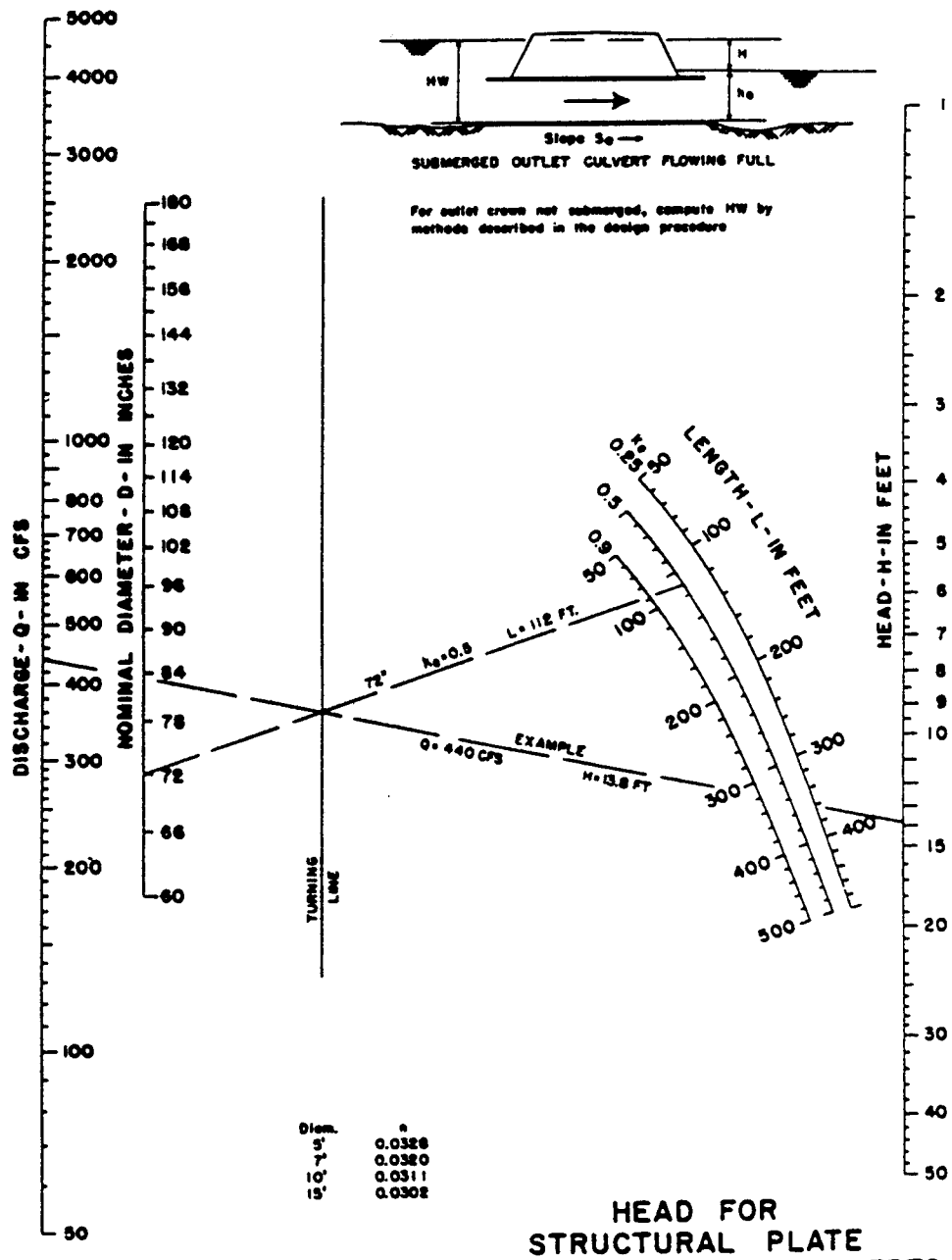
HEAD FOR
STANDARD
C. M. PIPE CULVERTS
FLOWING FULL
n = 0.024

BUREAU OF PUBLIC ROADS JAN. 1963



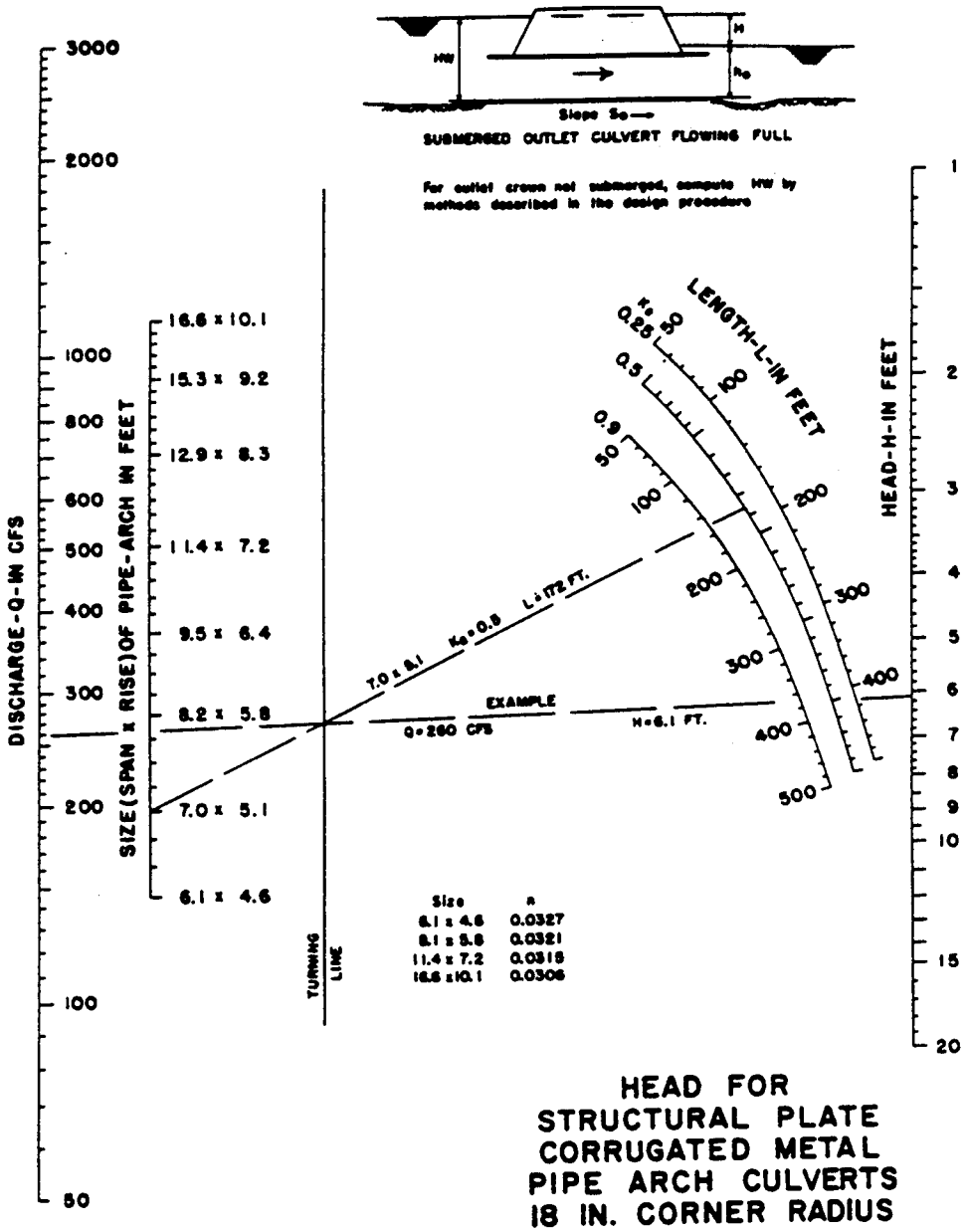
HEAD FOR STANDARD C. M. PIPE-ARCH CULVERTS FLOWING FULL n=0.024

The City of Colorado Springs / El Paso County Drainage Criteria Manual	Date 9-30-90
	Figure 9-25

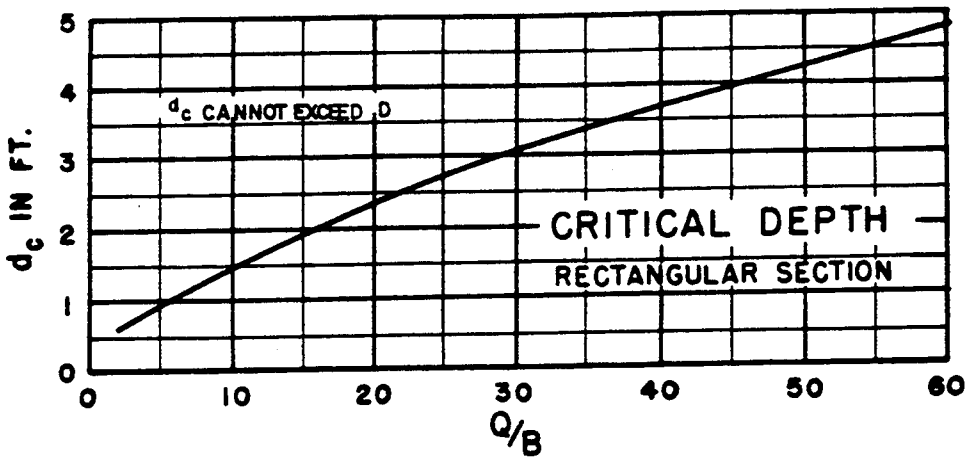


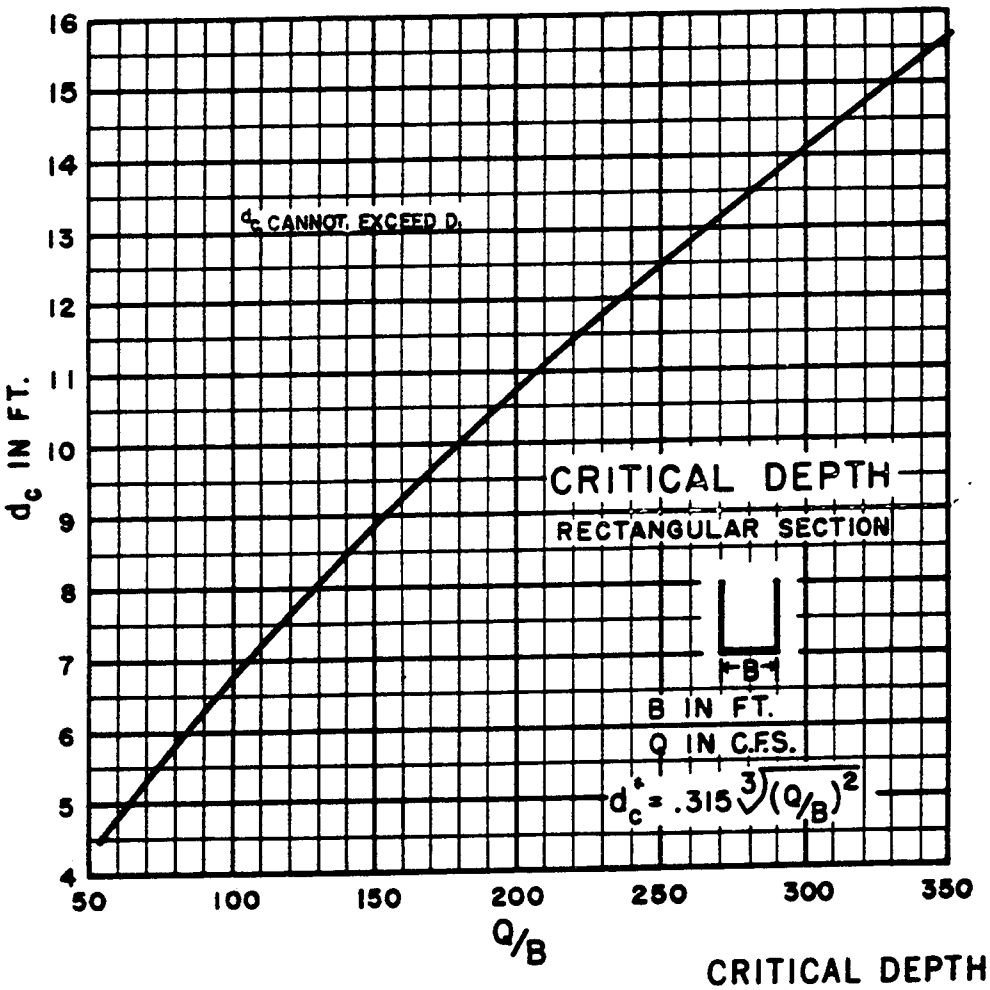
HEAD FOR
STRUCTURAL PLATE
CORR. METAL PIPE CULVERTS
FLOWING FULL
n = 0.0328 TO 0.0302

BUREAU OF PUBLIC ROADS JAN. 1963



HEAD FOR
STRUCTURAL PLATE
CORRUGATED METAL
PIPE ARCH CULVERTS
18 IN. CORNER RADIUS
FLOWING FULL
n = 0.0327 TO 0.0306





BUREAU OF PUBLIC ROADS JAN. 1963

CRITICAL DEPTH
RECTANGULAR SECTION

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Drainage Criteria Manual

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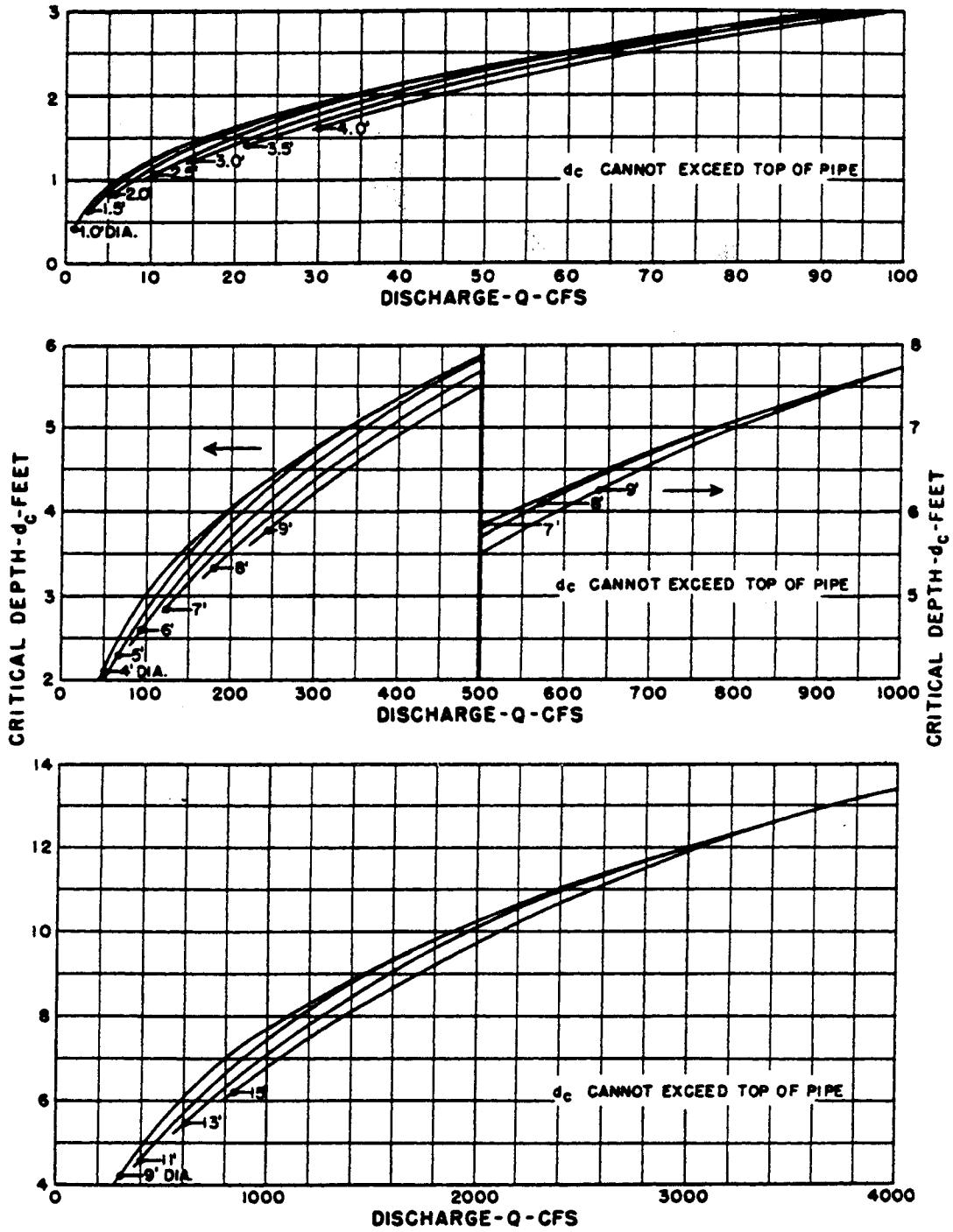
Figure
9-28



BUREAU OF PUBLIC ROADS

JAN. 1964

CRITICAL DEPTH
CIRCULAR PIPE

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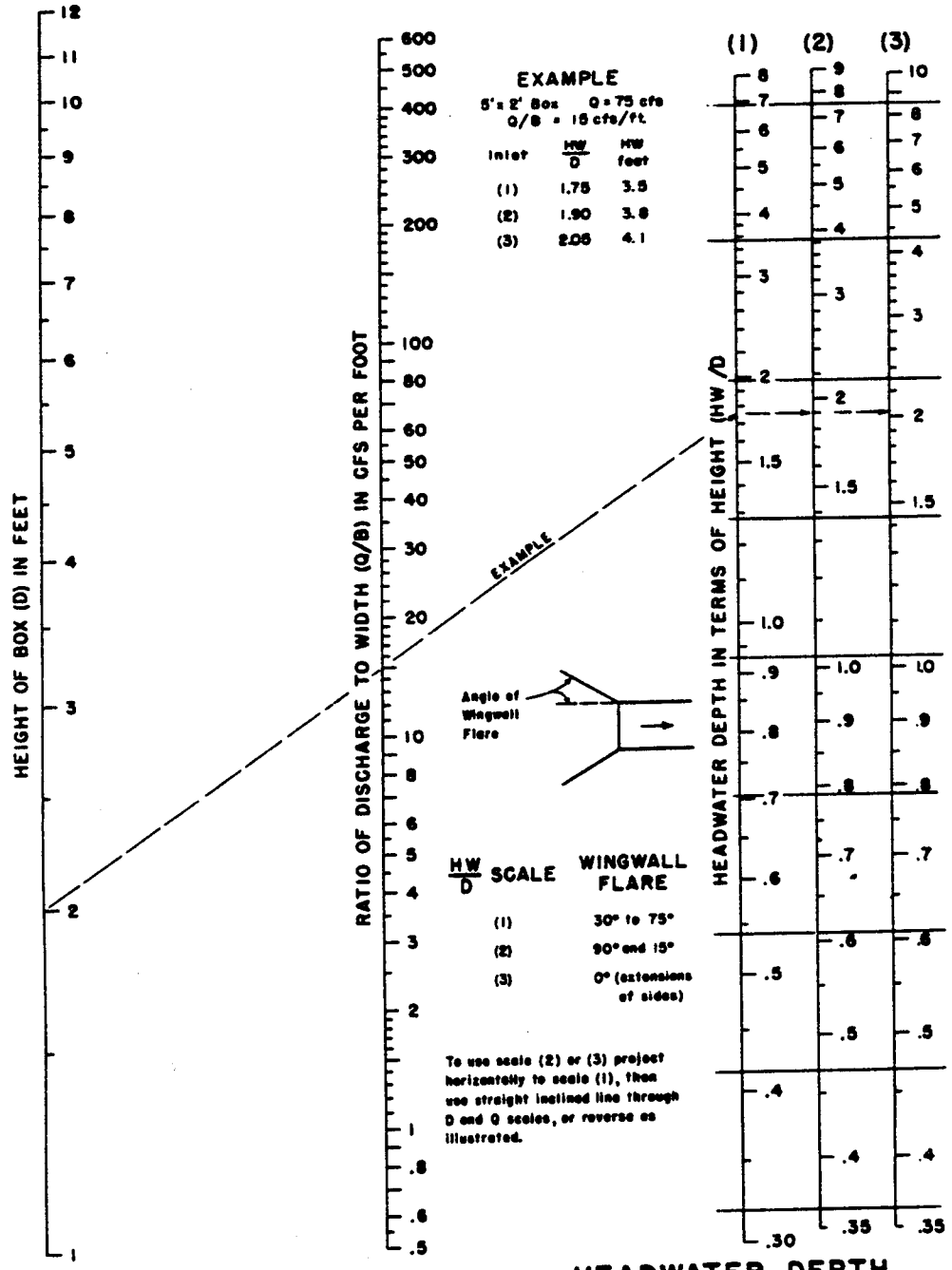
Figure
9-29



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

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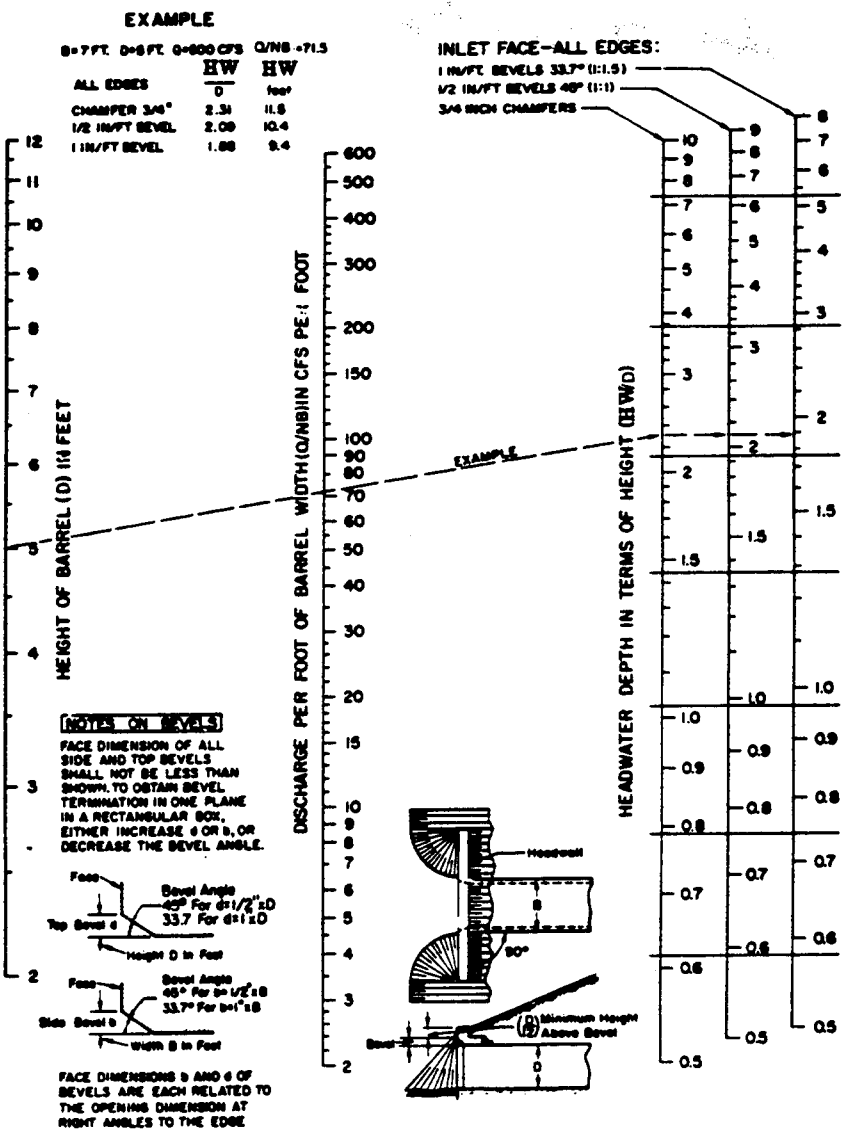
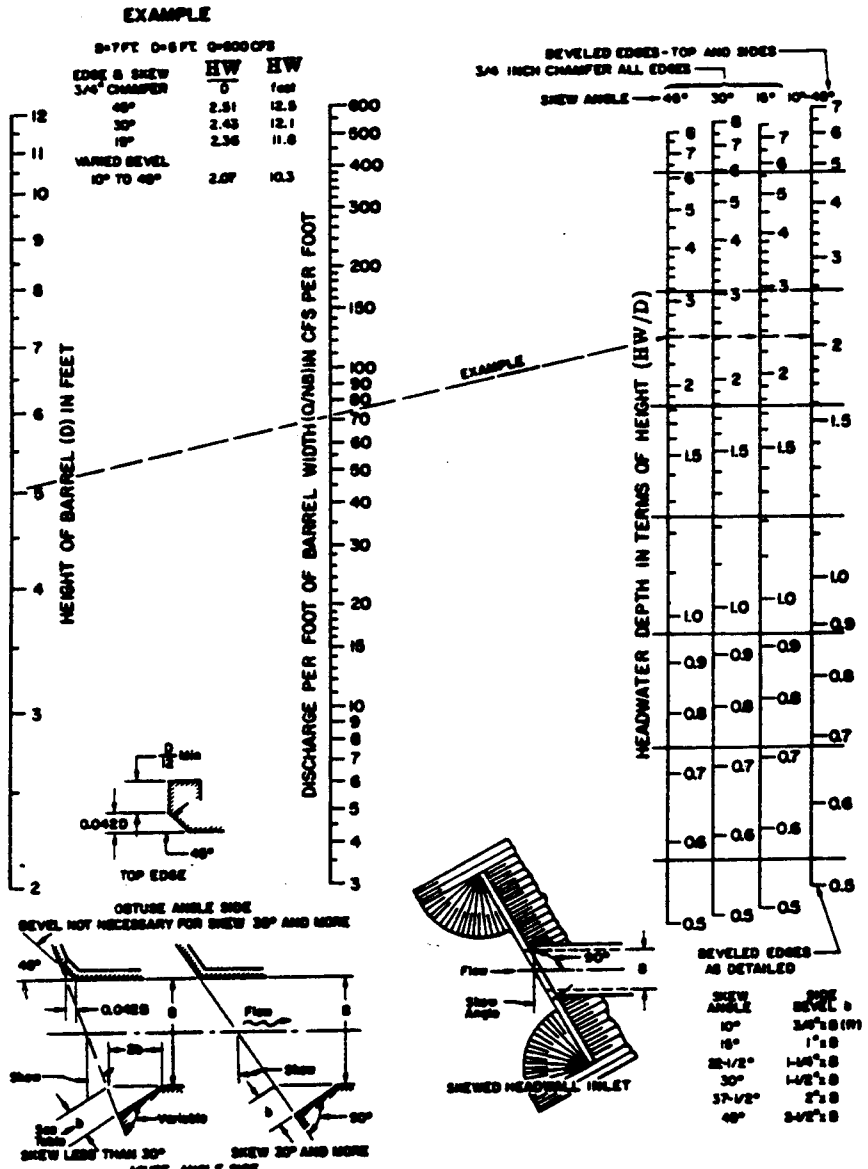
Figure
9-30



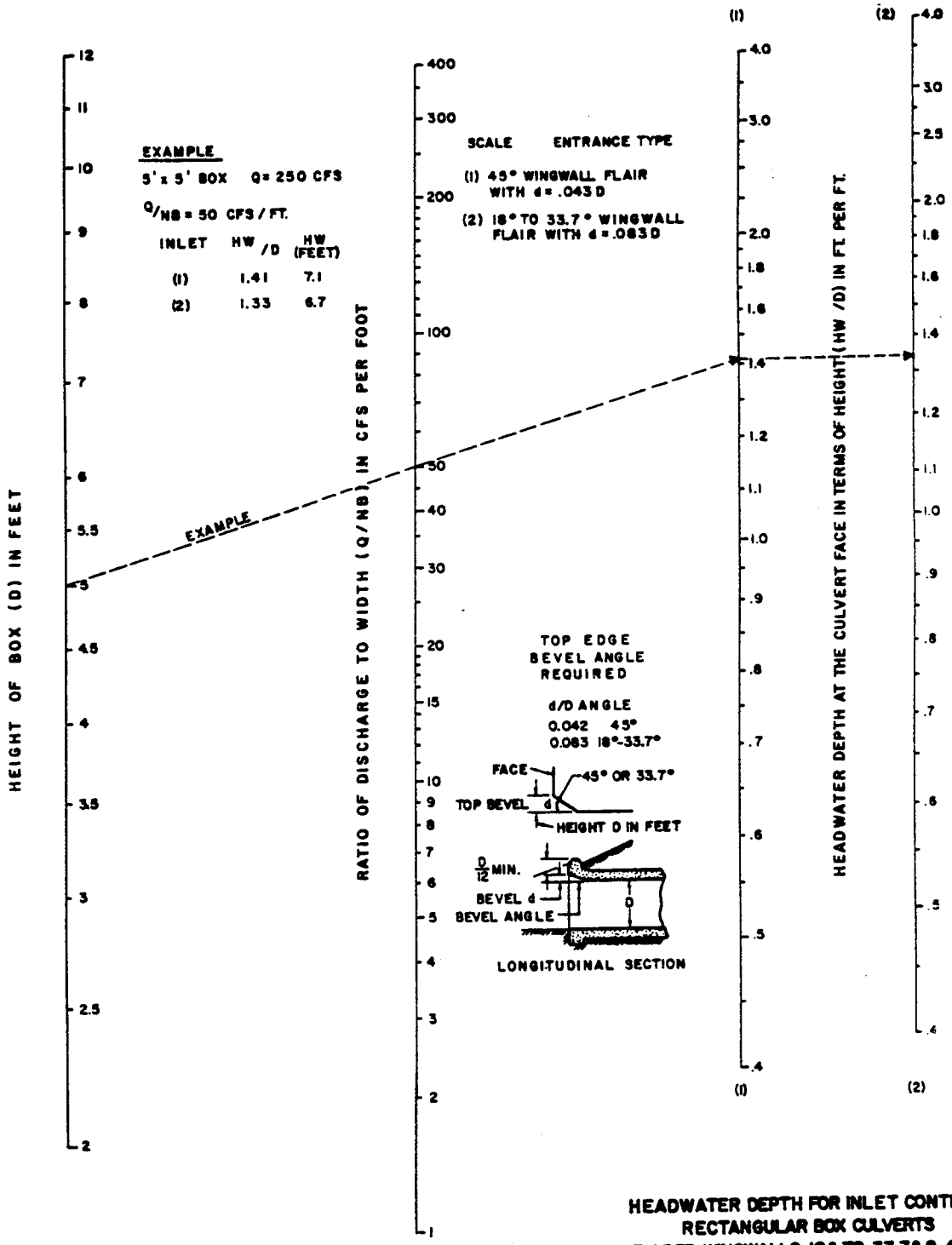
HEADWATER DEPTH FOR INLET CONTROL RECTANGULAR BOX CULVERTS 90° HEADWALL CHAMFERED OR BEVELED INLET EDGES

FEDERAL HIGHWAY ADMINISTRATION
MAY 1973



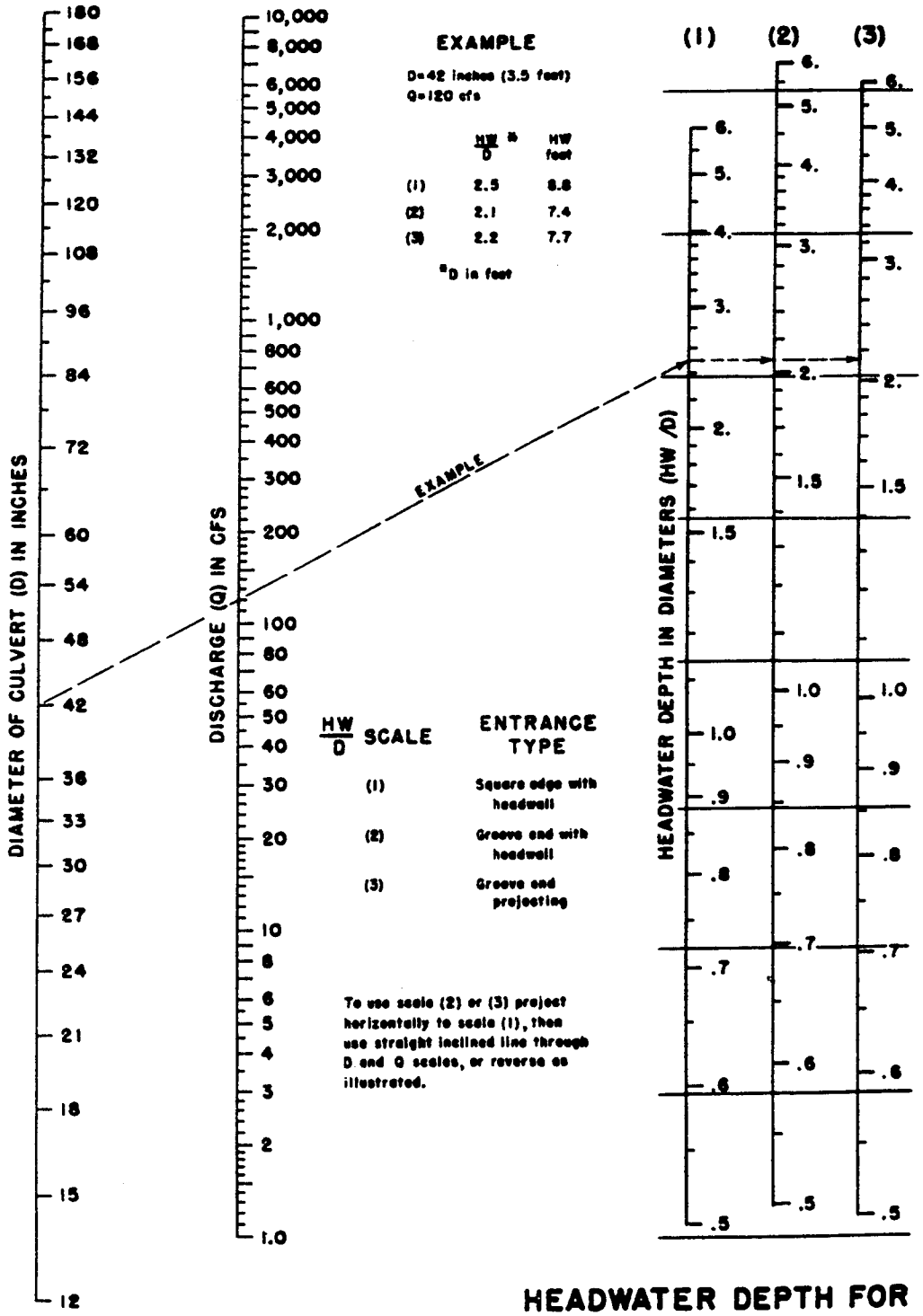
FEDERAL HIGHWAY ADMINISTRATION
MAY 1973

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



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

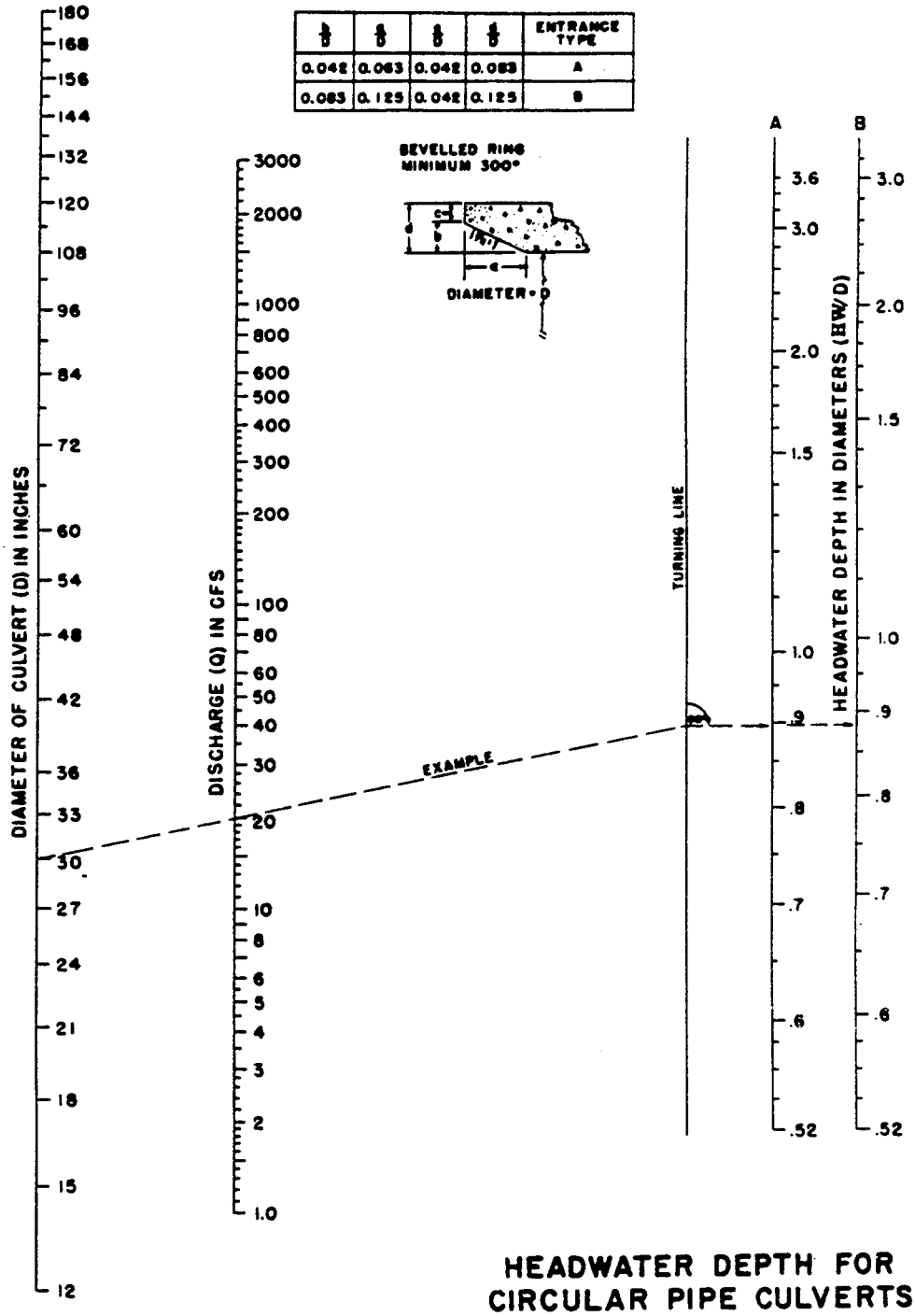
BUREAU OF PUBLIC ROADS JAN. 1963



HEADWATER SCALES 2&3
REVISED MAY 1964

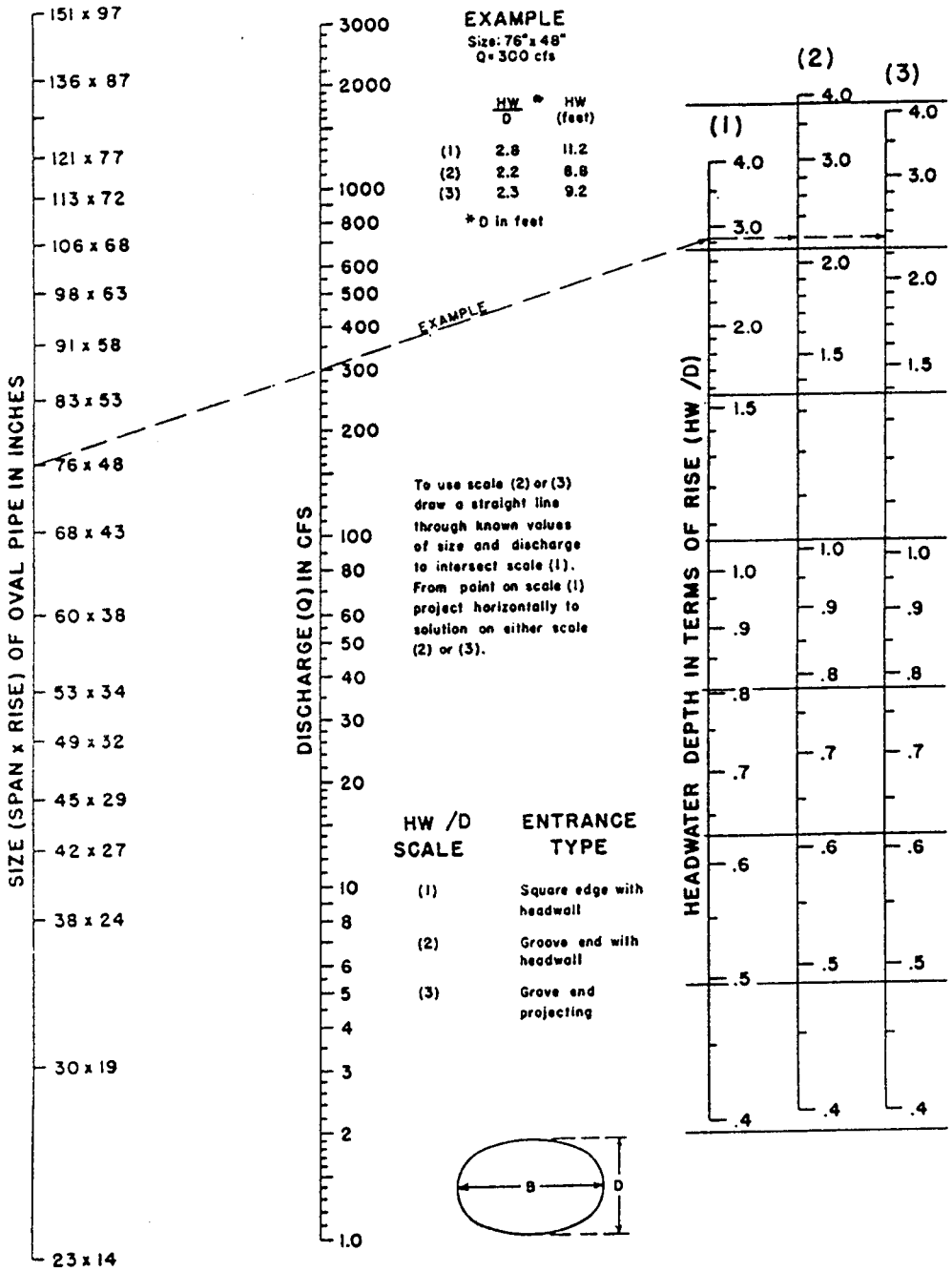
BUREAU OF PUBLIC ROADS JAN. 1963

HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL



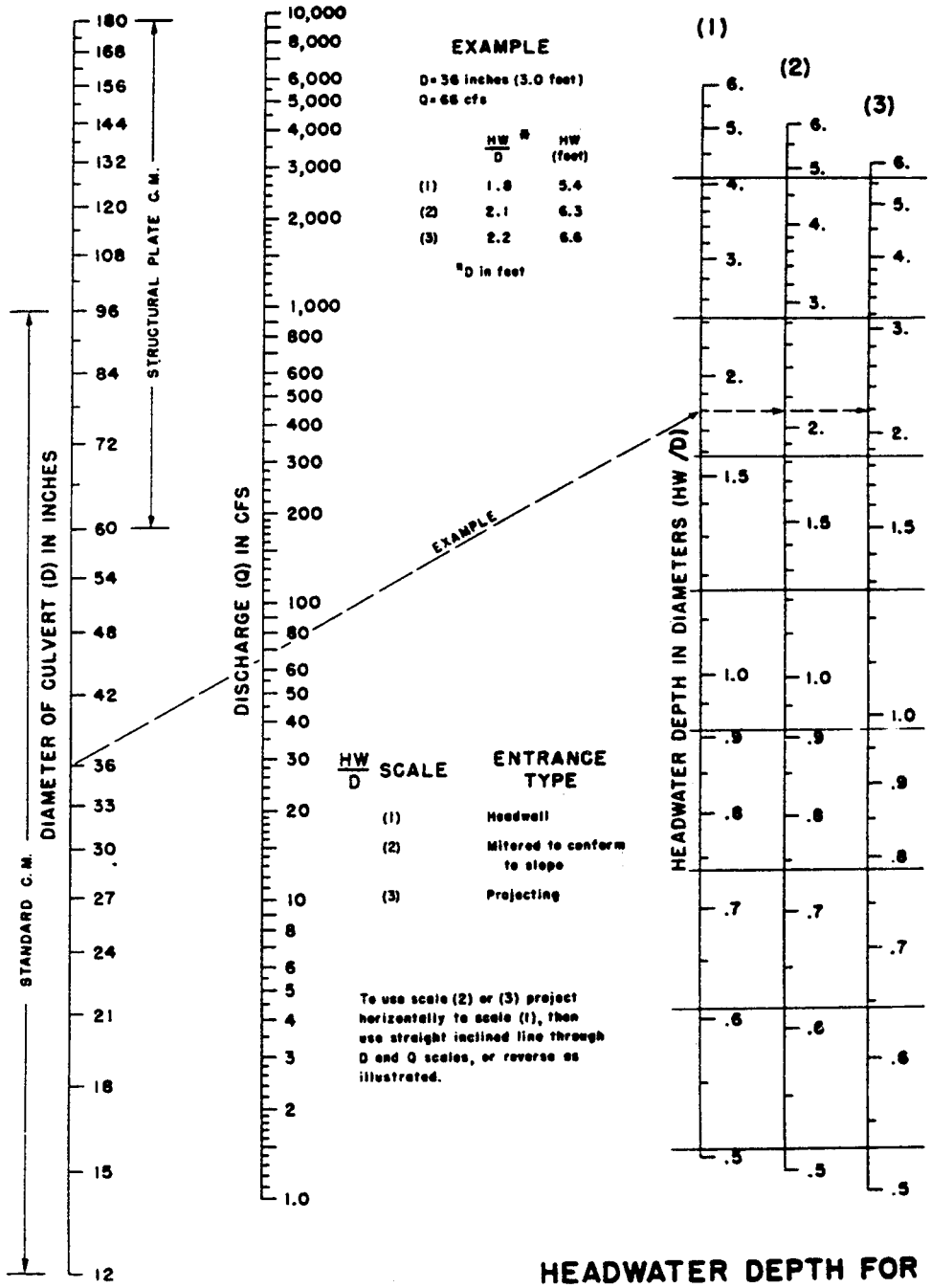
HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL

FEDERAL HIGHWAY ADMINISTRATION
MAY 1973



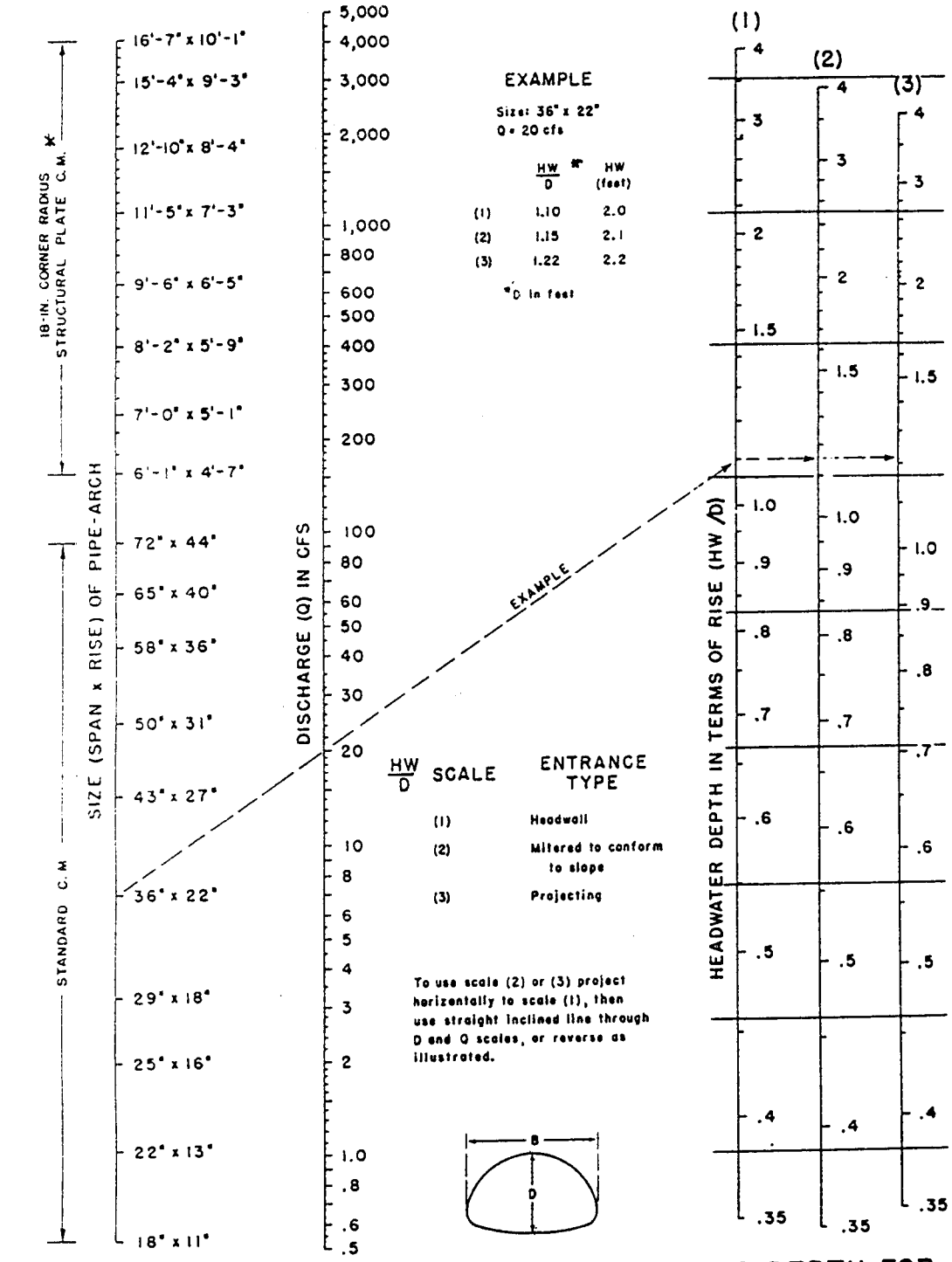
HEADWATER DEPTH FOR OVAL CONCRETE PIPE CULVERTS LONG AXIS HORIZONTAL WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963



BUREAU OF PUBLIC ROADS JAN. 1963

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



*ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATOR'S CATALOG

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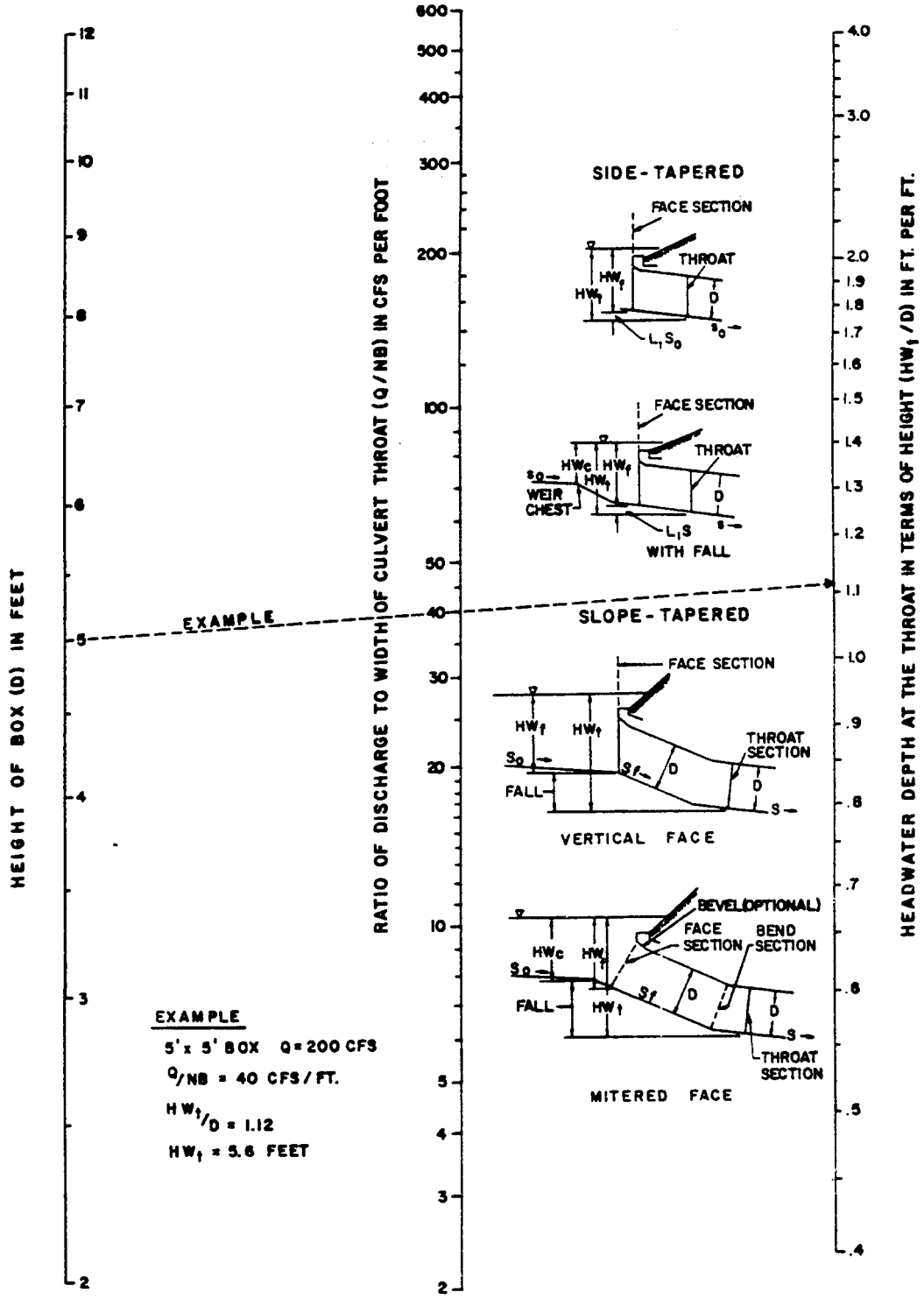
HEADWATER DEPTH FOR C. M. PIPE-ARCH CULVERTS WITH INLET CONTROL



REFERENCE :

Federal Highway Administration, Hydraulic Design of Highway Culverts ; Hydraulic Design Series No. 5 1985

THROAT CONTROL FOR BOX CULVERTS WITH TAPERED INLETS

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Date
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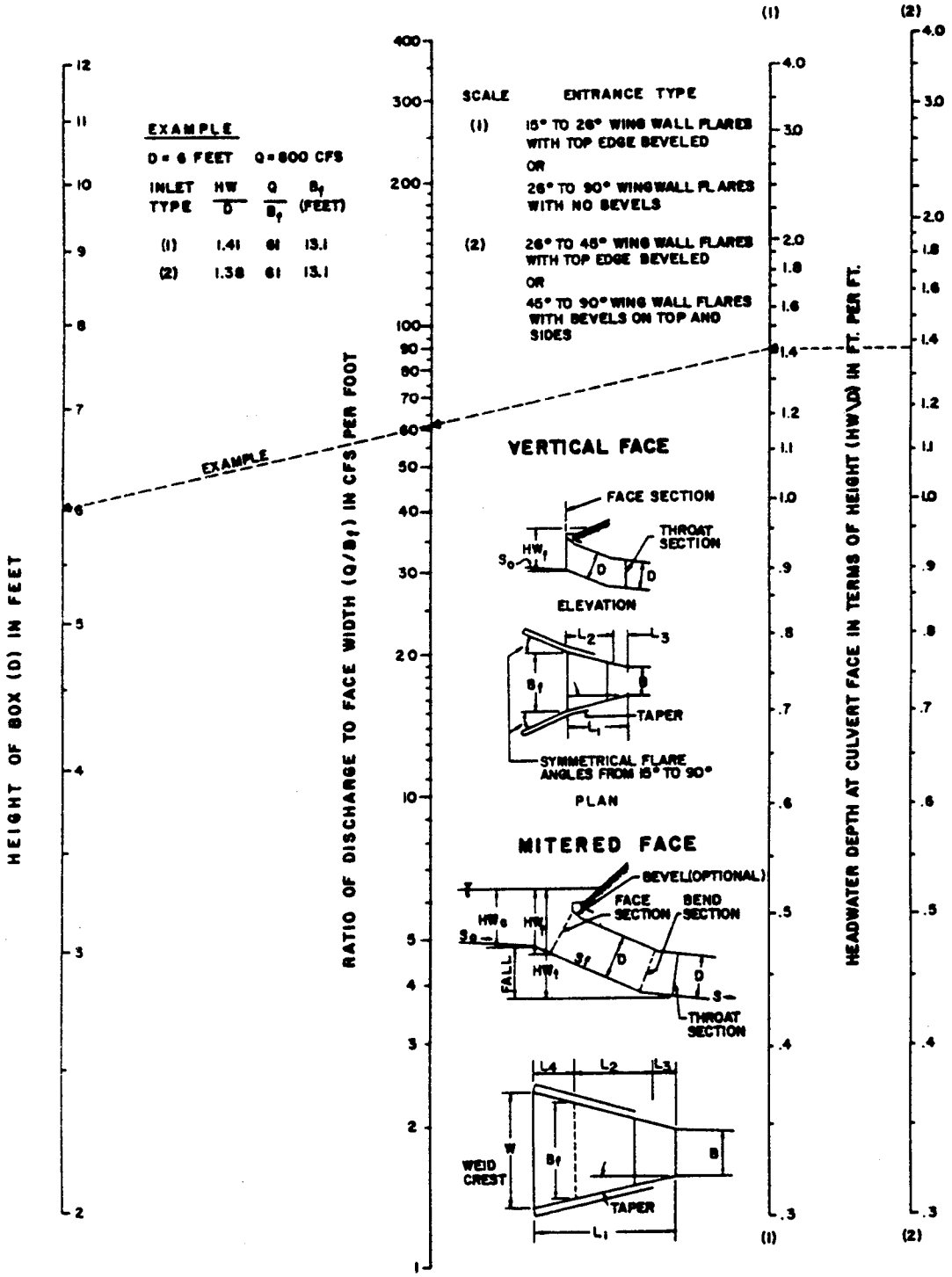
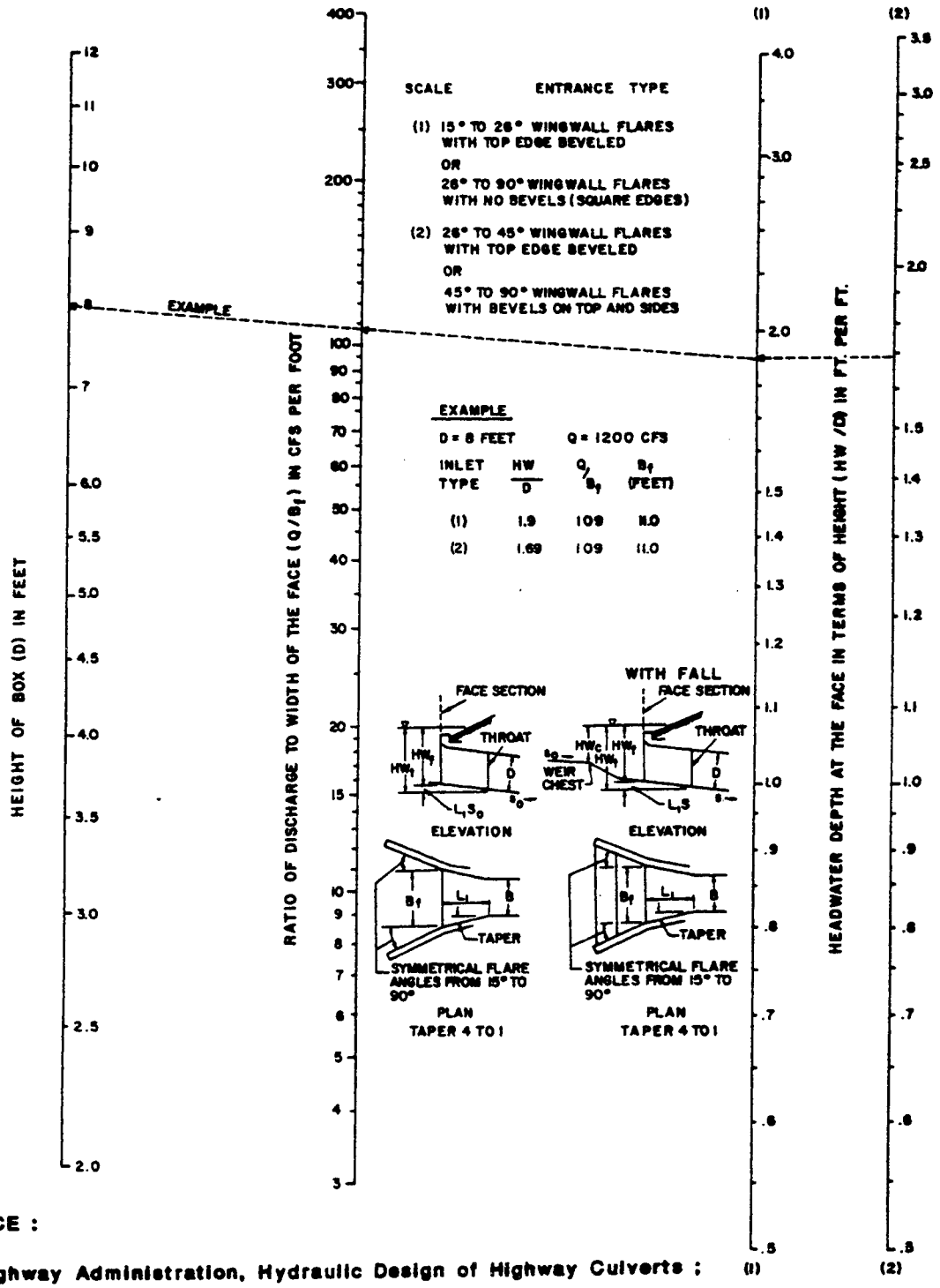
Figure
9-39



REFERENCE :

Federal Highway Administration,
Hydraulic Design of Highway Culverts ;
Hydraulic Design Series No. 5 1985

FACE CONTROL FOR BOX CULVERTS WITH SLOPE TAPERED INLETS



REFERENCE :

Federal Highway Administration, Hydraulic Design of Highway Culverts ; Hydraulic Design Series No. 5 1985

FACE CONTROL FOR BOX CULVERTS WITH SIDE TAPERED INLETS

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OCT. 1987

Figure
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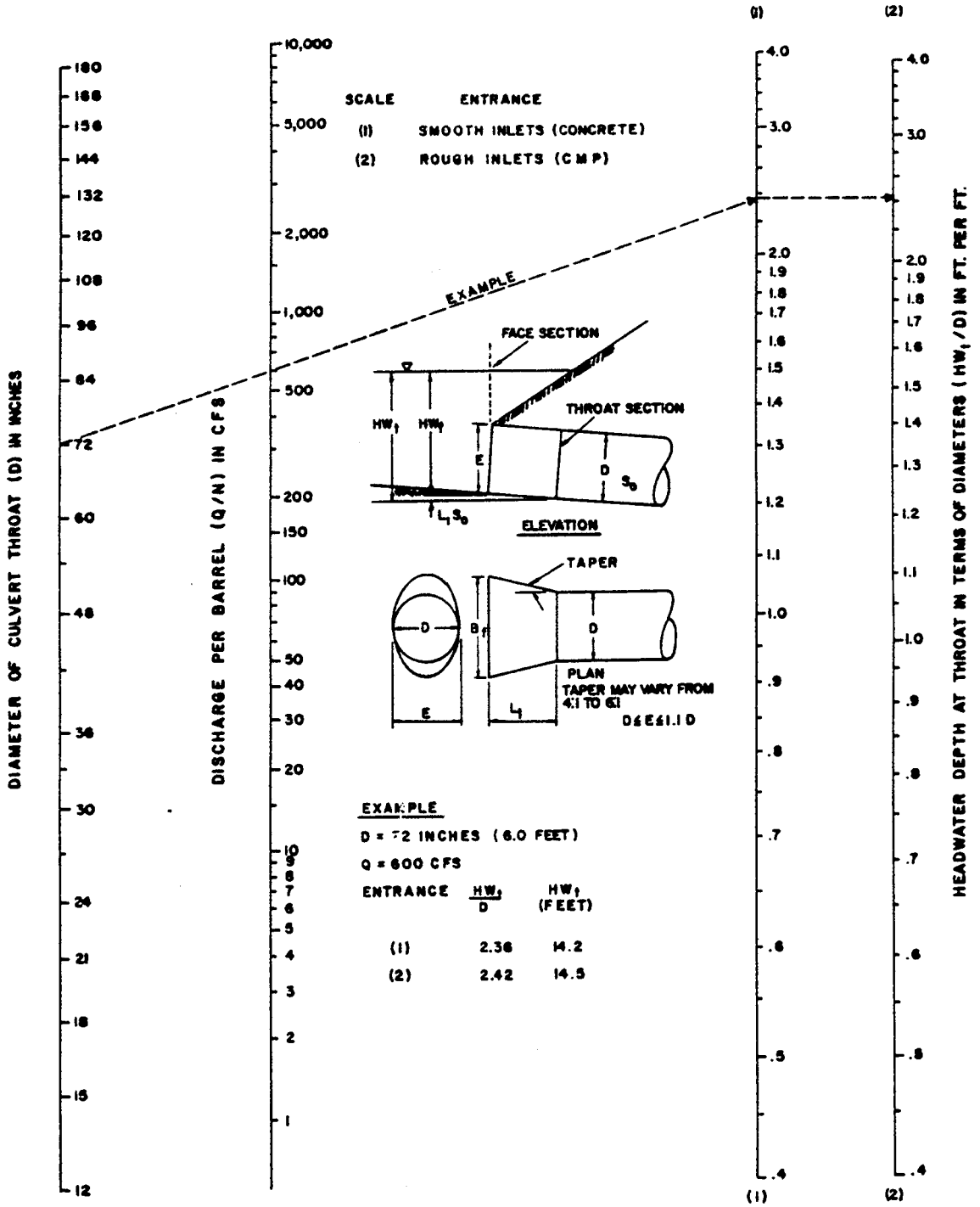

REFERENCE :

Federal Highway Administration,
Hydraulic Design of Highway Culverts ;
Hydraulic Design Series No. 5 1985

THROAT CONTROL
FOR SIDE-TAPERED INLETS TO PIPE CULVERT
(CIRCULAR SECTION ONLY)

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Figure
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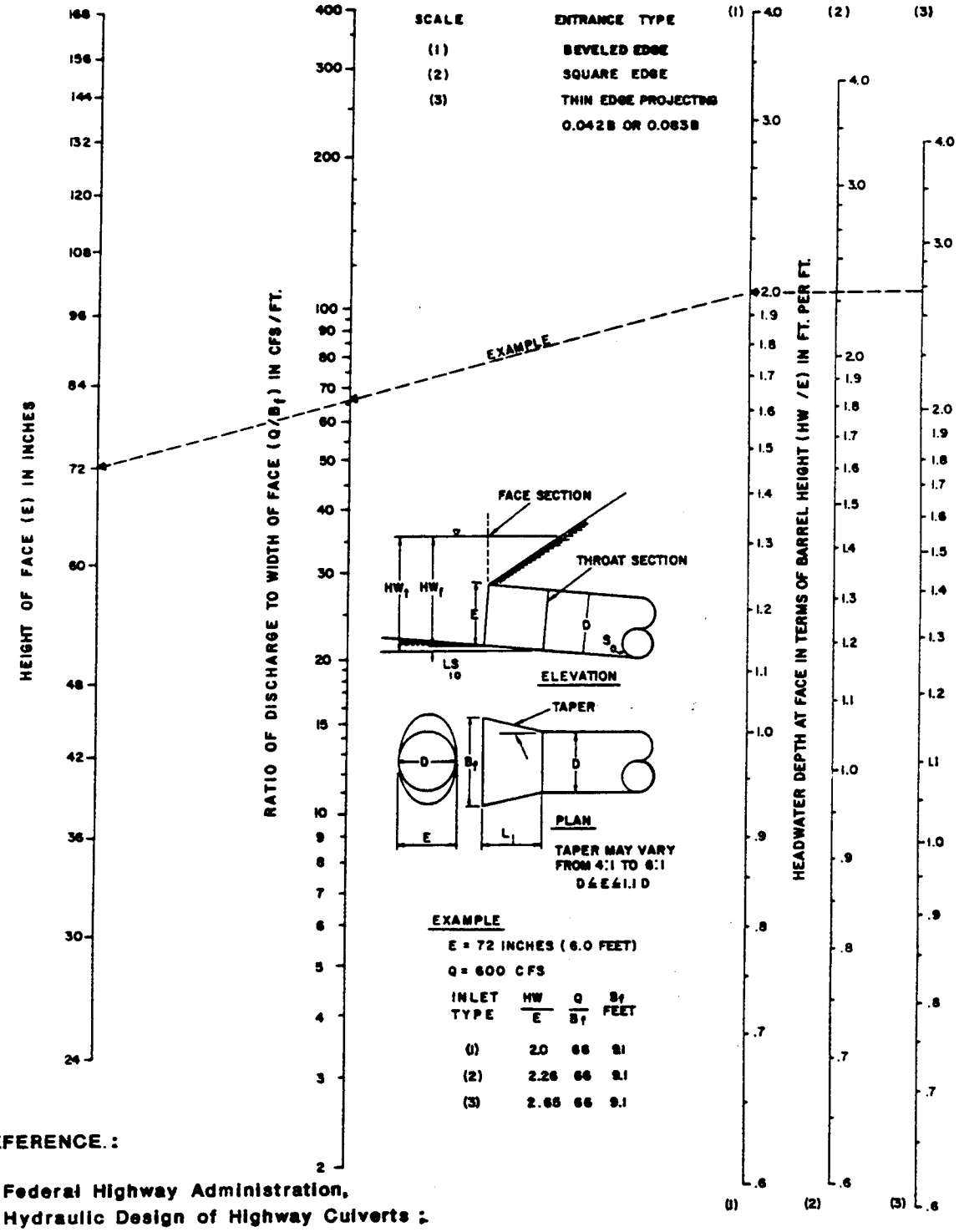


REFERENCE.:

Federal Highway Administration,
Hydraulic Design of Highway Culverts ;
Hydraulic Design Series No. 5 1985

FACE CONTROL FOR SIDE-TAPERED INLETS TO PIPE CULVERTS (NON-RECTANGULAR SECTIONS ONLY)

PROJECT : _____

STATION : _____

SHEET _____ OF _____

CULVERT DESIGN FORM

DESIGNER / DATE : _____ / _____

REVIEWER / DATE : _____ / _____

HYDROLOGICAL DATA

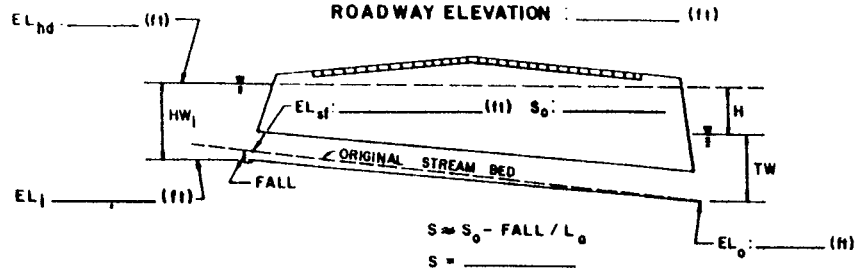
SEE ADD'L SHTS.

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

DESIGN FLOWS/TAILWATER

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

ROADWAY ELEVATION : _____ (ft)



$S \approx S_o - FALL / L_o$

$S =$ _____

$L_o =$ _____

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

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

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

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

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

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

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

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

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

SUBSCRIPT DEFINITIONS :

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

COMMENTS / DISCUSSION :

CULVERT BARREL SELECTED :

SIZE : _____

SHAPE : _____

MATERIAL : _____

ENTRANCE : _____

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Figure
9-44

REFERENCE : Federal Highway Administration, Hydraulic Design of Highway Culverts ; Hydraulic Design Series No. 5 1985

PROJECT: _____

STATION: _____

SHEET _____ OF _____

TAPERED INLET DESIGN FORM

DESIGNER / DATE: _____ / _____

REVIEWER / DATE: _____ / _____

DESIGN DATA:

Q _____ = _____ cfs ; EL_{hi} _____ ft

EL. THROAT INVERT _____ ft

EL. STREAM BED AT FACE _____ ft

FALL _____ ft TAPER _____ : 1 (4 : 1 TO 6 : 1)

STREAM SLOPE, S_o = _____ ft/ft

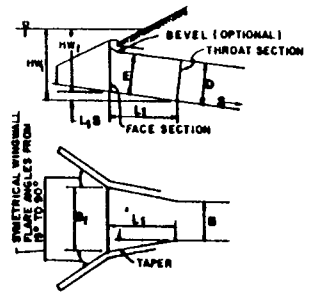
SLOPE OF BARREL, S = _____ ft/ft

S_f _____ : 1 (2 : 1 TO 3 : 1)

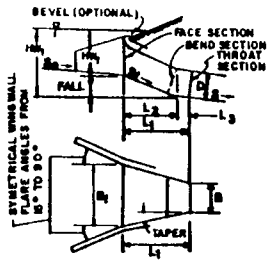
BARREL SHAPE AND MATERIAL: _____

N = _____, B = _____, D = _____

INLET EDGE DESCRIPTION _____



SIDE - TAPERED



SLOPE - TAPERED

COMMENTS

Q (cfs)	EL_{hi}	EL. THROAT INVERT	EL. FACE INVERT (1)	HW_f (2)	HW_f / E (3)	Q / B_f (4)	MIN. B_f (5)	SELECTED B_f	SLOPE - TAPERED ONLY: MIN. L_3 (6)	L_2 (7)	CHECK L_2 (8)	ADJ. L_3 (9)	ADJ. TAPER (10)	L_1 (11)	SIDE-TAPERED W/ FALL: EL. CREST INV.	HW_c (12)	MIN. W (13)

(1) SIDE - TAPERED: EL. FACE INVERT = EL. THROAT INVERT + 1 ft (APPROX.)
SLOPE - TAPERED: EL. FACE INVERT = EL. STREAM BED AT FACE

(2) $HW_f = EL_{hi}$ - EL. FACE INVERT

(3) $1.1D \geq E \geq D$

(4) FROM DESIGN CHARTS

(5) MIN. $B_f = Q/(Q/B_f)$

(6) MIN. $L_3 = 0.5NB$

(7) L_2 = (EL. FACE INVERT - EL. THROAT INVERT) S_f

(8) CHECK $L_2 = \left[\frac{B_f - NB}{2}\right] \cdot TAPER - L_3$

(9) IF (6)>(7), ADJ. $L_3 = \left[\frac{B_f - NB}{2}\right] \cdot TAPER - L_2$

(10) IF (7)>(6), ADJ. TAPER = $(L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(11) SIDE - TAPERED: $L = \left[\frac{B_f - NB}{2}\right] \cdot TAPER$

SLOPE - TAPERED: $L_1 = L_2 + L_3$

(12) $HW_c = EL_{hi}$ - EL. CREST INVERT

(13) MIN. $W = 0.35Q / HW_c^{1.5}$

SELECTED DESIGN

B_f _____

L_1 _____

L_2 _____

L_3 _____

BEVELS ANGLE ____°

b = ____ in ; d = ____ in

TAPER _____ : 1

S_f = _____ : 1

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Figure
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REFERENCE: Federal Highway Administration, Hydraulic Design of Highway Culverts; Hydraulic Design Series No. 5 1985

9-73

PROJECT : _____

STATION : _____

SHEET _____ OF _____

MITERED INLET DESIGN FORM

DESIGNER / DATE : _____ / _____

REVIEWER / DATE : _____ / _____

DESIGN DATA : N _____ ; B _____ ; D _____

Q _____ = _____ cfs ; EL_{hi} _____ ft

EL. THROAT INVERT _____ ft EL. STREAM BED AT CREST _____ ft

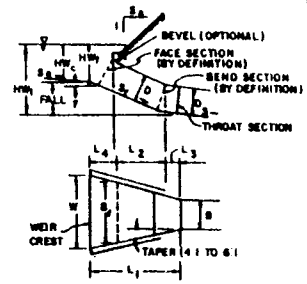
FALL _____ ft ; TAPER _____ : (4:1 TO 6:1)

STREAM SLOPE , S_o _____ ft/ft ; BARREL SLOPE , S = _____ ft/ft

SLOPE OF THE EMBANKMENT S_e = _____ : 1 ; S_f _____ : 1 (2:1 TO 3:1)

BARREL SHAPE AND MATERIAL : _____

INLET EDGE DESCRIPTION : _____



SLOPE - TAPERED INLET/MITERED FACE

COMMENTS

Q (cfs)	EL_{hi}	EL. THROAT INVERT	y (1)	EL. FACE INVERT (2)	HW_f (3)	HW_f / E (4)	Q / B_f (5)	MIN. B_f (6)	SELECTED B_f	MIN. L_3 (7)	L_4 (8)	L_2 (9)	CHECK L_2 (10)	ADJ. L_3 (11)	ADJ. TAPER (12)	L_1 (13)	EL. CREST INV.	HW_c (14)	MIN. W (15)	W (16)

(1) $y = \left[\frac{(S_e - S_f) - 1}{(S_e + S_f)(S_f^2 + 1)^{0.5}}\right] \cdot D$

(2) EL. FACE INVERT = EL. STREAM BED AT CREST - y

(3) HW_f = EL_{hi} - EL. FACE INVERT

(4) 1.1 D ≥ E ≥ D

(5) FROM DESIGN CHARTS

(6) MIN. B_f = Q / (Q / B_f)

(7) MIN. L_3 = 0.5 NB

(8) L_4 = S_f y + D / S_f

(9) L_2 = (EL. CREST INVERT - EL. THROAT INVERT) S_f - L_4
*** IF L_2 IS NEGATIVE DO NOT USE THIS INLET

(10) CHECK $L_2 = \left[\frac{B_f - NB}{2}\right]$ TAPER - L_3

(11) IF (10)>(9), ADJ. $L_3 = \left[\frac{B_f - NB}{2}\right]$ TAPER - L_2

(12) IF (9)>(10), ADJ. TAPER = (L_2 + L_3) / $\left[\frac{B - NB}{2}\right]$

(13) L_1 = L_2 + L_3 + L_4

(14) HW_c = EL_{hi} - EL. CREST INVERT

(15) MIN. W = 0.35 Q / (HW_c)$^{1.5}$

(16) W = NB + 2 $\left[\frac{L_1}{TAPER}\right]$
IF W<MIN. W, ADJUST TAPER

SELECTED DESIGN

B_f _____

L_1 _____

L_2 _____

L_3 _____

L_4 _____

BEVELS ANGLE _____ °

b = _____ in; d = _____ in

TAPER _____ : 1

S_f _____ : 1

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Figure
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REFERENCE : Federal Highway Administration, Hydraulic Design of Highway Culverts ; Hydraulic Design Series No. 5 1985

9.7 List of Symbols

Symbol	Units	Description
A_b	sq ft	Area of bend section of slope-tapered inlets
A_f	sq ft	Area of inlet face section
A_t	sq ft	Area of inlet throat section
AHW EL.	ft	Allowable headwater elevation at culvert entrance
B	ft	Width of culvert barrel or diameter of pipe culvert
b	in	Dimension of side be bevel
B_b	ft	Width of bend section of slope-tapered inlets
B_f	ft	Width of face section of improved inlets
C_d		Discharge coefficient based on bend section control
C_f		Discharge coefficient based on face section control
C_t		Discharge coefficient based on throat section control
D	ft	Height of box culvert or diameter of pipe culvert
d	in	Dimension of top bevel
d_c	ft	Critical depth of flow
E	ft	Height of side-tapered pipe-culvert face section, excluding bevel dimension
FALL	ft	Approximate depression of control section below the stream bed
g	ft/sec^2	Acceleration of gravity = 32.2
H	ft	Head or energy required to pass a given quantity of water through a culvert flowing in outlet control
H_b	ft	Depth of pool, or head, above the bend section invert
H_c	ft	Depth of pool, or head, above the crest

Symbol	Units	Description
H_f	ft	Depth of pool, or head, above the face section invert
H_t	ft	Depth of pool, or head, above the throat section invert
HGL	ft	Hydraulic grade line
HW	ft	Headwater elevation; subscript indicates control section (HW, as used in HEC #5, is a depth and is equivalent to H_f in this Manual)
HW_c	ft	Headwater elevation required for flow to pass crest in crest control
HW_f	ft	Headwater elevation required for flow to pass face section in face control
HW_o	ft	Headwater elevation required for culvert to pass flow in outlet control
HW_t	ft	Headwater elevation required for flow to pass throat section in throat control
h_o	ft	Elevation of equivalent hydraulic grade line referenced to the outlet invert
k_e		Entrance energy loss coefficient
k_b		A dimensionless effective pressure term for bend section control
k_t		A dimensionless effective pressure term for inlet throat control
L_a	ft	Approximate total length of culvert, including inlet face section control
L_1, L_2, L_3, L_4	ft	Dimensions relating to the improved inlet as shown in sketches of the different types of inlets
N		Number of barrels
n		Manning's roughness coefficient
P	ft	Length of depression
Q	cfs	Volume rate of flow
R	ft	Hydraulic radius = $\frac{\text{Area}}{\text{Wetted Perimeter}}$

Symbol	Units	Description
S	ft/ft	Slope of culvert barrel
S_e	ft/ft	Slope of embankment
S_f	ft/ft	Slope of FALL for slope-tapered inlets (a ratio of horizontal to vertical)
S_o	ft/ft	Slope of natural channel
T	ft	Depth of the depression
Taper	ft/ft	Sidewall flare angle (also expressed as the cotangent of the flare angle)
TW	ft	Tailwater depth at outlet of culvert referenced to outlet invert elevation
V	ft/sec	Mean velocity of flow
W	ft	Width of weir crest for slope-tapered inlet with mitered face
W_p	ft	Top width of depression
y	ft	Difference in elevation between crest and face section of a slope-tapered inlet with mitered face

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CHAPTER 10 OPEN CHANNELS AND STRUCTURES

10.1 General Statement

Open channels designed for use in the major drainage system have significant advantages in regard to cost, capacity, multiple use for recreational and aesthetic purposes, and potential for instream storage and ground water recharge. Disadvantages include potential right-of-way (ROW) constraints and maintenance costs. Careful planning and design are needed to increase the benefits and to minimize the disadvantages. Criteria regarding use and design of open channels is presented in Chapters 2 and 6.

The ideal open channel is one that is a stabilized water course developed by nature over time, characterized by stable bed and banks. The benefits of such a channel are that:

- Available channel storage can decrease peak flows.
- Maintenance needs can be low when the channel is properly stabilized.
- Natural subsurface infiltration of flows is provided.
- Native vegetation and wildlife may not have to be disturbed.
- The channel can provide a desirable green belt and recreational area adding significant social benefits.

Generally speaking, a stabilized natural channel, or the man-made channel which most nearly conforms to the character of a stabilized natural channel, is the most efficient and the most desirable. Unfortunately, these types of channels are not common in El Paso County.

Channel stability, particularly in unprotected alluvial materials, is a problem in urban hydrology because of the significant increase in low flow and peak storm runoff rates. A natural channel must be studied in sufficient detail to determine the measures needed to mitigate potential bottom scour and bank cutting. Erosion control measures can be provided at reasonable cost which will preserve the natural appearance without sacrificing hydraulic efficiency. This chapter provides the necessary criteria and methodology for selection and design of open channels.

10.2 Types of Channels

Channel improvements associated with urban development can be categorized generally as either soft lined or hard lined. Soft linings include native soils with or without vegetative cover. Hard linings include concrete, riprap, and soil cement. These

general categories can be further broken down into soft side, hard bottom channels, and hard side, soft bottom channels. Following is a brief discussion of these four channel types.

10.2.1 Soft Lined Channels

Unlined channels in the plains of eastern Colorado are subject to severe erosion and are therefore undesirable in urbanized or urbanizing developments. Grass lined channels are the preferred means of conveying storm water runoff because of their desirability from the standpoint of erosion protection, maintainability, accessibility and aesthetics. Grasses typically used for channel lining are Bermudagrass, Kentucky bluegrass, orchardgrass, redtop, Stalian ryegrass, and buffalograss. Depending on the length and thickness of the grass cover, the channel surface is stabilized in a two-fold manner; (1) velocity is retarded, lessening the hydrodynamic forces against the soils, and (2) the soil is protected and reinforced by the grass and its root system. Grass linings can be used in moderately steep channel systems in conjunction with using drop structures for grade control.

The City/County acceptance for maintenance is subject to adequate maintenance accessibility, drainage easements and the installation of adequate stabilization features.

10.2.2 Soft Side, Hard Bottom Channels (Low Flow)

The continuous presence of minor flows in a grass lined channel creates problems with respect to maintenance. Flatter slopes tend to get boggy, with a proliferation of aquatic growth, while steep slopes have a greater potential for erosion. In such cases, it is necessary to construct a low flow channel of concrete or riprap along the channel invert, depending on the amount of flow. This type of channel is appropriate as long as the major storm events can be conveyed without exceeding the velocity limitations of the grass lined portions of the channel.

10.2.3 Hard Side, Soft Bottom Channels

This configuration is generally associated with a natural stream or floodway whose channel bottom has been stabilized by a natural process, and whose banks require protection against erosion during major storm events. Over steepening of the channel banks to accommodate flood plain improvements commonly precludes the use of grass linings and necessitates the use of more erosion resistant materials.

10.2.4 Hard Lined Channels

Hard lined channels are required in erosive soils where right-of-way constraints limit or preclude the use of grass linings. The use of concrete and soil cement linings is generally more

appropriate for channels that have flat slopes because the smooth surfaces have very little flow retarding capability. Riprap is more appropriate for steeper channels because of higher roughness coefficients. Riprap should not be considered where super critical conditions may exist.

10.3 Channel Discharge

10.3.1 Manning's Equation

Understanding the basic concepts of open channel flow is necessary to properly design channels. In open channel flow, the water surface is not confined. Surface configuration, flow pattern and pressure distribution within the flow depends on gravity. In rigid-boundary open channel flow, no deformation or movement of the bed and banks is assumed, whereas in mobile-boundary hydraulics, bed configuration is considered a function of the flow. Discussions in this chapter pertain primarily to rigid-boundary open-channel flow, since they are most appropriate to open channel design in the City/County.

Designing a stable alluvial channel (one without a channel lining) or a stable lined channel under dynamic channel conditions requires an understanding of sediment transport and stream channel response. Analysis of an unlined channel under dynamic channel conditions requires an understanding of sediment transport and stream channel response. For example, unlined channels must be designed to minimize excessive scour while lined channels must be designed to prevent deposition of sediments. Channel linings in dryland areas typically are hard surfaced due to the difficulties in growing the required vegetation. Unlined channels are most successful when designed under the concept of dynamic equilibrium. These topics and other related design considerations are discussed in detail in basic fluid mechanics and sediment transport textbooks.

All variables used in fluid mechanics and hydraulics fall into one of three classes; those describing the boundary geometry, those describing the flow, and those describing the fluid. Various combinations of these variables define parameters that describe the state of flow in open channels.

Careful attention must be given to the design of drainage channels to provide adequate capacity and allow for minimum maintenance. The hydraulic characteristics of open channels shall be determined by using Manning's equation commonly expressed as:

$$Q = \frac{1.49}{n} AR^{0.67} S^{0.5} \qquad (10\text{-}1)$$

where:

n = coefficient of channel roughness, dimensionless;

A = cross-sectional area of channel flow, in sq. ft.;

R = hydraulic radius of channel, $\frac{A}{P}$, in feet;

P = wetted perimeter of channel flow, in feet;

S = slope of the energy gradient, in feet per foot; and

Q = channel discharge, in cfs.

A graphical solution for the Manning equation is presented on Figure 10-1.

10.3.2 Uniform Flow

Manning's equation is an accurate representation of flow conditions only when the rate of flow and channel characteristics (roughness, cross section geometry and slope) remain relatively constant, hence, uniform flow. For a channel of given roughness, discharge and slope, there is only one possible depth for maintaining a uniform flow. This depth is commonly expressed as the normal depth. The corresponding discharge is expressed as the normal discharge. Under uniform flow conditions, the water surface is assumed parallel to both the energy grade line and the bottom of the channel.

Uniform flow is most often considered a theoretical abstraction. A channel is commonly designed on the assumption that it will convey uniform flow at the normal depth, but because of conditions difficult if not impossible to evaluate, the actual flow depth can actually differ from the theoretical uniform flow depth.

10.3.3 Normal Depth

Normal depth computations are made so frequently that it is convenient to use nomographs for various types of open channel cross sections to eliminate the need for trial and error solutions. Nomographs for estimating uniform flow are shown on Figures 10-1 and 10-2.

10.4 Design Considerations

Typical channel cross sections are triangular, trapezoidal and parabolic in shape. A triangular channel is a special type of trapezoidal section with a bottom width of zero, however, their application is generally not recommended due to the difficulty of maintenance and is not acceptable as a public storm water facility. Trapezoidal channels of varying bottom widths and side slopes are the most commonly constructed channels. Parabolic channels are generally used only when a vegetated lining is required, although different sections may be selected.

Man-made open channels are commonly designed to have trapezoidal sections of adequate cross sections to incorporate care of maintenance, uncertainties in runoff estimates, changes in channel roughness coefficients, channel obstructions and sediment accumulations. Figure 10-3a shows several typical cross sections used for design of grass-lined open channels in urban areas, including channels with alternative low flow channel designs. Figures 10-4 and 10-5 illustrate typical cross sections for riprapped and alternate open channel design configurations. These channel configurations may be necessary where there are limited ROW constraints and where hard lined channels are required.

Determination of a representative Manning's "n" value is critical in the analysis of the hydraulic characteristics of an open channel. The "n" value for each channel reach should be based on the individual channel characteristics. Table 10-1 presents a method of determining the composite roughness coefficient for an unlined channel reach based on actual channel conditions. Typical minimum, normal and maximum roughness coefficients for various types of open channels are presented in Table 10-2.

Where practicable, unlined channels of a given soil type should have sufficient gradient to provide flow velocities that will be self-cleaning but yet will not be so great as to create unacceptable erosion in terms of long term head cutting of the invert, lateral migration and potential property damage adjacent to the channel. Maximum permissible design flow velocities for earth channels are presented in Table 10-3. Table 10-4 presents maximum permissible velocities for earth channels with varied grass linings and sloping configurations. Lined channels, drop structures, check dams, or concrete spillways may be required to control erosion that results from high channel flow velocities. Overall, the design of open channels, including stable, alluvial channel systems, is tied closely to the criteria for erosion and sediment control.

10.5 Channel Cross Sections

The open channel cross section shape may be any type suitable to the location considering local environmental conditions. Often the channel configuration can be chosen to suit open space and recreational needs. Selection of cross section geometry should also consider ease of construction and maintenance. Formulas used in channel size design for typical cross section geometries are presented on Table 10-5.

10.5.1 Side Slopes

Selection of a channel side slope configuration largely depends on the angle of repose of the parent material or channel lining. The angle of repose is the slope angle formed by particulate material under the critical equilibrium condition of incipient sliding. For a stable channel, the side slope must be flatter than the angle of repose.

TABLE 10-1

COMPOSITE ROUGHNESS COEFFICIENTS FOR UNLINED OPEN CHANNELS
(Reference: Chow, Ven Te, 1959; Open-Channel Hydraulics)

$$n = (n_o + n_1 + n_2 + n_3 + n_4)m \qquad (10\text{-}2)$$

	Channel Conditions	Value
Material Type n_o	Earth	0.020
	Fine Gravel	0.024
	Coarse Gravel	0.028
Degree of Irregularity n_1	Smooth	0.000
	Minor	0.005
	Moderate	0.010
	Severe	0.020
Variation of Channel Cross Section n_2	Gradual	0.000
	Alternating Occasionally	0.005
	Alternating Frequently	0.010 - 0.015
Relative Effect of Obstructions n_3	Negligible	0.000
	Minor	0.010 - 0.015
	Appreciable	0.020 - 0.030
	Severe	0.040 - 0.060
Vegetation n_4	Low	0.005 - 0.010
	Medium	0.010 - 0.025
	High	0.025 - 0.050
	Very High	0.050 - 0.100
Degree of Meandering m	Minor	1.000 - 1.200
	Appreciable	1.200 - 1.500
	Severe	1.500

TABLE 10-2

TYPICAL ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS
(Reference: Chow, Ven Te, 1959; Open-Channel Hydraulics)

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

TABLE 10-2 (Continued)

TYPICAL ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

Type of Channel and Description	Minimum	Normal	Maximum
NATURAL STREAMS			
Minor streams (top width at flood stage 100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
LINED OR BUILT-UP CHANNELS			
a. Corrugated Metal	0.021	0.025	0.030
b. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	

TABLE 10-2 (Continued)

TYPICAL ROUGHNESS COEFFICIENTS FOR OPEN CHANNELS

Type of Channel and Description	Minimum	Normal	Maximum
c. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
d. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
e. Asphalt			
1. Smooth		0.013	
2. Rough		0.016	
f. Grassed	0.030	0.040	0.050

TABLE 10-3

MAXIMUM PERMISSIBLE DESIGN OPEN CHANNEL FLOW VELOCITIES IN EARTH*

Soil Types	Permissible Mean Channel Velocity (ft/sec)
Fine Sand (noncolloidal)	2.0
Coarse Sand (noncolloidal)	4.0
Sandy Loam (noncolloidal)	2.5
Silt Loam (noncolloidal)	3.0
Ordinary Firm Loam	3.5
Silty Clay	3.5
Fine Gravel	5.0
Stiff Clay (very colloidal)	5.0
Graded, Loam to Cobbles (noncolloidal)	5.0
Graded, Silt to Cobbles (colloidal)	5.5
Alluvial Silts (noncolloidal)	3.5
Alluvial Silts (colloidal)	5.0
Coarse Gravel (noncolloidal)	6.0
Cobbles and Shingles	5.5
Hard Shales and Hard Pans	6.0
Soft Shales	3.5
Soft Sandstone	8.0
Sound rock (usu. igneous or hard metamorphic)	20.0

* These velocities shall be used in conjunction with scour calculations and as approved by City/County.

TABLE 10-4

MAXIMUM PERMISSIBLE VELOCITIES FOR EARTH CHANNELS WITH VARIED GRASS LININGS AND SLOPES

Channel Slope	Lining	Permissible Mean Channel Velocity * (ft/sec)
0 - 5%	Sodded grass	7
	Bermudagrass	6
	Reed canarygrass	5
	Tall fescue	5
	Kentucky bluegrass	5
	Grass-legume mixture	4
	Red fescue	2.5
	Redtop	2.5
	Sericea lespedeza	2.5
	Annual lespedeza	2.5
	Small grains (temporary)	2.5
5 - 10%	Sodded grass	6
	Bermudagrass	5
	Reed canarygrass	4
	Tall fescue	4
	Kentucky bluegrass	4
	Grass-legume mixture	3
Greater than 10%	Sodded grass	5
	Bermudagrass	4
	Reed canarygrass	3
	Tall fescue	3
	Kentucky bluegrass	3

* For highly erodible soils, decrease permissible velocities by 25%.

* Grass lined channels are dependent upon assurances of continuous growth and maintenance of grass.

Except in horizontal curves, the flatter the open channel side slopes, the better. Side slopes for grass-lined channels shall be no steeper than 4H:1V, which is the practical limit for mowing equipment. Concrete-lined channels, or those which for other reasons require minimum or no slope maintenance, (i.e., channels lined with grouted riprap or soil cement), may have side slopes as steep as 2H:1V. Riprap lined channels may have slopes as steep as 2.5H:1V. Roadside ditches may have slopes as steep as 4H:1V.

For channels which are being constructed within existing site constraints including bridges and structures, concrete side slopes may be 1.5H:1V. These channels must have adequate fencing for general safety of the public.

10.5.2 Depth

Channel depth should not exceed 5.0′ at the 100-year storm when the 100-year flow is approximately 1500 cfs or less. Excessive depths should be avoided to minimize high velocities and for other public safety considerations.

10.5.3 Bottom Width

Open channels with narrow bottoms are difficult to maintain and can be subjected to high flow velocities during periods of excess runoff. It is desirable to design open channels such that the bottom width is at least twice the design flow depth, but not less than eight (8) feet for channels conveying more than 400 cfs.

10.5.4 Low Flow Channels

Channel low flows, including base flows, from urban areas must be given special attention. If erosion of the bottom of the channel appears to be a potential problem, low flows shall be carried in a riprapped or concrete lined channel which generally has a minimum conveyance capacity of a 10-year duration storm. A minimum conveyance capacity of down to 10% of the 100 year storm event may be allowed if overbank conditions and scour velocities permit or only as otherwise approved by the City/County Engineer.

10.5.5 Freeboard

In open channel flow, problems have been encountered with maintaining the flow within the designed channel, i.e., the flow overtops the channel lining, resulting in serious erosion problems and possible failure. Many of these problems can be prevented when design consideration is given to (1) centrifugal forces that occur where the channel is curved and (2) additional channel depth to account for debris accumulation or variance in construction that results in differences in roughness coefficients. These two design concepts are known as

superelevation and freeboard, respectively. For smaller channels, the freeboard is often sufficient to account for centrifugal forces and superelevation need not be considered.

For channels characterized by high flow velocities during periods of runoff, wave action, air bulking, and splash and spray along the top water surface are erosion potential mechanisms. Therefore, freeboard heights should be chosen to provide a suitable safety margin. The following formula shall be used to estimate freeboard. The height of freeboard shall be a minimum of one (1) foot.

$$\text{Freeboard (in feet)} = 1.0 + 0.025\ (v)(d^{0.33}) \qquad (10\text{-}3)$$

where:

v = average channel flow velocity, in fps; and
d = depth of flow, in ft.

10.5.6 Superelevation

To meet the minimum freeboard requirement of a channel along a sharp bend, <u>additional</u> design flow depth at the outside bank or wall, or height of additional embankment height protection, must be provided. This amount can be determined by the following relationship:

$$H = C\,\frac{v^2 W}{gR} \qquad (10\text{-}4)$$

where:

C = coefficient;

= 0.5, subcritical flow, simple curve

= 1.0, supercritical flow, simple curve

v = average velocity of flow in channel, in fps;

W = channel width at level water surface

g = acceleration of gravity constant, 32.2 ft/sec^2;

R = channel centerline radius of curvature, in ft; and

H = additional height of freeboard on outside edge of channel, in ft.

The minimum length of transition from superelevated freeboard to normal freeboard shall be equal to twice the top width of the channel. The minimum radius of curvature to the channel centerline shall be equal to three times the top width of the design flow but not less than 100 feet.

For supercritical flows with the possibility of cross waves generated downstream of the simple curve, channel heights should be increased as for superelevation not only in the curve but a considerable distance downstream.

10.6 Backwater Analysis

For final design, water-surface profiles must be computed for all channels. Computation of the water-surface profile should utilize standard backwater analysis, and should consider all losses due to changes in channel velocity, drops, curves, bridge openings, and other obstructions. Computations begin at a known point, and extend in an upstream direction for subcritical flow.

Backwater computation can be made using the methods presented in Open Channel Hydraulics by Ven Te Chow (1959). The use of backwater analysis can be easily programmed on calculators or computers. Many computer programs are also available for computation of backwater curves. The most general and widely used program is the HEC-2 Water Surface Profiles, developed by the U.S. Army Corps of Engineers (1973), and is the program recommended for flood water profile computations. This program can be used to compute water-surface profiles for both natural and man-made channels.

When designing for bends in open channels the effects of superelevation and energy losses due to resistance in the bends must be considered in backwater computations. Generally, the effects of superelevation can be calculated by Equation (10-4), as presented in Section 10.5.6. In addition to superelevation on bends, flow separation in the bend creates a backwater effect that must also be considered. More detail on determining these effects may be found in Chow (1959).

10.7 Supercritical Flow

The specific energy head for open channel flow may be expressed by the following equation, which is the sum of the potential energy (represented by the water depth, d) and the kinetic energy (represented by the velocity head, $V^2/2g$):

$$H = d + \frac{v^2}{2g}$$

where:

d = depth of flow, in ft;

v = velocity of flow, in fps;

g = acceleration of gravity constant, 32.2 ft/sec^2; and

H = specific energy head, in ft.

When depth of flow is plotted against specific energy for a given channel discharge at a section, the resulting curve shows that, at a given specific energy, there are two possible flow depths (see Figure 10-6). At minimum energy, only one depth of flow exists. This is known as the critical depth. At critical depth, the depth is equal to twice the velocity head, or:

$$d_c = \frac{v^2}{g} \qquad (10\text{-}6)$$

The Froude Number (dimensionless), F, is defined as:

$$F = \frac{V}{\sqrt{gd}} \qquad (10\text{-}7)$$

For F = 1, or unity, the flow is critical. If the Froude Number is greater than 1, the flow is supercritical; when the Froude Number is less than 1, the flow is subcritical. Figure 10-7 shows the relationship of the energy profile to the water surface profile for various subcritical and supercritical flow conditions. To minimize the potential for unstable flow (transitioning uncontrollably from subcritical to supercritical). channels should be designed for Froude numbers less than 0.9.

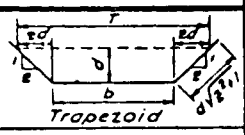
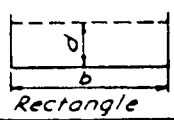
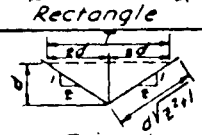
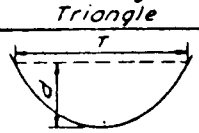
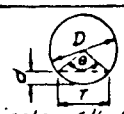
Supercritical flow in an open channel, particularly in an urbanized area creates certain hazards which the designer must take into consideration. From a practical standpoint, it is generally good design practice to avoid curvature in such a channel. Careful attention must be taken to ensure against excessive oscillatory waves (unstable flow) which may extend along the entire length of the channel from only minor obstructions upstream. Imperfections at joints of lined channels may rapidly cause a deterioration of the joints, in which case a complete failure of the channel may occur. In addition, high-velocity flow entering cracks or joints creates an uplift force by the conversion of velocity head to pressure head which can damage the channel lining. It is evident that when designing a lined channel with supercritical flow, the designer must use the utmost care and judgement and consider all relevant factors.

10.8 Energy Dissipators

Energy dissipators are often necessary at the end of outfall sewers, culverts or channels. Various types of energy dissipators are useful at locations where the designer wants to change the flow from supercritical to subcritical downstream from a high-velocity channel or conduit in order to reduce or limit potential erosion.

Table 10-5

Hydraulic Elements of Open Channel Sections

Section	Area a	Wetted Perimeter p	Hydraulic Radius r	Top Width T
 Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$
 Rectangle	bd	$b+2d$	$\frac{bd}{b+2d}$	b
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2zd$
 Parabola	$\frac{2}{3}dT$	$T+\frac{8d^2}{3T}$ [1]	$\frac{2dT^2}{3T^2+8d^2}$ [1]	$\frac{3a}{2d}$
 Circle - < 1/2 full [2]	$\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$\frac{\pi D\theta}{360}$	$\frac{45D}{\pi\theta}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$
Circle - > 1/2 full [3]	$\frac{D^2}{8}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$\frac{45D}{\pi(360-\theta)}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$

[1] Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$
When $d/T > 0.25$, use $p = \frac{1}{2}\sqrt{16d^2+T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

[2] $\theta = 4\sin^{-1}\sqrt{d/D}$
[3] $\theta = 4\cos^{-1}\sqrt{d/D}$ } Insert θ in degrees in above equations

10.8.1 Impact-Type

Generally, this type of basin lends itself to use with pipes. It is an effective stilling device even with deficient tailwater where the discharge is relatively small. This basin can be used with either an open chute or a closed conduit structure. The design shown on Figure 10-8 has been used for discharges up to 400 cfs. For larger discharges, multiple basins could be placed side-by-side.

The general arrangement of the basin and the dimensional requirements for various discharges are shown on Figure 10-8a and presented in Figure 10-8b. This type of basin is subject to local flow turbulences and large dynamic forces which must be considered in the structural design. The structure must be made sufficiently stable to resist sliding due to the impact load on the baffle wall. The entire structure must also resist the severe vibrations inherent in this type of device, and the individual structural members must be sufficiently strong to withstand the large dynamic loads.

Riprap should be provided along the bottom and sides adjacent to the structure to avoid potential local scour of the outlet channel downstream from the end sill when a shallow tailwater condition exists. Downstream wingwalls placed at 45 degrees may also be effective in reducing scouring tendencies and flow concentrations downstream.

As stated in Reference 7, the impact-type energy dissipator discussed was developed by the Bureau of Reclamation and requires no tailwater for successful performance.

Energy dissipation is initiated by flow striking the vertical hanging baffle and being deflected upstream by the horizontal portion of the baffle and by the floor, creating horizontal eddies.

The notches shown in the baffle, Figure 10-A-3, are provided to aid in cleaning the basin after prolonged non-use of the structure. If the basin is full of sediment, the notches provide concentrated jets of water for cleaning. The basin is designed to carry the full discharge over the top of the baffle if the space beneath the baffle becomes completely clogged. Although this performance is not good, it is acceptable for short periods of time.

The design information is presented as a simple dimensionless curve, Figure 10-A.1. It represents the ratio of energy entering the dissipator to the width of dissipator required, plotted with respect to the Froude number. The Los Angeles County tests indicate that limited extrapolation of this curve is permissible.

In calculating the energy and Froude number, the equivalent depth of flow entering the dissipator from a pipe or irregular-shaped conduit must be computed on the basis of:

$$y_e = (A/2)^{1/2}$$

In other words, the cross section flow area in the pipe is converted into an equivalent rectangular cross section in which the width is twice the depth of flow. The conduit preceding the dissipator can be open, closed, or of any cross section. The design method is enhanced by ignoring the size and shape of conduit entirely, except for the determination of the depth of flow entering the dissipator.

Suppose the conduit leading to the dissipator is circular and flowing half full, the water area will be $D^2/8$. The representative depth of flow used in computing the energy entering, H_o, and the Froude number, F, will be:

$$y_e = (A/2)^{1/2} \quad \text{or} \quad (D^2/16)^{1/2}$$

The specific energy $H_o = y_e + V_o^2/2g$ and the Froude number

$$F = V_o/(gy_e)^{1/2}.$$

Although tailwater is not necessary for successful operation, a moderate depth of tailwater will improve the performance. For best performance set the basin so that maximum tailwater does not exceed $h_3 = (h_2/2)$.

The basin should be constructed horizontal for all entrance conduits with slopes greater than 15°, a horizontal section of at least four conduit widths long should be provided immediately upstream of the dissipator. Although the basin will operate fairly effectively with entrance pipes on slopes up to 15°, experience has shown that it is more efficient when the recommended horizontal section of pipe is used. In every case, the proper position of the entrance invert, should be maintained.

When a hydraulic jump is expected to form in the downstream end of the pipe and the entrance is submerged, a vent about one-sixth the pipe diameter should be installed at a convenient location upstream from the jump.

For erosion reduction and better basin operation, use the alternative end sill and 45° wingwall design as shown in Figure 10-A.3.

For protection against undermining, a cutoff wall should be added at the end of the basin. Its depth will depend on type of soil present.

Riprap should be placed downstream of the basin for a length of at least four conduit widths.

The sill should be set as low as possible to prevent degradation downstream. For best performance, the downstream channel should be at the same elevation as the top of the sill. A slot should be at the same elevation as the top of the sill. A slot should be placed in the end sill to provide for drainage during periods of low flow.

To provide structural support and aid in priming the device, a short support should be placed under the center of the baffle wall.

Use of the basin is limited to installations where the velocity at the entrance to the stilling basin does not exceed 50 feet per second and discharge is less than 400 cfs. This dissipator is not recommended where debris or ice buildup may cause substantial clogging.

The following design procedures are provided:

1. From the maximum discharge and velocity, compute the flow area at the end of the approach pipe. Compute y_e for a rectangular section of equivalent area twice as wide as the depth of flow, $y_e = (A/2)^{1/2}$.

2. Compute the Froude number F and the energy at the end of the pipe H_o. Enter the curve on Figure 10-A.1 and determine the required width of basin W.

3. With W know, obtain the remaining dimensions of the dissipator structure from Figure 10-8b.

<u>EXAMPLE PROBLEM</u>

Given: $D = 48$ inches

$S_o = 0.15$

$Q = 300$ cfs

Find: USBR Impact Basin dimensions for use at the outlet of a concrete pipe.

$n = 0.015$
$V_o = 40$ fps
$y_o = 2.3$ feet

Solution: Since Q is less than 400 cfs and Vo less than 50 fps, the dissipator may be tried at this site.

1. Compute y_e

$y_e = (A/2)^{1/2}$

$A = Q/V_o = 300/40 = 7.5$ sq. ft.

$y_e = (7.5/2)^{1/2} = 1.94$ feet

2. Compute F and H_o and find W

$F = V_o/gy_e)^{1/2} = 40/(32.2 \times 1.94)^{1/2} = 5.05$

$H_o = y_e + V_o^2/2g = 1.94 + (40)^2/64.4 = 26.8$ feet

From Figure 10-A.1

$H_o/W = 1.68$

$W = 26.8/1.68 = 16.0$ feet

3. From Figure 10-8b select remaining dimensions

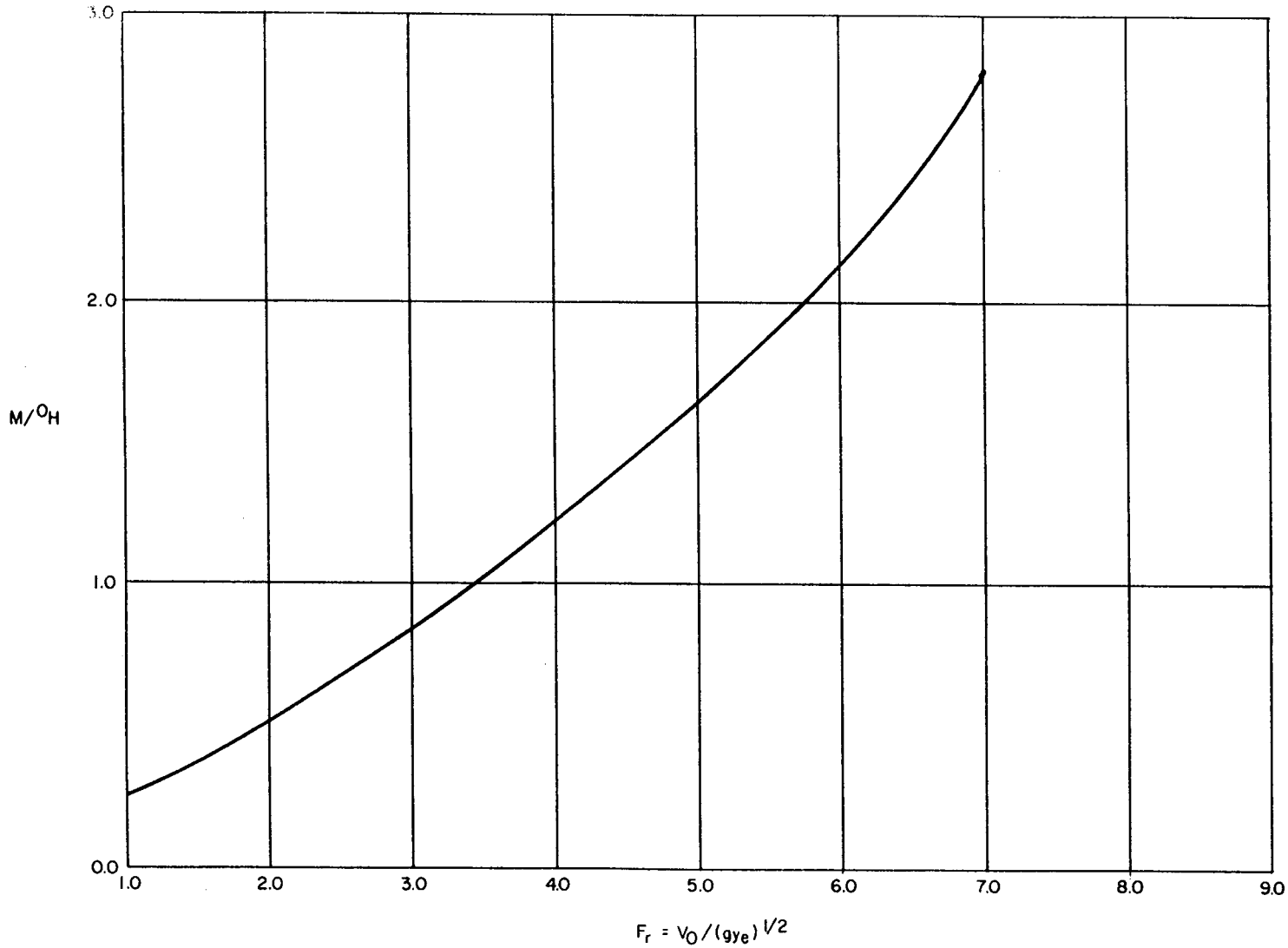
Design a second baffle wall dissipator at the end of a long rectangular concrete channel 4 feet wide using the same depth of flow = 2.3 feet, $S_o = 0.15$ and n = 0.015 as in the previous example and compare results.

The discharge for the rectangular channel flowing at a depth of 2.3 feet will be 375 cfs. The computations and comparison with the first example are tabulated below.

Channel			Circular	Rectangular
Depth of flow	y_o	ft	2.3	2.3
Area of flow	A	sq ft	7.50	9.20
Discharge	Q	cfs	300	375
Velocity	V_o	fps	40	40.9
Flow depth (rect. section)	y_e	ft	1.9	2.3
Velocity Head	$V_o^2/2g$	ft	24.9	26
$H_c = y_e + V_o^2/2g$		ft	26.8	28.3
$F = V_o/(gy_e)^{1/2}$		--	5.05	4.75
From Figure 10-A.1	H_o/W	--	1.68	1.55
Width of basin	W	ft	16.0	18.3
$H_L/H_o(100)$ - Low Tailwater			67%	65%

Entering Figure 10-8.b with W = 16 feet and W = 18, the remaining dimensions of the two dissipators can be read directly in feet and inches. The basin width is taken to the nearest half foot, while the other dimensions are read to the nearest inch. This degree of accuracy is sufficient.

Also see, "Hydraulic Design of Stilling Basins," Journal of the Hydraulics Division, A.S.C.E., paper 1406, October 1957.



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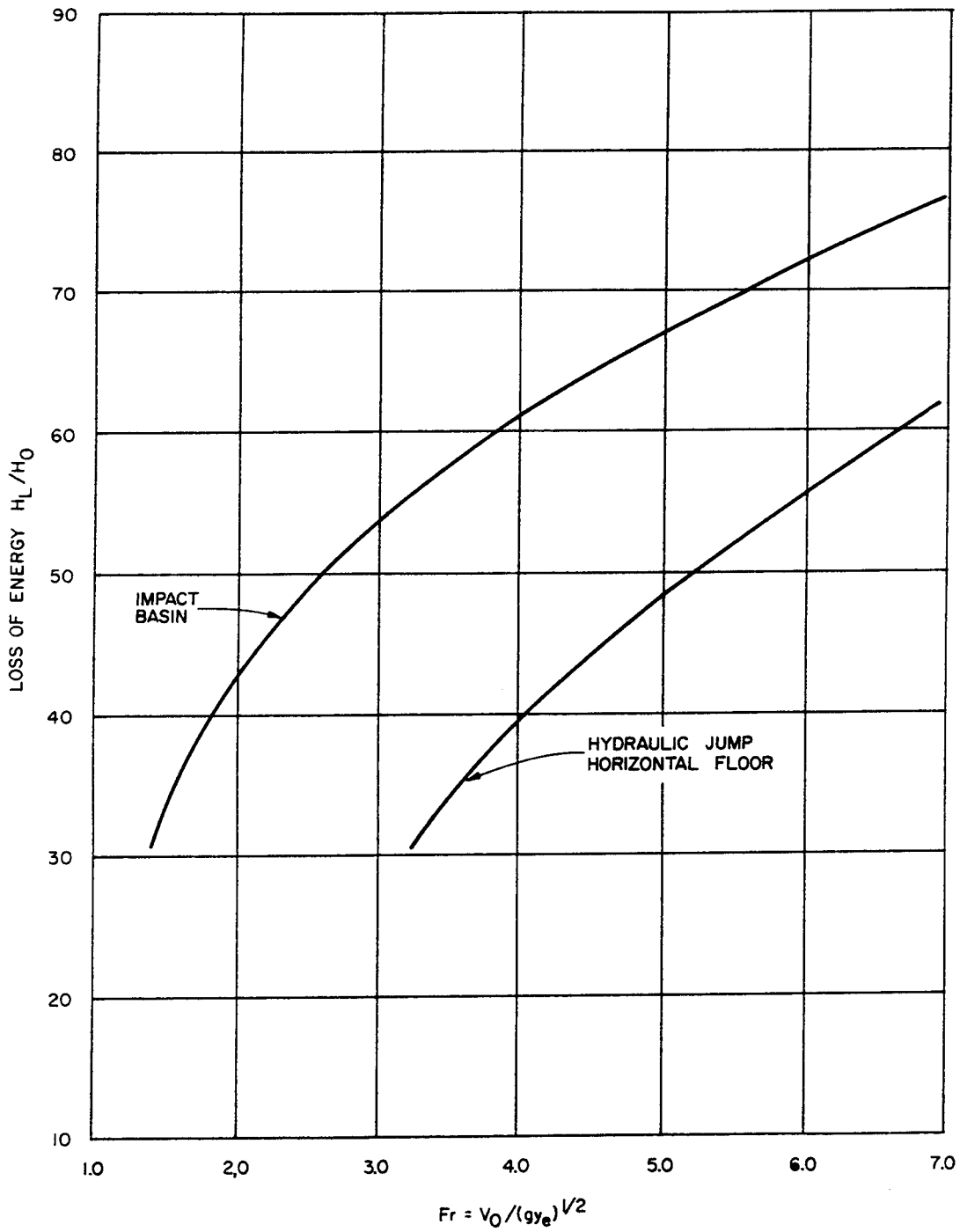
Design Curve - Impact Basin

Date
9-10-90

Figure
10-A.1



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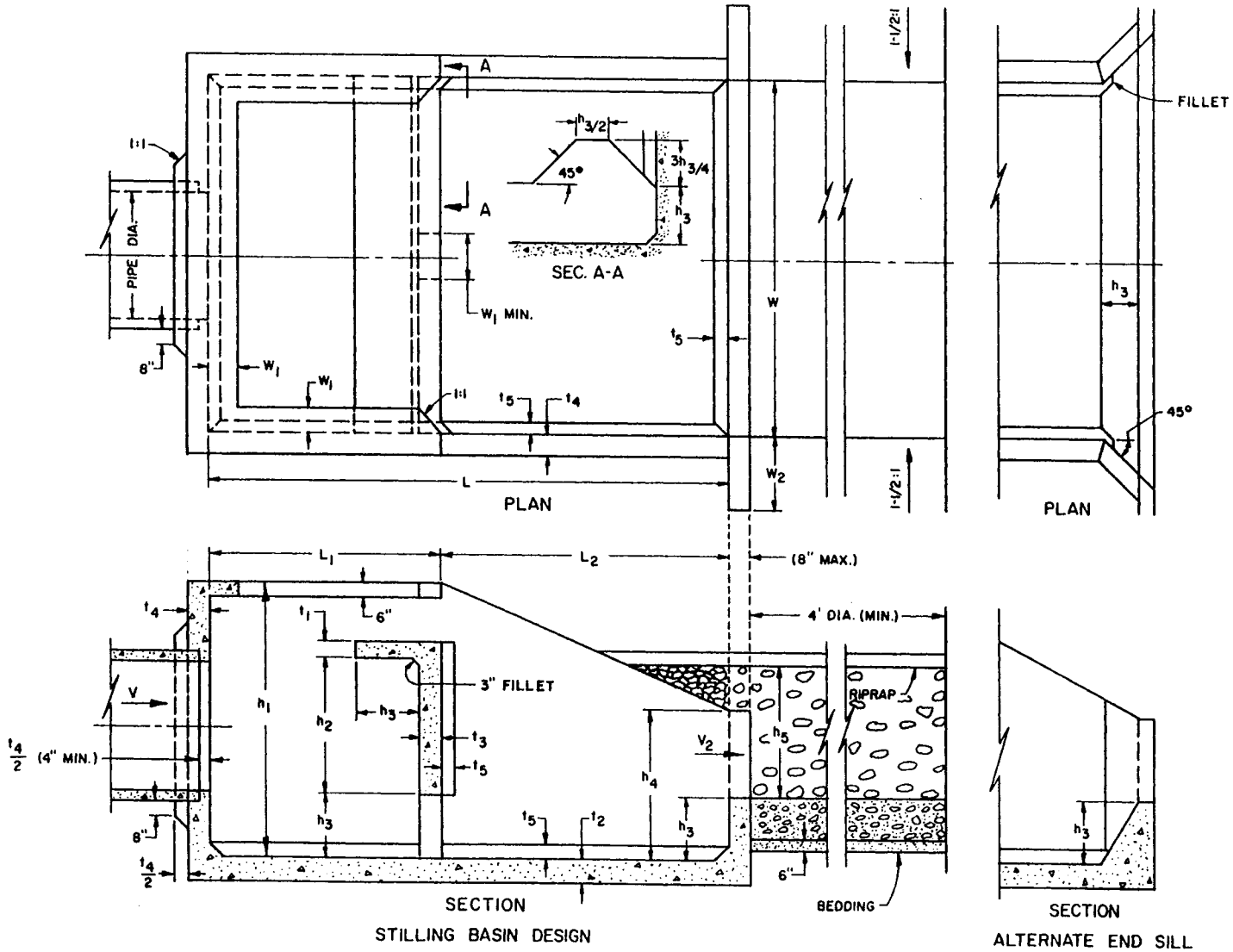
Energy Loss - Impact Basin - Hydraulic Jump	Figure 10-A.2



IMPACT ENERGY DISSIPATOR - USBR TYPE VI

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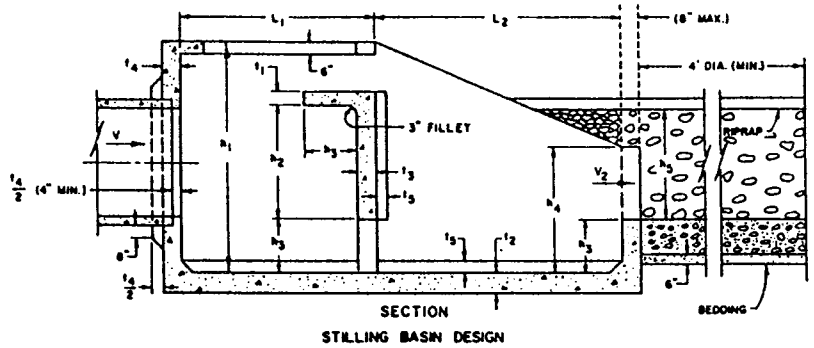
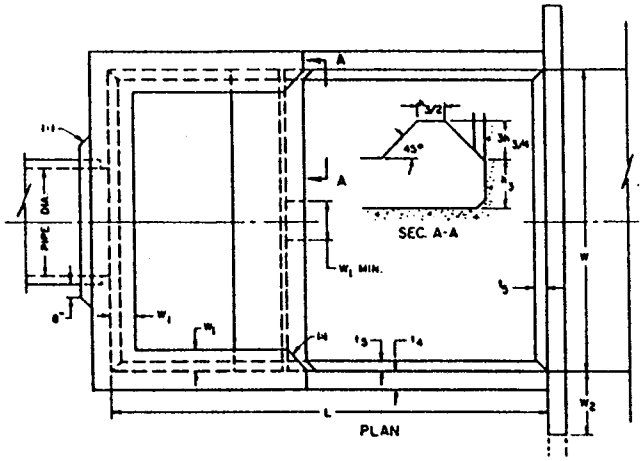
Figure
10-A.3



STILLING BASIN DESIGN



	H_O	H_O/W	W	h_1	L	h_2	h_3	L_1	L_2
	h_4	W_1	W_2	t_3	t_2	t_1	t_4	t_5	

IMPACT TYPE ENERGY DISSIPATOR USBR VI - DESIGN CHART

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Figure
10-A.4

10.8.2 Contra Costa Type

As stated in Reference 7, the Contra Costa energy dissipator was developed at the University of California, Berkley, in conjunction with Contra Costa County, California. The dissipator was developed to meet the following conditions: (1) to reestablish natural channel flow conditions downstream from the culvert outlet; (2) to have self-cleaning and minimum maintenance properties; (3) to drain by gravity when not in operation; (4) to be easily and economically constructed; and (5) to be applicable for a wide range of culvert sizes and operating conditions.

Field experience with this dissipator has been very limited. Its use should not be extended beyond the range of the model tests.

The dissipator is best suited to small and medium size culverts of any cross section where the depth of flow at the outlet is less than the culvert height. It is applicable for medium and high velocity effluents. The dissipator design is such that the flow leaving the structure will be at minimum energy when operating without tailwater. When tailwater is present, the performance will improve. A sketch of the dissipator arrangement is shown in Figure 10-B.4.

The initial step is to determine the equivalent depth of flow (y_e) at the culvert outfall: For box culverts $y_e = y_n$ or y brink. For oval, elliptical, circular, or other shapes, convert the areas of flow at the culvert outfall to an equivalent rectangular cross section with a width equal to twice the depth of flow $y_e = (A/2)^{1/2}$. In so doing, the design information is applicable to oval, elliptical, and circular culverts.

The Froude number is computed by using y_e rather than the actual depth of flow at the culvert outfall, $F^2 = V/gy_e$. By entering Figure 10-B.1 with F^2, and an assumed value of L_2/h_2, a trial height of the second baffle, h_2, can be determined. The equation of the lines in Figure 10-B.1 has been changed slightly from that presented in the original paper to compensate for replacing the depth of flow in a circular pipe (y_o) by the equivalent rectangular flow depth, y_e. The original equation

$$L_2/h_2F^2 = 1.2(h_2/y_o)^{1.8}$$

has been revised to

$$L_2/h_2F^2 = 1.35(h_2/y_e)^{1.8} \qquad (10\text{-A-1})$$

The remaining two equations, used for determining other dimensions of the dissipator, remain unchanged; these are given below and plotted on Figures 10-B.2 and 10-B.3.

$$L_3/L_2 = 3.75(h_2/L_2)^{0.68} \qquad (10\text{-A-2})$$

$$y_2/h_2 = 1.3(L_2/h_2)^{0.36} \qquad (10\text{-A-3})$$

The three equations may be used for proportioning the dissipator but Figures 10-B.1, B.2, and B.3 are more convenient and practical to use for design purposes.

The value of L_2/h_2 varied from 2.5 to 7.0 in the experiments and a value of 3.5 is recommended for best performance wherever economically feasible. The value of h_2/y_e should always be greater than unity. After determining values of h_2 and L_2 from Figure 10-B.1, the dimension L_3 can be obtained by entering Figure 10-B.2 with L_2 and the assumed value of L_2/h_2. Should the dimensional proportioning thus obtained be uneconomical or fail to properly fit the site, a second value of L_2/h_2 is assumed and the process repeated.

From Figure 10-B.4, the height h_1 of the first baffle is half the height of the second baffle h_2, and the position of the first baffle is half way between the culvert outlet and the second baffle or $L_2/2$. Side slopes of the trapezoidal basin for all experimental runs were 1:1. The width of basin (W) may vary from one to three times the width of the culvert. The floor of the basin should be essentially level. The height of the end sill may vary from $0.06(y_3)$ to $0.10(y_3)$. After obtaining satisfactory basin dimensions, the approximate maximum water surface depth, y_2, without tailwater, can be obtained from Figure 10-B.3.

The following design procedure is provided: The dissipator design should only be applied within the design limitations:

$2.5<L_2/h_2<7.0$

$D<W<3D$

$Y_0<D/2$

and side slopes of 1:1

The following steps outline the procedure for the design of the design of the Contra Costa energy dissipator:

1. Analyze flow conditions that are expected to occur at the outfall of culvert for the design discharge.

   If the depth of flow at the outlet is one half culvert diameter or less, the Contra Costa dissipator is applicable.

2. Compute y_e:

   $y_e = y_o$; for rectangular

   $y_e = (A/2)^{1/2}$; for other shapes

3. Compute the parameter $F^2 = V_o^2/gy_e$

4. The width of the basin floor is selected to conform to the natural channel. If there is no defined channel, the width is set at a maximum of three times the culvert width.

5. Assume a value of L_2/h_2 between 2.5 and 7 and with the aid of Figures 10-B.1 and B.2, determine h_2, L_2 and L_3: Give due consideration to the optimum value of $L_2/h_2 = 3.5$ as well as to the engineering and economic requirements of the particular situation. Repeat the procedure, if necessary, until a dissipator is defined, which optimizes the design requirements. The first baffle height (h_1) is $0.5h_2$.

6. The approximate maximum water surface depth without tailwater can be obtained for the final arrangement from Figure 10-B.3.

7. Riprap may be necessary downstream especially for the low tailwater cases. Freeboard and a cutoff wall also should be considered to prevent overtopping and undermining of the basin.

Example Problem

Given: Diameter of culvert = 4.0 feet
$Q = 300$ cfs
$y_o = 2.3$ ft
$V_o = 40$ fps
$A_o = 7.50$ sq ft

Find: A Contra Costa energy dissipator dimensions

Solution:

1. $y_o = 2.3$ ft D/2, OK

2. $y_e = (7.5/2)^{1/2} = 1.94$ ft

3. $F^2 = V_o^2/g(y_e) = 40^2/(32.2 \times 1.94) = 25.6$

4. $W = 2D = 8$ ft

5. By assuming $L_2/h_2 = 3.5$ and entering Figure 10-B.1 with $F^2 = 25.6$, a value of 3.50 is obtained for h_2/y_e. Therefore,

$h_2 = 3.50(1.94) = 6.8$ ft and

$h_1 = .5(6.8) = 3.4$ ft

$L_2 = 3.5(6.8) = 23.8$ ft

Entering Figure 10-B.2 with L_2 = 23.8 feet and L_2/h_2 = 3.5, L^3 is found to be 38 feet. If the maximum rise in water surface, without tailwater, is desired, this can be obtained from figure 10-B.3. Strictly speaking, the value of y_2 from this chart applies for a bottom width W/D = 2 and 1:1 side slopes. For the problem at hand, these values are essentially correct.

8. Entering Figure 10-B.3 with h_2 = 6.8 ft and L2/h2 = 3.5, gives y2 = 13.9 feet. If the height of end sill is based on this value,

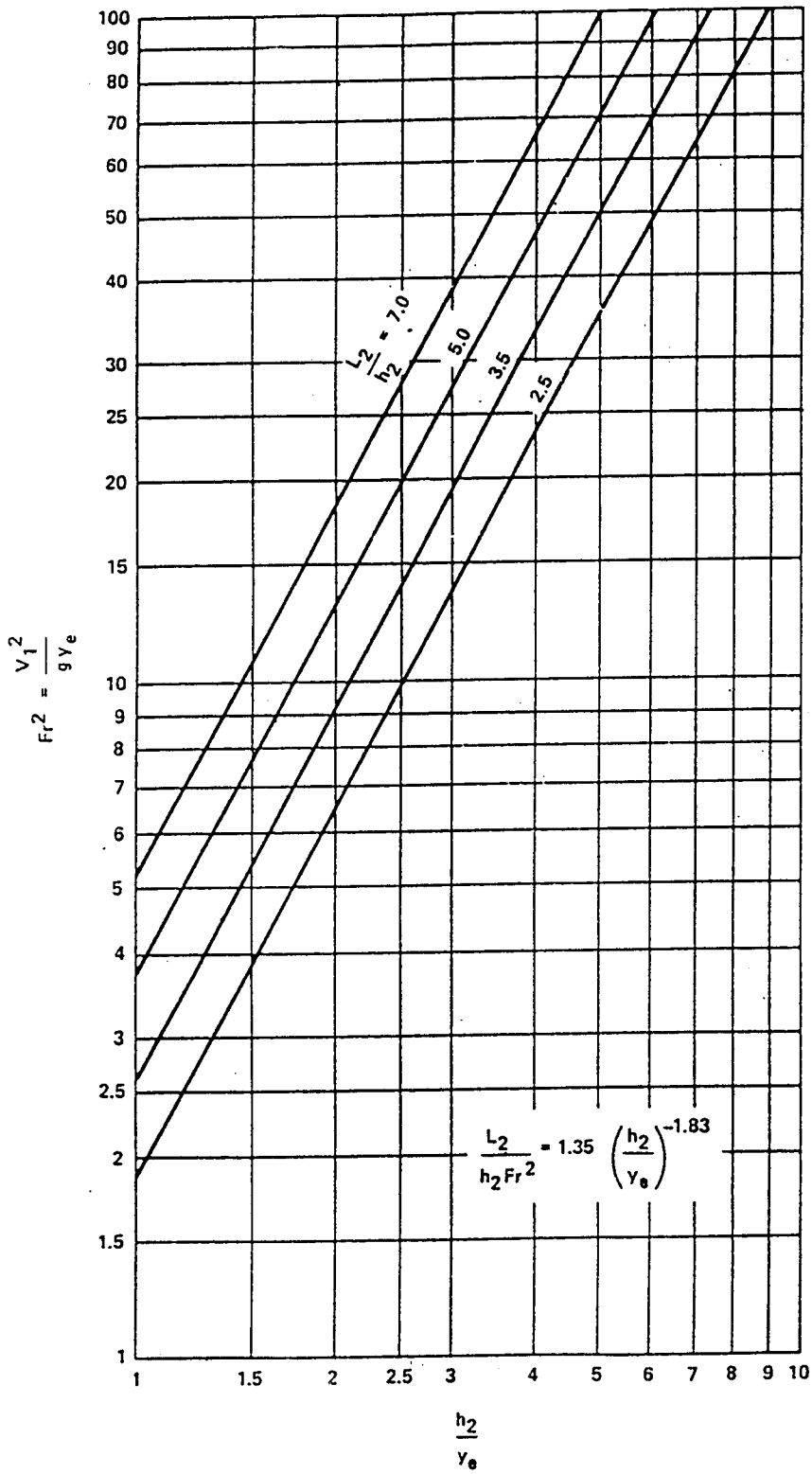
$$h_3 = 0.09(y_2) = 0.09(13.9) = 1.25 \text{ ft}$$

If the above proportioning proves compatible with the topography at the site and the dissipator is economically satisfactory, the above dimensions are final; if not, a different value of L_2/h_2 is selected and the design procedure repeated. Assuming the first computation is acceptable, the various dimensions of the dissipator are:

Dimensions in Feet and Inches

	W	First Baffle	Second Baffle	End Sill
Hor. Distance	8-0	12-0	24-0	62-0
Height	-	3-6	7-0	1-3
Length (baffle)	-	8-0	8-0	10-6

Also see, Keim, R. S., THE CONTRA COSTA ENERGY DISSIPATOR, Journal of Hydraulics Division, A.S.C.E., paper 3077, March 1962, p. 109.



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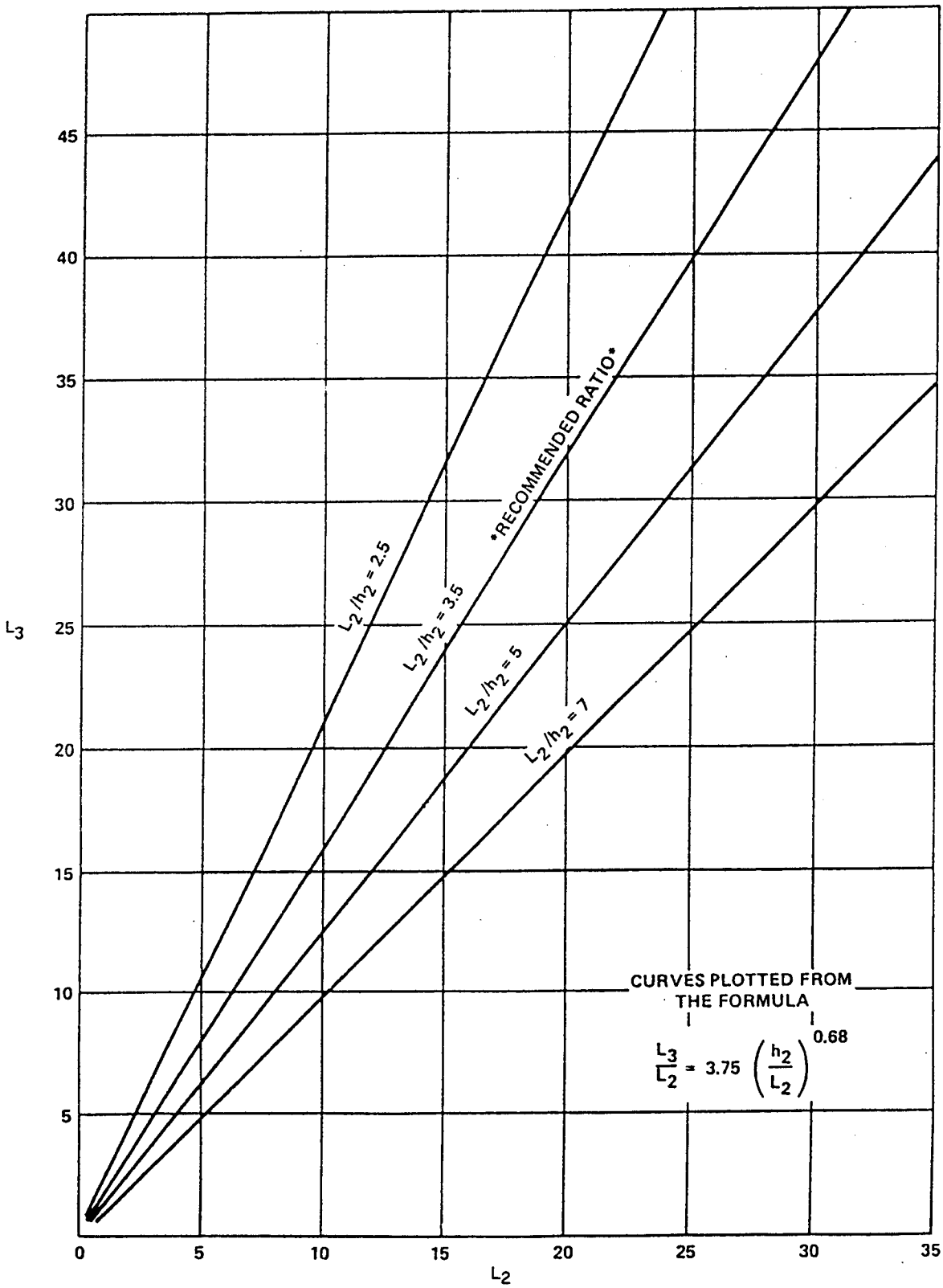
Baffle Height, Contra Costa Energy Dissipator	Figure 10-B.1



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Length of Contra Costa Energy Dissipator

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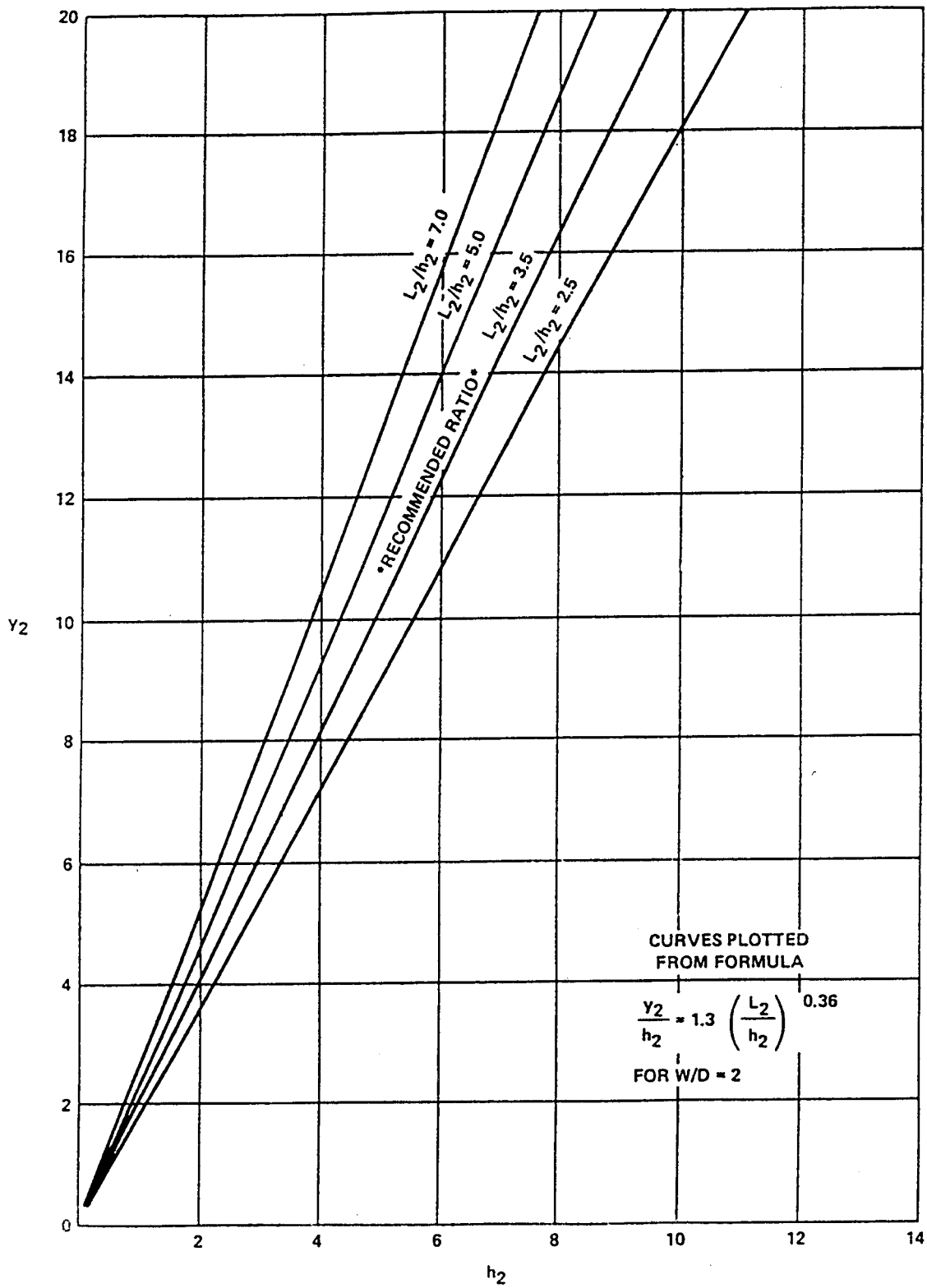
Figure 10-B.2



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Depth Over Baffle - Contra Costa Energy Dissipator

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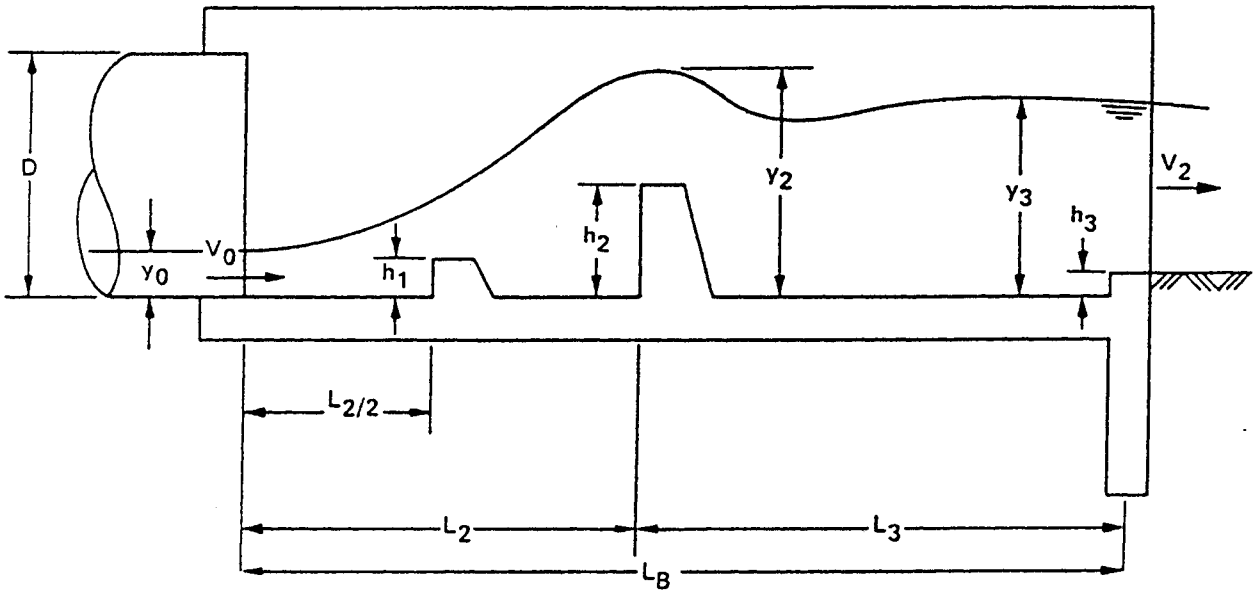
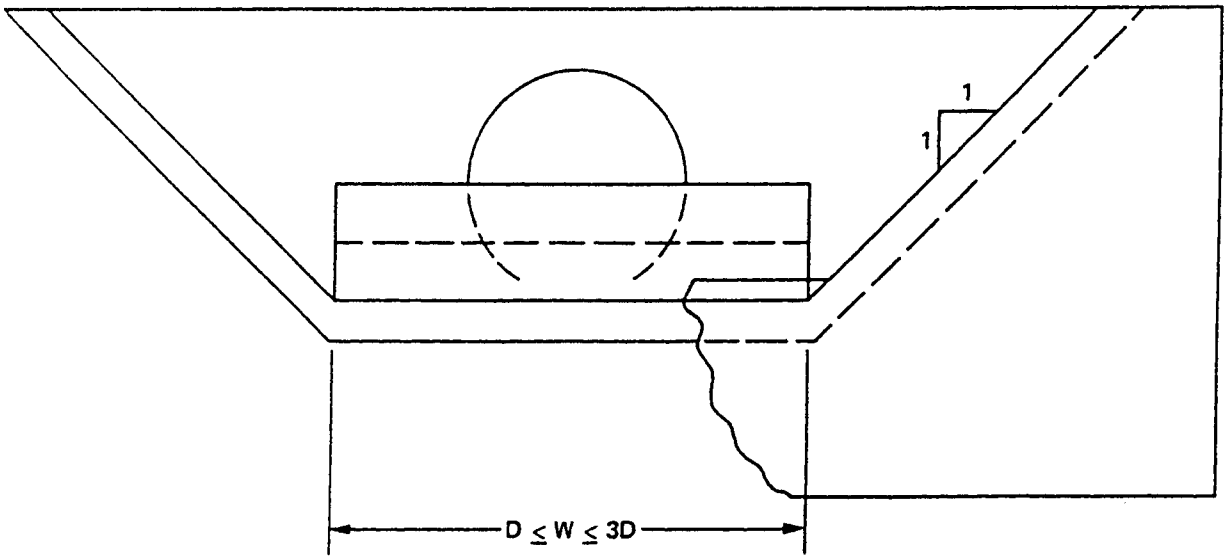
Figure 10-B.3





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Contra Costa Energy Dissipator - Dimensions

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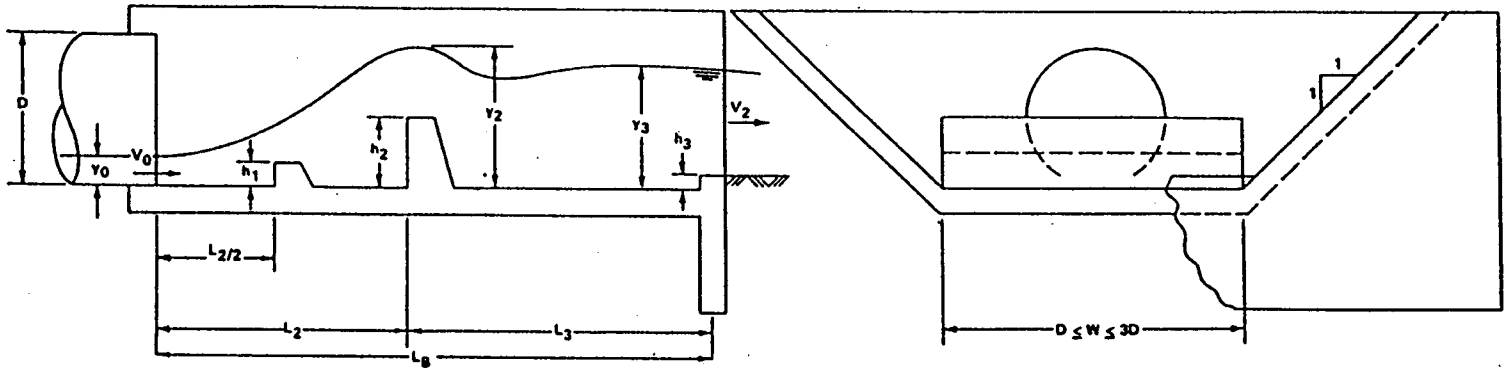
Figure 10-B.4



y_o/D(1)	y_e	F_r^2	W(2)	L_2/h_2	h_2/y_e	h_2	h_1	L_2	L_3

LB	y_2	h_3

(1) y_o/D should be approx. 0.5

(2) $D<W \leq 3D$

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Contra Costa Design Chart

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Figure 10-B.5

10.8.3 Plunge Pool Type (Riprap Basin)

As stated in Reference 7, the design procedure for riprap energy dissipators is based on data obtained during a study "Flood Protection at Culvert Outfalls" sponsored by the Wyoming Highway Department and conducted at Colorado State University. The purpose of the experimental program was to establish relationships between flow properties and the dimensions of riprapped basins at culvert outfalls.

The depth (h_s), length (L_s), and width (W_s) of the scour hole were related to the characteristic size of riprap (d_{50}), discharge (Q), brink depth (y_o), and tailwater depth (TW).

The dimensions of a scour hole in a basin constructed with angular rock were approximately the same as the dimensions of a scour hole in a basin constructed of rounded material when rock size and other variables were similar.

When the ratio of tailwater depth to brink depth (TW/y_o) was less than 0.75 and the ratio of scour depth to size of riprap (h_s/d_{50}) was greater than 2.0, the scour hole functioned very efficiently as an energy dissipator. The concentrated flow at the culvert brink plunged into the hole, a jump formed against the downstream extremity of the scour hole, and flow was generally well dispersed as it left the basin.

The mound of material which formed on the bed downstream of the scour hole contributed to the dissipation of energy and reduced the size of the sour hole; i.e., if the mound from a stable scoured basin was removed and the basin was again subjected to design flow, the scour hole enlarged somewhat.

For high tailwater basins (TW/y_o greater than 0.75) the high velocity core of water emerging from the culvert retained its jetlike character as it passed through the basin, and diffused in a manner very similar to that of a concentrated jet diffusing in a large body of water. As a result, the scour hole was much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rock-lined basin.

General details of the basin recommended in this report are shown on Figure 10-C.5. Principal features of the basin are:

1. The basin is preshaped and lined with riprap.

2. The surface of the riprapped floor of the energy dissipating pool is constructed at an elevation h_s below the culvert invert. h_s is the approximate depth of scour that would occur in a thick pad of riprap, constructed at the outfall of the culvert, if subjected to the design discharge. The ratio of h_s to d_{50} of the material should be greater than 2 and less than 4.

3. The length of the energy dissipating pool is $10(h_s)$ or $3W_o$ which ever is larger. The overall length of the basin is $15(h_s)$ or $4W_o$ which ever is larger.

The following design procedure is provided:

1. Estimate the flow properties at the brink of the culvert. Establish the brink invert elevation such that $TW/yo \leq 0.75$ for design discharge.

2. For subcritical flow conditions (culvert set on mild or horizontal slope) utilize Figures 10.C.1 or 10.C.2 to obtain y_o/D, then obtain V_o by dividing Q by the wetted area associated with y_o. D is the height of a box culvert or pipe diameter. If the culvert is on a steep slope, V_o will be the normal velocity obtained by using the Manning equation for appropriate slope, section, and discharge.

3. From site inspection and from field experience in the area, determine whether or not channel protection is required at the culvert outlet.

4. If channel protection is required, compute the Froude number for brink conditions (ye = (A/2)1/2). Select d50/ye appropriate for locally available riprap (usually the most satisfactory results will be obtained if $0.25<d_{50}/y_e<0.45$). Obtain hs/ye from Figure 10.C.4, and check to see that 2<hs/d50<4. Recycle computations if hs/d50 falls out of this range.

5. Size basin as shown in Figure 10.C.5.

6. Design procedures where allowable dissipator exit velocity is specified:

   a. Determine the average normal flow depth in the natural channel for the design discharge.

   b. Extended the length of the energy basin (if necessary) so that the width of the energy basin at section A-A, Figures 10.C.5, times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

7. In the exit region of the basin, the walls and apron of the basin should be warped (or transitioned) so that the cross section of the basin at the exit conforms to the cross section of the natural channel. Abrupt transition of surfaces should be avoided to minimize separation zones and resultant eddies.

8. If high tailwater is a possibility and erosion protection is necessary for the downstream channel, the following design procedure is suggested:

Design a conventional basin for low tailwater conditions in accordance with the instructions above. Estimate centerline velocity at a series of downstream cross sections using the information shown in Figure 10.C.3. Shape downstream channel and size riprap using Figure 10.C.7 and the stream velocities obtained above.

<u>Design Example No. 1</u>

Given: 8 ft by 6 ft box culvert, Q = 800 cfs, super-critical flow in culvert, normal flow depth = brink depth y_o = 4 ft, tailwater depth TW = 2.8 ft.

Find: Riprap basin dimensions for these conditions:

Solution:

1. $y_o = y_e$ for rectangular section, $y_e = 4$ ft
2. $V_o = Q/A = 800/(4)(8) = 25$ fps
3. $F = V_o/[(32.2)(y_e)]^{1/2} = 25/[(32.2)(4)]^{1/2} = 2.20$
4. $TW/y_e = 2.8/4.0 = 0.7$ $TW/y_e<0.75$ O.K.
5. Try $d_{50}/y_e = 0.45$, $d_{50} = (0.45)(4) = 1.80$ ft

   From Figure 10.C.4, $h_s/y_e = 1.6$

   $h_s = (4)(1.6) = 6.4$ ft

   $h_s/d_{50} = 6.4/1.8 = 3.6$ ft, $2<h_s/d_{50}<4$ O.K.
6. $L_s = (10)(6.4) = 64$ ft

   L_s min $= (3)(W_o) = (3)(8) = 24$ ft, use $L_s = 64$ ft

   $L_B = (15)(6.4) = 96$ ft

   L_B = min $(4)(W_o) = (4)(8) = 32$ ft, use $L_B = 96$ ft

Other basin dimensions designed in accordance with details shown in Figures 10.C.5.

Design Example No. 2

Given: 8 ft by 6 ft box culvert, Q = 800 cfs, supercritical flow in culvert, normal flow depth = brink depth, y_o = 4 ft, tailwater depth TW = 4.2 ft, downstream channel can tolerate 7 fps for design discharge.

Find: Riprap basin dimensions for these conditions.

Solution:

Note: High tailwater depth, $TW/y_o = 1.05>0.75$

1. Design riprap basin (Design Example 1) use steps 1-7 d_{50} = 1.8 ft, h_s = 6.4 ft, L_s = 64 ft, L_B = 96 ft.

2. Design riprap for downstream channel. Utilize Figure 10-C.3 for estimating average velocity along the channel. Compute equivalent circular diameter D_e for brink area from:

$$A = D_e^2/4 = (y_o)(W_o) = (4)(8) = 32 \text{ ft}^2$$

$$D_e = [32(4)/\quad]^{1/2}$$

$$D_e = 6.4 \text{ ft}$$

$$V_o = 25 \text{ fps (Design Example 1)}$$

L/De	L	V_L/V_o	V_L	Rock Size d_{50}
	(Compute)	(Figure 10-C.3)	ft/sec	Figure 10-C.7
10	64	0.59	14.7	1.4
15	96	0.36	9.0	0.6
20	128	0.30	7.5	0.4
21	135	0.28	7.0	0.4

Riprap should be at least the size shown. As a practical consideration, the channel can be lined with the same size rock used for the basin. Protection must extend at least 135 feet downstream from the culvert brink.

Design Example No. 3

Given: 6 feet diameter cmp, $Q = 135$ cfs, $S_o = 0.004$, Mannings $n = 0.024$ normal depth in pipe for $Q = 135$ cfs is 4.5 feet, normal velocity is 5.9 fps, flow is subcritical, tailwater depth (TW) is 2.0 feet.

Find: Riprap basin dimensions for these conditions:

Solution:

1. Determine y_o and V_o :

   $Q/D^{2.5} = 135/(6)^{2.5} = 1.53$

   $TW/D = 2.0/6 = 0.33$

   From Figure 10-C.1, $y_o/D = 0.45$

   $y_o = (0.45)(6) = 2.7$ ft

   $TW/y_o = 2.0/2.70 = 0.74$ $TW/y_o<0.75$ O.K.

   Brink Area (A) for $y_o/D = 0.45$ is

   $A = (0.343)(36) = 12.3$ sq ft
   (0.343 is from Table 8-1)

   $V_o = Q/A = 135/12.3 = 11.0$ fps

2. $y_e = (A/2)^{1/2} = (12.3/2)^{1/2} = 2.48$ ft

3. $F = V_o/[(32.2)(y_e)]^{1/2} = 11/[(32.2)(2.48)]^{1/2} = 1.23$

4. Try $d_{50}/y_e = 0.25$, $d_{50} = (0.25)(2.48) = 0.62$ ft

   From Figure 10-C.4, $h_s/y_e = 0.75$, $h_s = (0.75)(2.48)$

   $= 1.86$ ft
   check: $h_s/d_{50} = 1.86/0.62 = 3$, $2<h_s/d_{50}<4$ O.K.

5. $L_s = (10)(h_s) = (10)(1.86) = 18.6$ ft

   or $L_s = (3)(W_o) = (3)(6) = 18$ ft, Use $L_s = 18.6$ ft

   $L_B = (15)(h_s) = (15)(1.86) = 27.9$ ft

   or $L_B = (4)(W_o) = (4)(6) = 24$ ft, Use $L_B = 27.9$ ft

   $d_{50} = 0.62$ ft use $d_{50} = 8$ in

Other basin dimensions are designed in accordance with details shown on Figure 10-C.5.

It is recognized that there is some chance of limited degradation of the floor of the dissipator pool for rare event discharges. With the protection afforded by the $2(d_{50})$ thickness of riprap by the heavy layer of riprap adjacent to the roadway prism, and the apron riprap in the downstream portion of the basin, the damage should be superficial.

Concerning the use of filter material, several factors should be considered. Bank material adjacent to a culvert is not subjected to flow for long continuous periods. Also, the streambed material may be sufficiently well graded and not require a filter. If some siltation of the basin accompanied by plant growth is anticipated, it may be that a filter will not be required. If required, a filter cloth or filter material should be designed.

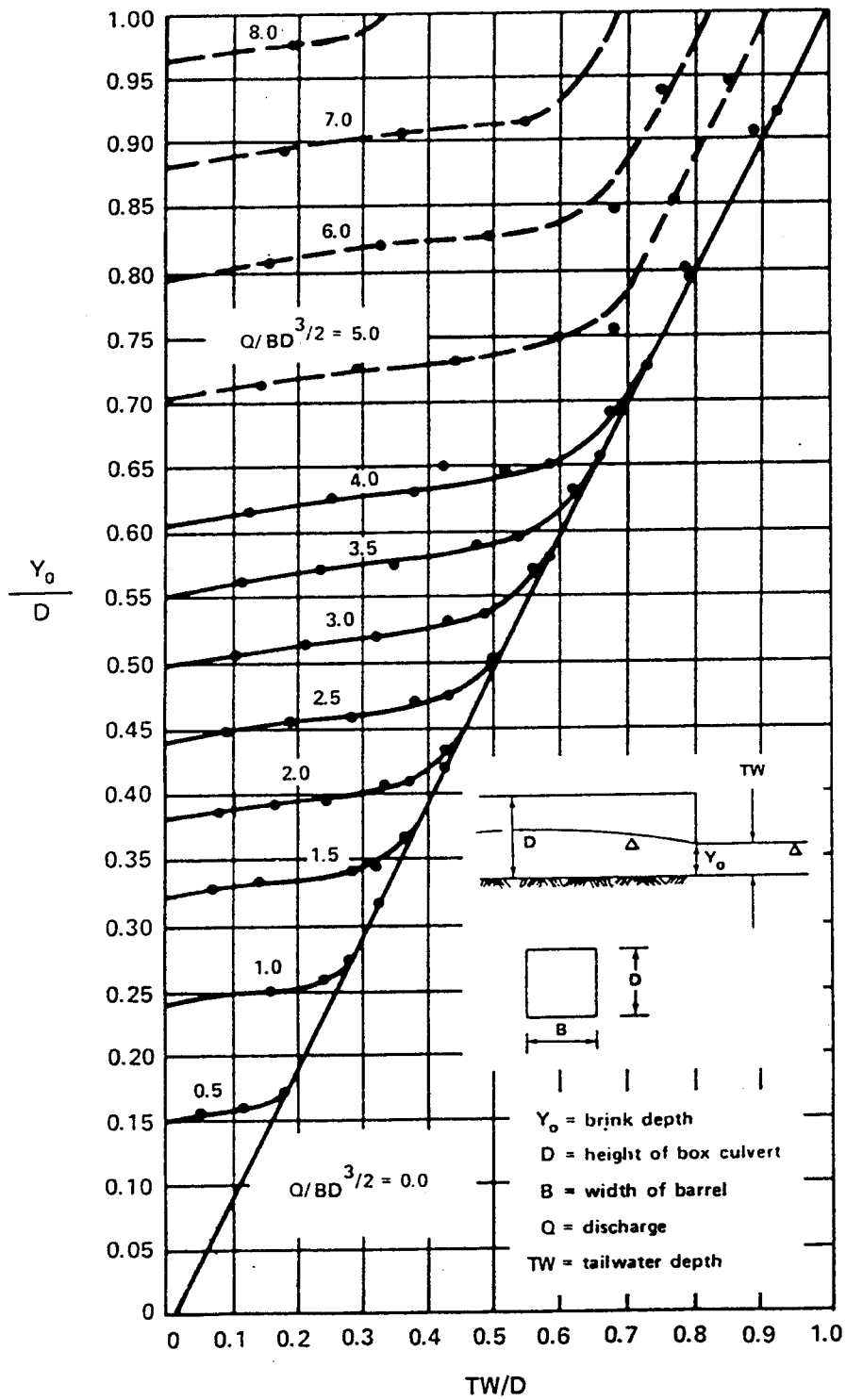


Figure 10-C.1 Dimensionless Rating Curves for the Outlets of Rectangulars Culverts on Horizontal and Mild Slopes .

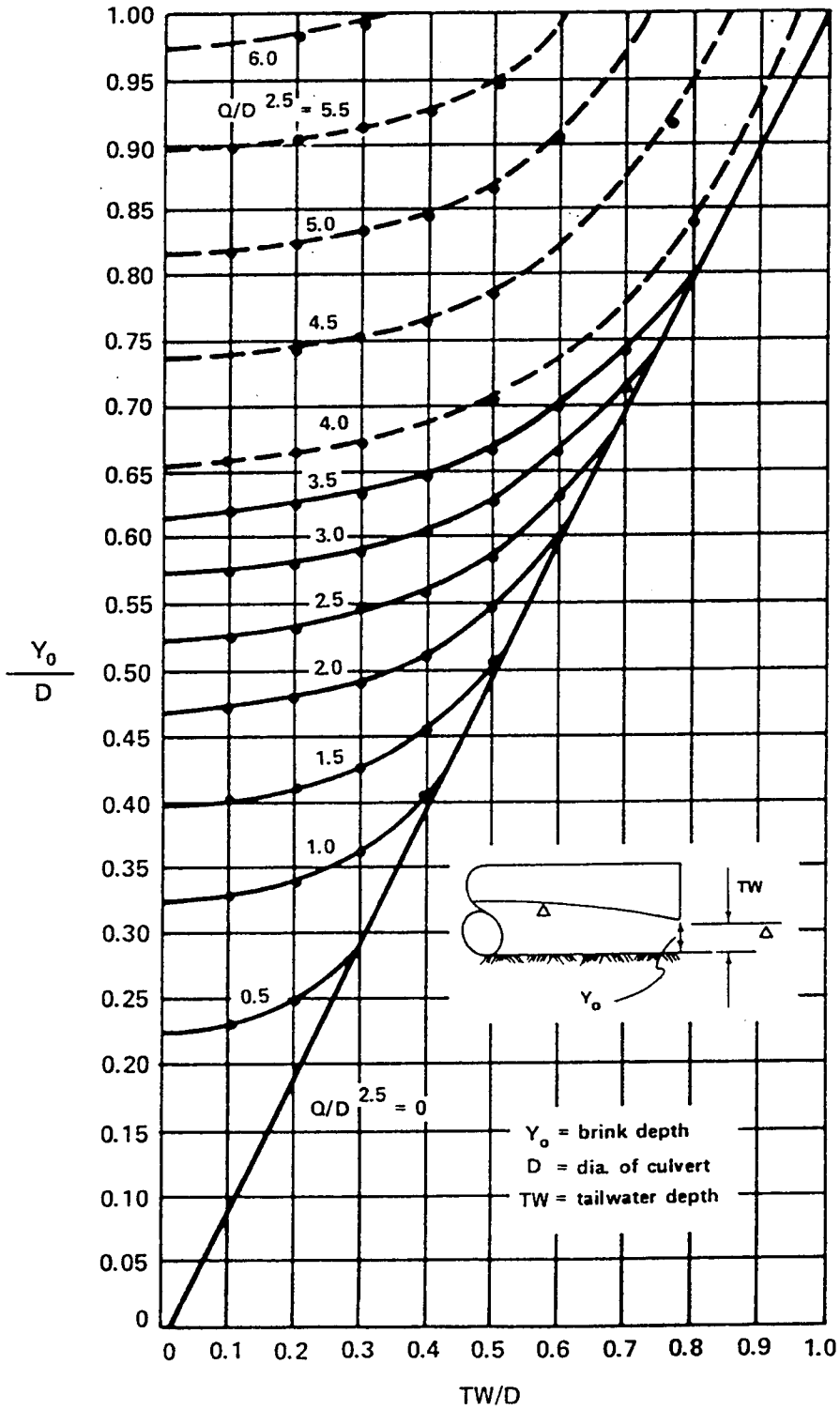
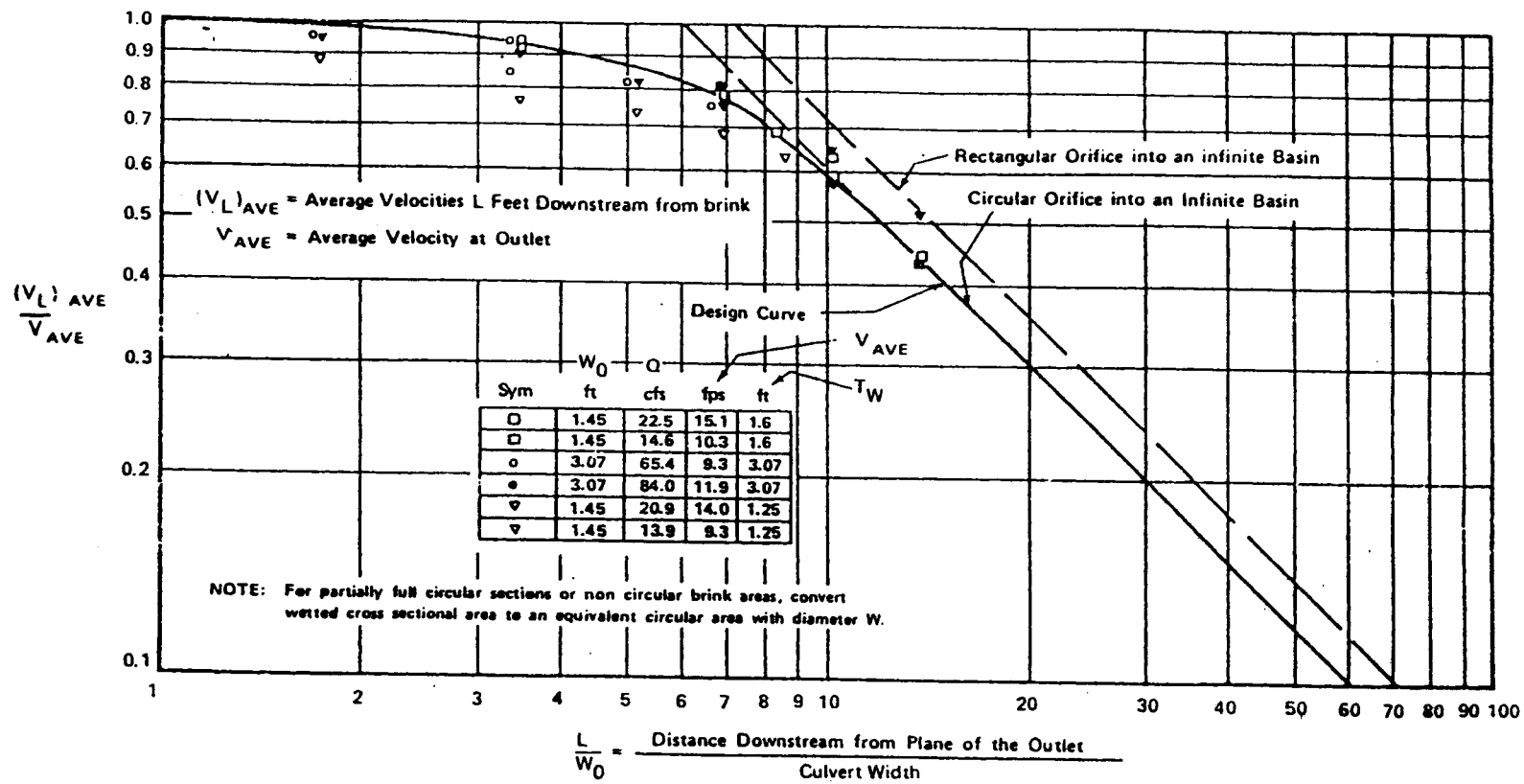


Figure 10-C.2 Dimensionless Rating Curve for the Outlets of Circular Culverts on Horizontal and Mild Slopes



Sym	W_0 ft	Q cfs	fps	ft
□	1.45	22.5	15.1	1.6
□	1.45	14.6	10.3	1.6
○	3.07	65.4	9.3	3.07
●	3.07	84.0	11.9	3.07
▽	1.45	20.9	14.0	1.25
▽	1.45	13.9	9.3	1.25

Figure 10-C.3 Distribution of Centerline Velocity for Flow from Submerged Outlets to be used for Predicting Channel Velocities Downstream from Culvert Outlet where High Tailwater prevails. Velocities obtained from the use of this Chart can be used with Figure 2 of HEC No. 11 for sizing riprap (DO NOT use Figure 1 HEC No. 11, use Mean Velocity Values).

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Figure 10-C.3

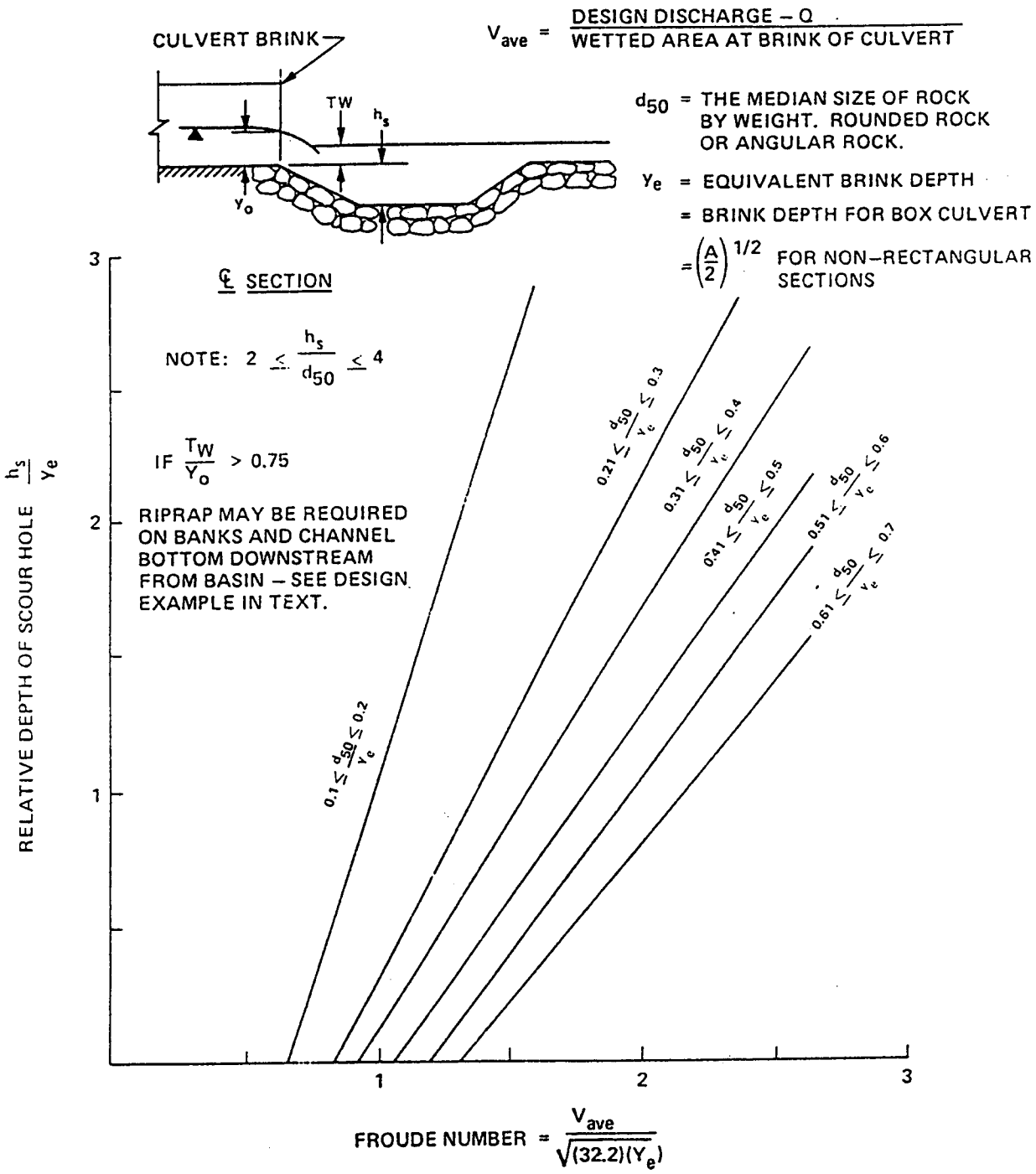
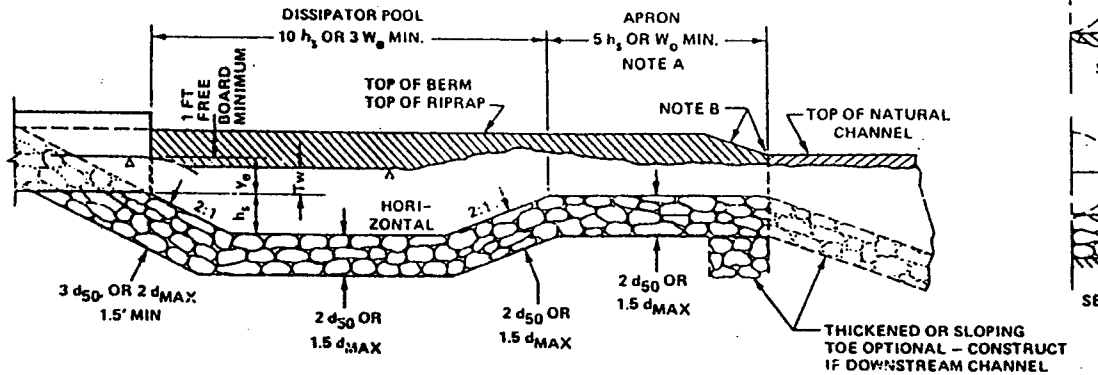


FIGURE 10-C.4 RELATIVE DEPTH OF SCOUR HOLE VERSUS FROUDE NUMBER AT BRINK OF CULVERT WITH RELATIVE SIZE OF RIPRAP AS A THIRD VARIABLE

NOTE A – IF EXIT VELOCITY OF BASIN IS SPECIFIED, EXTEND BASIN AS REQUIRED TO OBTAIN SUFFICIENT CROSS-SECTIONAL AREA AT SECTION A–A SUCH THAT Q_{des}/(CROSS SECTION AREA AT SEC. A–A) = SPECIFIED EXIT VELOCITY.

NOTE B – WARP BASIN TO CONFORM TO NATURAL STREAM CHANNEL. TOP OF RIPRAP IN FLOOR OF BASIN SHOULD BE AT THE SAME ELEVATION OR LOWER THAN NATURAL CHANNEL BOTTOM AT SEC. A–A.



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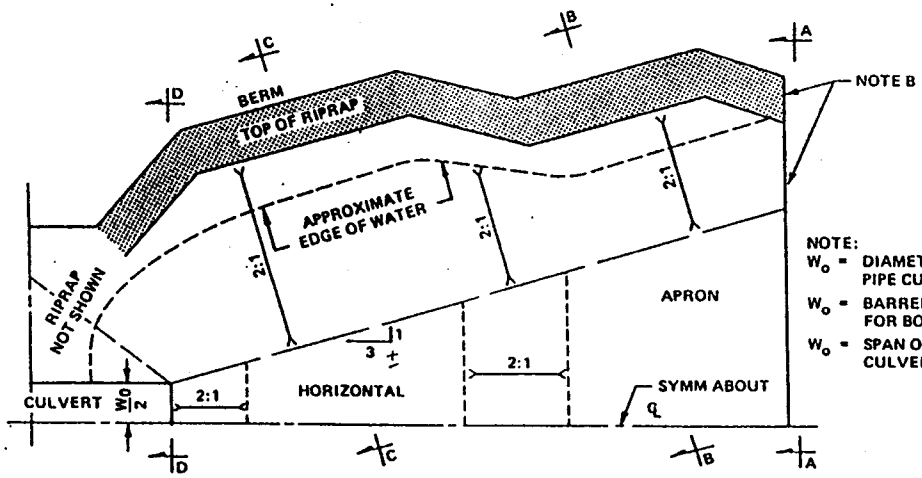
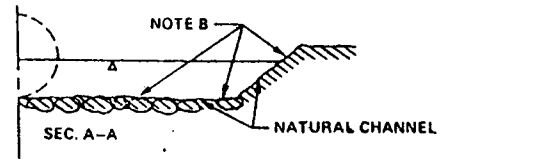
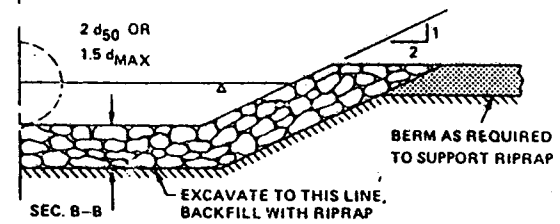


HALF PLAN

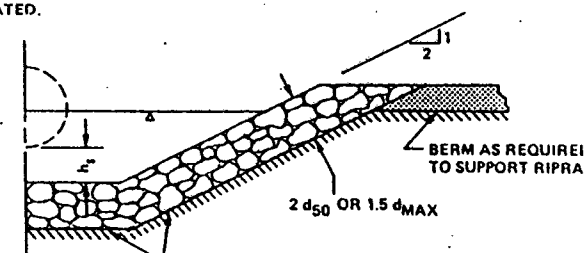
NOTE:
W_o = DIAMETER FOR PIPE CULVERT
W_o = BARREL WIDTH FOR BOX CULVERT
W_o = SPAN OF PIPE-ARCH CULVERT



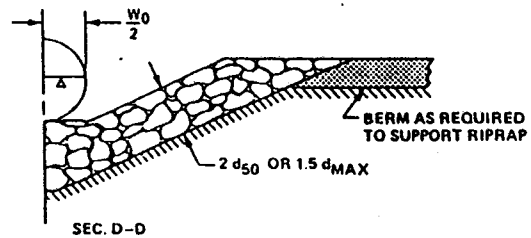
SEC. A–A



SEC. B–B

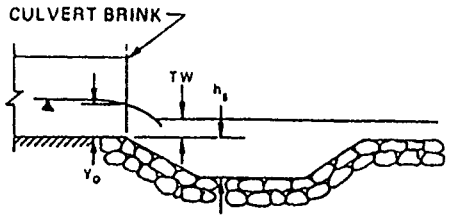


SEC. C–C



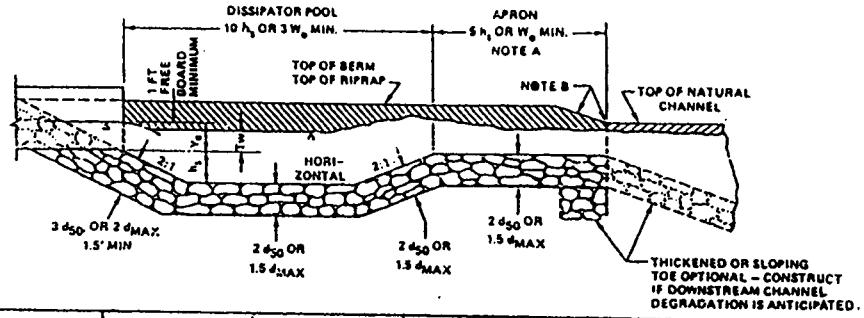
SEC. D–D

The City of Colorado Springs / El Paso County Drainage Criteria Manual Details of Riprapped Culvert Energy Basin	Date 9-30-90 Figure 10-C.5



NOTE A – IF EXIT VELOCITY OF BASIN IS SPECIFIED, EXTEND BASIN AS REQUIRED TO OBTAIN SUFFICIENT CROSS-SECTIONAL AREA AT SECTION A–A SUCH THAT Q_{des}/(CROSS SECTION AREA AT SEC. A–A) = SPECIFIED EXIT VELOCITY.

NOTE B – WARP BASIN TO CONFORM TO NATURAL STREAM CHANNEL. TOP OF RIPRAP IN FLOOR OF BASIN SHOULD BE AT THE SAME ELEVATION OR LOWER THAN NATURAL CHANNEL BOTTOM AT SEC. A–A.



	TW	y_e	(1) TW/y_e	d_{50}/y_e	d_{50}	h_s/y_e	h_s	(2) h_s/d_{50}	
LOW TW $TW/y_e \le 0.75$									
	V ALLOWABLE	L/D_e (3)	L	V_{ave}/V_L	V_L				
HIGH TW $TW/y_e > 0.75$									

		Larger of		
Length of Pool	=	10 h_s or $3W_o$	=	_____ ft.
Length of Apron	=	5 h_s or W_o	=	_____ ft.
Thickness of Approach	=	$3d_{50}$ or $2d_{max}$	=	_____ ft.
Thickness of Remainder of Basin	=	$2d_{50}$ or $1.5d_{max}$	=	_____ ft.

(1) $TW/y_e \le 0.75$ for Low TW Design

(2) $2 < hs/d_{50} < 4$

(3) $D_e = [4A/\pi]^{1/2}$

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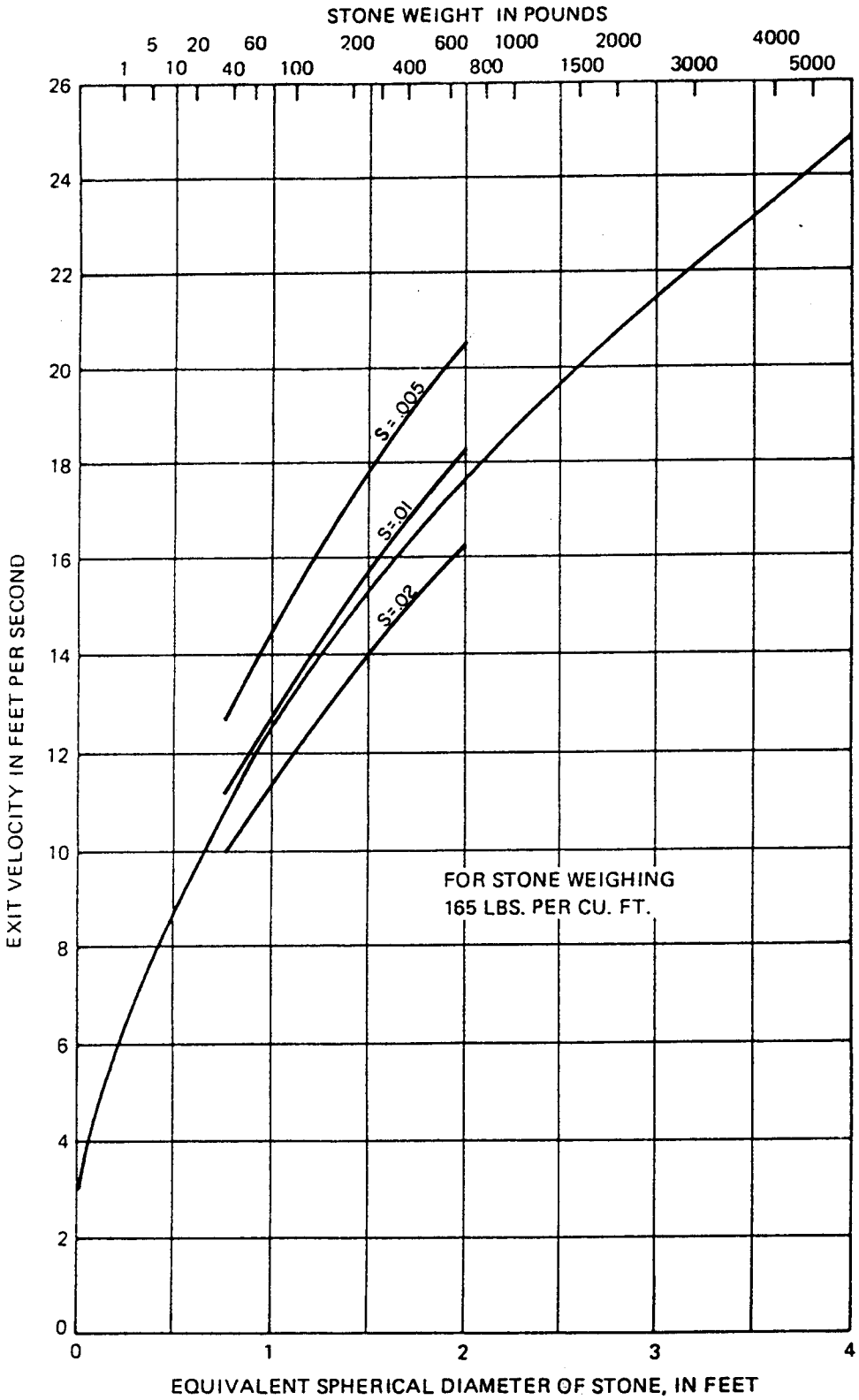
Riprap Basins Design Chart

Date 9-30-90

Figure 10-C.6

SPECIFIC GRAVITY = 2.64



NOTE: FOR LONGITUDINAL SLOPES GREATER THAN 1.1% METHOD DESCRIBED IN 10.10.2 AND 10.10.3 SHOULD BE UTILIZED

The City of Colorado Springs / El Paso County Drainage Criteria Manual	Date 9-30-90
Riprap Size for use Downstream of Energy Dissipators	Figure 10-C.7

10.8.4 Drop Structures - General

The function of drop structures is to convey water from a higher to a lower elevation (i.e., grade control) and to dissipate excess energy resulting from its fall. A channel located in this same terrain would ordinarily be steep enough to cause severe erosion in earth channels or disruptive flow in lined channels. The water can therefore be conveyed through a drop structure designed to safely dissipate the excess energy. Different kinds of drop structures that may be used are vertical, rectangular inclined, pipe drops, baffle chute and sloped drops.

Due to the energy dissipating features of drop structures, these structures must be properly designed and constructed to withstand the dynamic, hydrostatic and erosive forces encountered. Consideration should be given to the maximum design storm event as well as the more frequent lower duration storms in relation to the long term performance of the structure.

A proper analysis of the foundation and adjacent soil conditions at each drop structure location including a thorough hydraulic analysis is essential for the installation and the reliable functioning of the structures.

The design, construction and performance of drop structures installed in unlined or partially lined channels is still under study and subject to modifications in theory and construction techniques. Reference is made to the evaluation and design recommendations for drop structures prepared for the Denver Urban Drainage and Flood Control District. (Reference 14)

10.8.5 Vertical Drops

Vertical drops are often the most economical for drops of less than four (4) feet. They can consist of a simple weir above a vertical retaining wall and a splash-pool-type energy dissipator that are combined in a single structure. These structures can be constructed from steel sheet piling, or reinforced concrete. Typical vertical drops are shown on Figures 10-9a, 10-9b and 10-10.

Vertical drops constructed entirely from reinforced concrete require a minimum of maintenance and generally are self cleaning as compared with vertical drops having riprap basins. Public safety may be more of a concern with vertical drops and handrails and other safety features should be considered along the sides of the drop.

As stated in Reference 7, changing the channel slope from steep to mild, by placing drop structures at intervals along the channel reach, changes a continuous steep slope into a series of gentle slopes and vertical drops. Instead of slowing down and transferring high erosion producing velocities into low

non-erosive velocities, drop structures control the slope of the channel in such a way that the high, erosive velocities never develop. The kinetic energy or velocity gained by the water as it drops over the crest of each structure is dissipated by a specially designed apron or stilling basin.

The drop structures discussed here require aerated nappes and are, in general, for subcritical flow in the upstream as well as downstream channel. The stilling basin protects the channel against erosion below the drop and dissipates energy. This is accomplished through the impact of the falling water on the floor, redirection of the flow and turbulence. The stilling basin used to dissipate the excess energy can vary from a simple concrete apron to an apron with flow obstructions such as baffle blocks, sills, or abrupt rises. The length of the concrete apron required can be shortened by the addition of these appurtenances.

The flow geometry at straight drop structures, Figure 10.D.1, can be described by the drop number, defined:

$$N_d = q^2/gh_o^3$$

Where q is the discharge per unit width of the crest overfall, g is the acceleration of gravity and h_o is the height of the drop. The functions are:

$$L_1/h_o = 4.30\ N_d^{0.27}$$

$$y_1/h_o = 1.00\ N_d^{0.22}$$

$$y_2/h_o = 0.54\ N_d^{0.425} \text{ and}$$

$$y_3/h_o = 1.66\ N_d^{0.27}$$

Where L_1, the drop length is the distance from the drop wall to the position of the depth y_2; y_1 is the pool depth under the nappe; y_2 is the depth of flow at the toe of the nappe or the beginning of the hydraulic jump; and y_3 is the tailwater depth sequent to y_2, Figure 10-D.1.

The free-falling nappe reverses its curvature and turns smoothly into supercritical flow on the apron at the distance L_1 from the drop wall. The mean velocity at the distance L_1 is parallel to the apron; the depth y_2 is the smallest depth in the downstream channel and the pressure is nearly hydrostatic. The depth of supercritical flow in the downstream direction increases due to channel resistance and at some point will reach a depth sufficient for the formation of the hydraulic jump.

For a given drop height h_o, and discharge q, the sequent depth y_3, in the downstream channel and the drop length L_1 may be computed. By comparing the channel tailwater depth TW with the computed y_3, the flow type (TW less than y_3, TW = y_3 or TW greater than y_3) can be determined. The flow type determines the design of the stilling basin for the drop structure.

If the tailwater depth TW is less than y_3, the hydraulic jump will recede downstream. If the tailwater depth is greater than y_3, the hydraulic jump will be submerged. If TW is equal to y_3, the hydraulic jump begins at depth y_2 (Figure 10.D.1), no super-critical flow exists on the apron and the distance L_1 is a minimum.

When the tailwater depth TW is less than y_3, it is necessary to provide:

1. an apron at the bed level and end sill or baffles.

2. an apron below the downstream bed level, and an end sill.

The choice of design type and the design dimensions will, for a given unit discharge (q), depend on the drop height (h_o) and on the downstream depth (TW).

The apron may be designed to extend to the end of the hydraulic jump. However, including an end sill allows the use of a shorter and more economical stilling basin.

The geometry of the undisturbed flow should be taken into consideration in the design of a straight drop stilling basin. If the overfall crest length is less than the width of the approach channel, it is important that a transition be properly designated by shaping the approach channel to reduce the effect of end contractions. Otherwise the contraction at the ends of the spillway notch may be so pronounced that the jet will land beyond the stilling-basin and the concentration of high velocities at the center of the outlet may cause additional scour in the downstream channel.

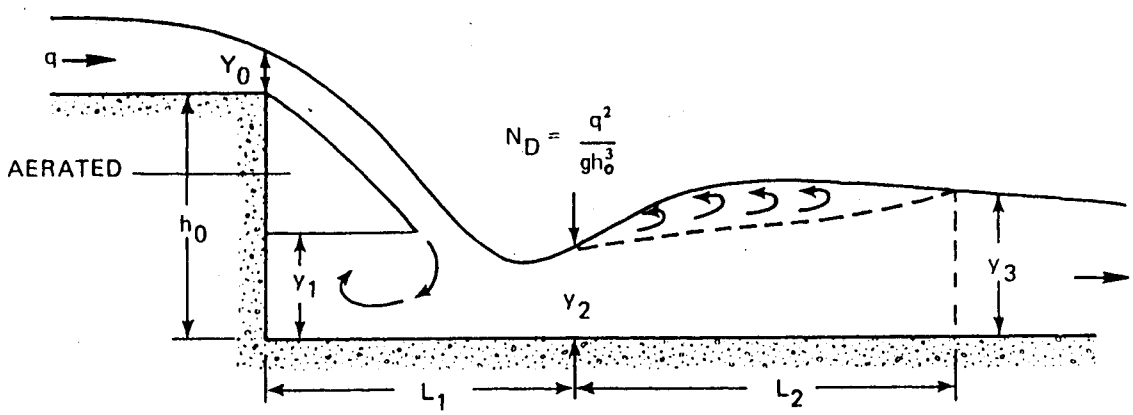


FIGURE 10-D.1 FLOW GEOMETRY OF A STRAIGHT DROP SPILLWAY

A general design for a stilling basin at the toe of a drop structure was developed by the Agricultural Research Service, St. Anthony Falls Hydraulic Laboratory, University of Minnesota. The basin consists of a horizontal apron with blocks and sills to dissipate energy. Tailwater also influences the amount of energy dissipated. The stilling basin length computed for the minimum tailwater level required for good performance may be inadequate at high tailwater levels. Dangerous scour of the downstream channel may occur if the nappe is supported sufficiently by high tailwater so that it lands beyond the end of the stilling basin. A method for computing the stilling basin length for all tailwater levels is presented.

The design is applicable to relative heights of fall ranging from $1.0(h_o/y_c)$ to $15(h_o/y_c)$ and to crest lengths greater than $1.5y_c$. Here h_o is the vertical distance between the crest and the stilling basin floor, and yc is the critical depth of flow at the crest. The straight drop structure is effective if the drop does not exceed 15 feet and if there is sufficient tailwater.

There are several elements which must be considered in the design of this stilling basin. These include the length of basin, the position and size of floor blocks, the position and height of end sill, the position of the wingwalls, and the approach channel geometry. Figure 10-D.3 illustrates a straight drop structure which provides adequate protection from scour in the downstream channel.

Design Procedure

1. Calculate the specific head in approach channel.

$$H = y_o + \frac{Vo^2}{2g} \quad \text{. . . . . 10-A-1}$$

2. Calculate critical depth.

$$y_c = \frac{2}{3} H \quad \text{. . . . . 10-A-2}$$

3. Calculate the minimum height for tailwater surface above the floor of the basin.

$$y_3 = 2.15\ y_c \quad \text{. . . . . 10-A-3}$$

4. Calculate the vertical distance of tailwater below the crest. This will generally be a negative value since the crest is used as a reference point.

$$h_2 = -\ (h - y_o) \quad \text{. . . . . 10-A-4}$$

5. Determine the location of the stilling basin floor relative to the crest.

$$h_o = h_2 - y_3 \quad \text{. . . . . 10-A-5}$$

6. Determine the minimum length of the stilling basin, L_B, using:

$$L_B = L_1 + L_2 + L_3 = L_1 + 2.55y_c \quad \ldots\ldots\ldots \quad 10\text{-A-}6$$

where L_1 is the distance from the headwall to the point where the surface of the upper nappe strikes the stilling basin floor. This given by:

$$L_1 = (L_f + L_s)/2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 10\text{-A-}7$$

where

$$L_f = (-0.406 + \sqrt{3.195 - 4.368h_o/y_c})y_c$$

$$L_s = [0.691 + 0.228(L_t/y_c)2 - (h_o/y_c)]y_c/[0.185 + 0.456(L_t/y_c)]$$

$$L_t = (-0.406 + \sqrt{3.195 - 4.368\ h_2/y_c})y_c$$

OR L_1 can be found graphically from Figure 10-D.2.

L_2 is the distance from the point at which the surface of the upper nappe strikes the stilling basin floor to the upstream face of the floor blocks, Figure 10-D.3. This distance can be determined by:

$$L_2 = 0.8y_c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 10\text{-A-}8$$

L_3 is the distance between the upstream face of the floor blocks and the end of the stilling basin. This distance can be determined from:

$$L_3 > 1.75y_c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 10\text{-A-}9$$

7. Proportion the floor blocks as follows:

   a. height is 0.8 y_c,

   b. width and spacing should be 0.4 y_c with a variation of $\pm$ 0.15 y_c permitted,

   c. blocks should be square in plan, and

   d. blocks should occupy between 50 percent and 60 percent of the stilling basin width.

8. Calculate the end sill height, (0.4 y_c).

9. Longitudinal sills, if used, should pass through, not between, the floor blocks. These sills are for structural purposes and are neither beneficial nor harmful hydraulically.

10. Calculate the sidewall height above the tailwater level, ($0.85\ y_c$).

11. Wingwalls should be located at an angle of 45° with the outlet centerline and have a top slope of 1 to 1.

12. Modify the approach channel as follows:

    a. crest of spillway should be at same elevation as approach channel,

    b. bottom width should be equal to the spillway notch length, W_o at the headwall, and

    c. protect with riprap or paving for a distance upstream from the headwall equal to three times the critical depth, y_c.

13. No special provision of aeration of the space beneath the nappe is required if the approach channel geometry is as recommended in 12 above.

Design Example

Given: $Q = 250$ cfs
$S_o = 0.002$ ft/ft, and the downstream channel has 3:1 side slopes with a 10 foot bottom.

$n = 0.03$

Normal depth of flow, $y_o = 3.36$ ft

Normal velocity, $V_o = 3.7$ fps

Vertical drop, $h = 6$ ft

Find: Straight drop structure dimensions

Solution:

1. $$H = 3.36 \text{ ft} + \frac{(3.7 \text{ fps})^2}{2 \times 32.2 \text{ ft/sec}^2} = 3.57 \text{ ft}$$

2. $y_c = (2/3)(3.57 \text{ ft}) = 2.38$ ft

3. $y_3 = 2.15(2.38 \text{ ft}) = 5.12$ ft

4. $h_2 = -(6.0 - 3.36) = -2.64$ ft

5. $h_o = -2.64 - 5.12 = -7.76$ ft

The floor of the stilling basin is, therefore, 1.76 ft below the grade line of the downstream channel.

6. $h_o/y_c = -7.76/2.38 = -3.26$
   $h_2/y_c = -2.64/2.38 = -1.11$

   From Figure 10-D.2

   $L_1/y_c = 3.95$
   $L_1 = 9.40$ ft
   $L_2 = 0.8\ y_c = (0.8)(2.38) = 1.90$ ft
   $L_3 \geq 1.75\ y_c = 1.75(2.38) = 4.17$ ft or 4.20 ft
   $L_B = 9.40 + 1.90 + 4.20 = 15.50$ ft

7. Proportion floor blocks

   a. Height = $0.8\ y_c(2.38) = 1.90$ ft

   b. Width = $0.4\ y_c = 0.4(2.38) = 0.95$ ft
      Spacing = $0.4\ y_c = 0.4(2.38) = 0.95$ ft

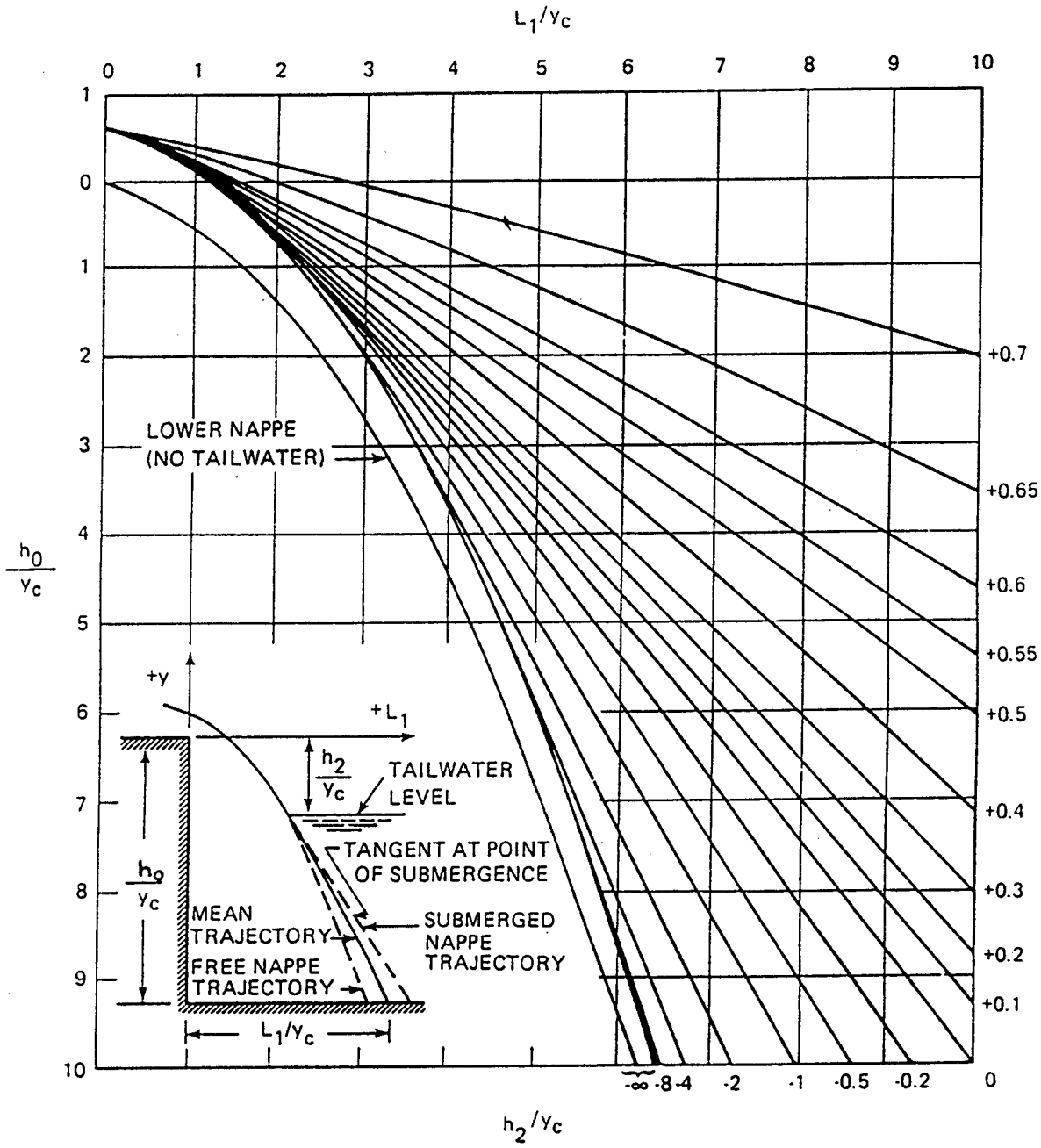
8. Calculate end sill height = $0.4y_c = 0.4(2.38) = 0.95$ ft

9. Use longitudinal sills passing through the floor blocks

10. Calculate sidewall height above tailwater
    $= 0.85\ y_c = 0.85(2.38) = 1.90$ ft

11. Locate wingwalls at 45° angle with outlet centerline.

12. Protect approach channel with riprap or paving for 7.14 ft (3 x 2.38) upstream of the headwall.



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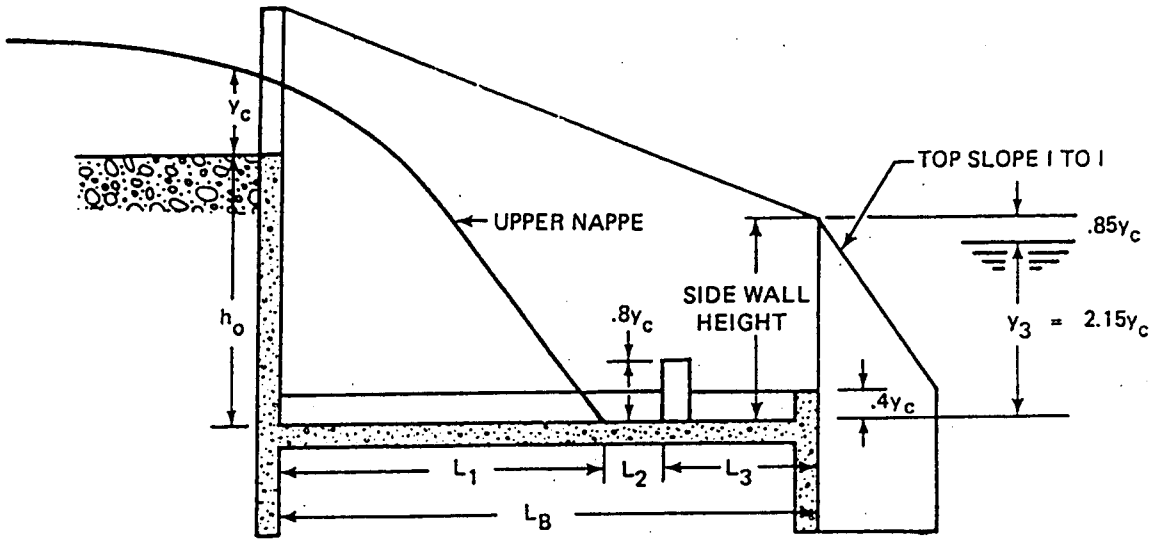
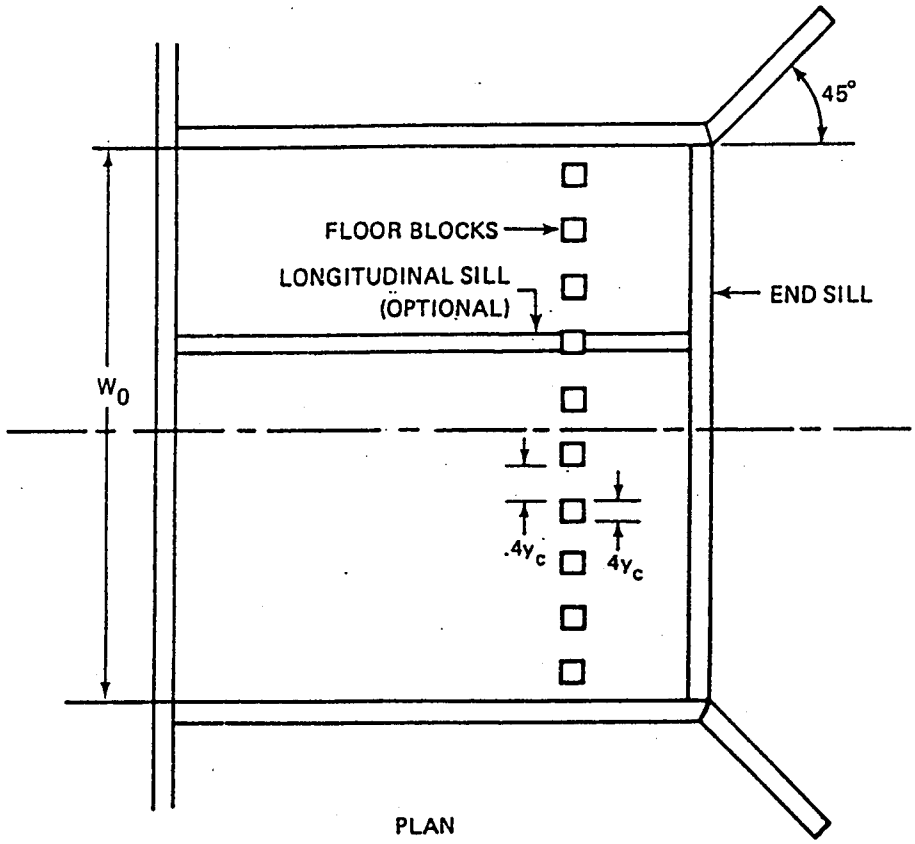
Design Chart for Determination of L_1

Date
9-30-90

Figure
10-D.2

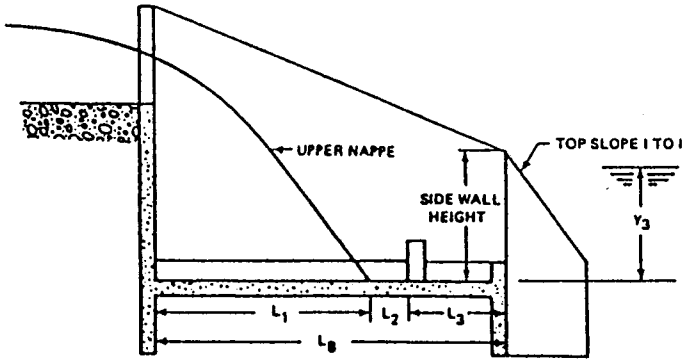


SECTION AT CENTER LINE



PLAN

The City of Colorado Springs / El Paso County Drainage Criteria Manual	Date 9-30-90
Straight Drop Spillway Stilling Basin	Figure 10-D.3



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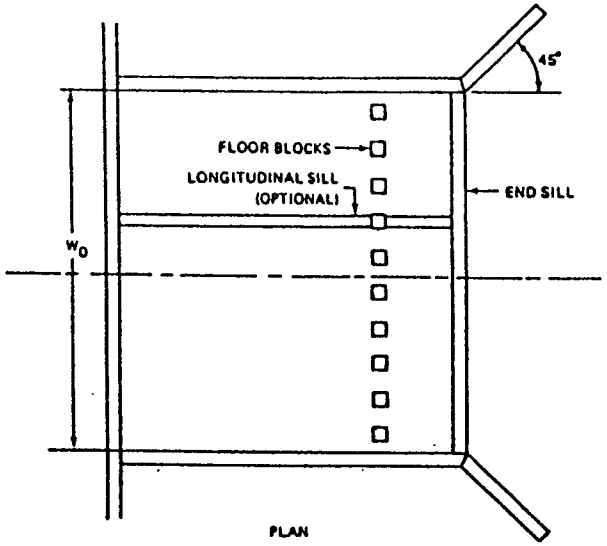

PLAN

	Natural S_o	New S_o	Normal Depth Upstream	Normal Depth Downstream	y_c	W_o
INITIAL DATA						

	L_1	L_2	L_3	L_B	Floor Blocks Height	Floor Blocks Width	Spacing (1)	End Sill Height	Sidewall Height
DISSIPATOR DIMENSIONS									

	y_3 min

(1) Block should occupy 50 to 60 percent of Basin Width.

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Straight Drop Structure Design Chart

Date 9-30-90

Figure 10-D.4

10.8.6 Rectangular Inclined and Pipe Drops

Rectangular inclined drops and pipe drops are used where the decrease in elevation is in the range of 3 to 15 feet over a relatively short distance. Usually a pipe drop will be selected for smaller flows and a rectangular inclined drop will be selected for larger flows. If the drop crosses another waterway or a roadway, it will probably be more economical to use a pipe drop.

10.8.7 Baffle Chute

The Baffle Chute structure (Figure 10-13) is commonly used where the drop in elevation is greater than 15 feet. The decision as to whether to use a chute structure or a series of smaller drops should be based upon a hydraulic and economic study of the two alternatives. Drops should not be so closely spaced as to possibly preclude uniform flow between outlet and inlet ends of consecutive structures, particularly where checks or control notches are not used at the inlets. The danger is that sufficient tailwater depths may not exist to produce hydraulic jumps in the pools, and thus "shooting flow" may develop through the series of drops and possibly damage the channel. Also, with drops too closely spaced on a steep slope, problems of excavation and backfill may make such construction undesirable or prohibitive. About 200 feet of channel should be the minimum distance between the inlet and outlet ends of consecutive drop structures. The economic study should compare costs of a series of drops versus a single chute structure considering advantages and disadvantages pertinent to the specific conditions. Since the maintenance costs for a series of drops is usually considerably more than for a single chute structure that would perform the same hydraulic function, it is sometimes economically justifiable to spend considerably more in initial costs for a chute structure than for a series of drops. More complete discussions on chute structures are presented in References 1, 2, and 4.

10.8.8 Sloped Drops

Sloped drop structures are primarily constructed of reinforced concrete, soil cement, riprap or a combination of these materials. Typical sloped drop structures are shown on Figures 10-14.

Drop structures constructed of loose riprap are generally more difficult to maintain due to the uncertainties of the supporting soils, groundwater conditions and the potential for the movement of the stone by the dynamic and erosive forces encountered. Consequently long term maintenance costs are also greater. The stone should be carefully selected for size gradation and quality. Placement of the stone should consider the proper gradation, interlocking and low void ratio. Close construction supervision is recommended. A filter material is normally

provided for loose riprap to prevent loss of the supporting subgrade material. Crestwalls and cutoff walls are recommended to contain the riprap.

Loose riprap sloped drops should be 4 feet or less in height with safe slopes between 6 to 1 and 10 to 1.

Where grouted riprap is utilized, consideration must be given to seepage control and up lift forces. Subdrains or toe drains are often considered essential. Piping under the grouted rock should be prevented and granular filter material is normally not used with placement of the stone.

Proper placement and grouting of the stone should be specified to obtain grout penetration to the subgrade. Low pressure grouting equipment and "pencil" vibrations are recommended.

Grouted riprap sloped drops should be 4 feet in height or less but can be up to 12 feet in height if additional soil testing and seepage control is incorporated into the design and construction.

10.8.9 Ditch Checks

Ditch checks should be provided only in roadside ditches when slopes and flow velocities exceed the criteria outlined in Tables 10-3 and 10-4. Ditch checks may be constructed of railroad ties, riprap, grouted riprap, or reinforced concrete. Some typical structures for ditches are shown on Figures 10-17 and 10-18.

10.9 Flow Transitions

A flow transition structure is a change of channel cross section designed to allow for a minimum amount of flow disturbance. Several types of transitions are shown on Figure 10-19. Of these, the abrupt (headwall) and the straight line (wingwall) are the most common.

Special inlet transitions are useful when the conservation of flow energy is essential because of allowable headwater considerations. Chapter 9 includes a discussion on culvert design with improved inlets.

Outlet transitions (expansions) must be considered in the design of all culverts energy dissipators and channel protection. The standard wingwall-apron combinations and expansions downstream of dissipator basins are most common.

10.10 Riprap

Placement of riprap in channel systems is used for preventing or limiting channel bed and bank erosion damage caused by excessive channel flow or surges from a stilling basin. In particular, placement of riprap on the channel bottom and banks downstream of an energy dissipator structure is required for alleviating possible undermining of the structure.

Experience has shown that a primary reason for riprap failure is placement of undersized individual stones in the maximum size range. Failure has also occurred because of improper engineering design for gradation of riprap, seepage control and/or bedding filter requirements.

It is recommended that the stone selection and proper gradation be determined by the engineer and contractor at the quarry prior to delivery at the construction site. It is also recommended that a sample riprap placement section be installed by the contractor to field verify conformance with the specification prior to the complete placement of the project work.

Design of riprap should take into account the following parameters:

- Stone durability
- Stone density
- Stone size
- Stone shape
- Stone gradation
- Velocity of flow against the stone
- Filter bed requirements
- Channel side slopes
- Froude number

10.10.1 Quality

Riprap shall consist of field stone or rough, unhewn quarry stone predominantly angular in shape. The stone shall be hard and of such quality that it will not disintegrate on exposure to erosion or chemical weathering, and it shall be suitable in all other respects for the purpose intended. The specific gravity of the individual stones shall be at least equal to 2.50. The largest dimension of the stone shall not be greater than three (3) times the smallest dimension.

Rubble concrete may be used if approved by the City/County provided it has a density of at least 150 pounds per cubic foot and otherwise meets the requirements of applicable abrasion test standards. The concrete must be broken such that large flat slabs are eliminated. If a concrete slab is 6 inches thick, the slab would be broken up so that it is no wider than 18 inches in any direction. Any projecting reinforcing steel must be removed prior to placement.

10.10.2 Design for Subcritical Flow

The basic method recommended for use in designing riprap for subcritical flow conditions (for Froude number estimated at less than 0.9) is the Urban Drainage and Flood Control District procedure. The appropriate gradation designation ranging from very light (VL) to very heavy (VH) is based on the following relationship:

$$\frac{VS^{0.17}}{(S_s-1)^{0.66}} = 1.0$$

where V = mean channel flow velocity, in fps;

S = longitudinal channel slope, in feet per foot (ft/ft); and

S_s = specific gravity of stone (minimum S_s = 2.50)

Table 10-6 presents the riprap requirement for channel linings based on this relationship. Gradations for the various classifications are presented in Table 10-7, and Figure 10-21.

10.10.3 Design for Supercritical Flow

For supercritical flow, riprap should be designed according to the procedure described by Simons, Li & Associates, Inc. for steep slope channel design. (Reference 5) The basic premise for using this method is that for high flow velocities, it is more important to concentrate design on riprap gradation and placement than to increase maximum stone size to unrealistic limits and thereby invalidating Manning's n roughness coefficient approximations. The basic assumption for steep slope riprap is that d_{max} should be no greater than 1.25 times the median stone size, d_{50}, and that the d_{50} size should be 2-3 times greater than the 10-20 percent stone size, D_{10-20}. (See Table 10-8)

The thickness of a riprap layer should be a minimum of two (2) times the dimension of the 50 percent stone size (d_{50}) and should be placed over a layer of granular bedding, as outlined in Section 10.10.4 or a filter fabric cloth manufactured for that purpose (See Section 10.10.5). A typical riprap and bedding installation for channel bank protection is shown on Figure 10-23.

For erosion protection at conduit outlets, see Section 10.8 of this manual or Reference 7.

TABLE 10-6

RIPRAP REQUIREMENTS FOR CHANNEL LININGS **

$VS^{0.17}/(S_s-1)^{0.66}$* ($ft^{1}/2/sec$)	Rock Type ***
1.4 to 3.2	VL
3.3 to 3.9	L
4.0 to 4.5	M
4.6 to 5.5	H
5.6 to 6.4	VH

* where:

V = mean channel flow velocity, in fps;

S = longitudinal channel slope, in feet per foot (ft/ft); and

S_s = specific gravity of stone (minimum S_s = 2.50)

** Table valid only for Froude number of 0.8 or less and side slopes no steeper than 2h:1v.

*** Type VL and L riprap may be buried after placement to reduce vandalism.

TABLE 10-7

CLASSIFICATION AND GRADATION OF ORDINARY RIPRAP

Riprap Designation	% Smaller Than Given Size By Weight	Intermediate Rock Dimension (Inches)	d_{50}* (Inches)
Type VL	70-100	12	
	50-70	9	
	35-50	6	6
	2-10	2	
Type L	70-100	15	
	50-70	12	
	35-50	9	9
	2-10	3	
Type M	70-100	21	
	50-70	18	
	35-50	12	12
	2-10	4	
Type H	100	30	
	50-70	24	
	35-50	18	18
	2-10	6	
Type VH	100	42	
	50-70	33	
	35-50	24	24
	2-10	9	

*d_{50} = Mean particle size

TABLE 10-8

RIPRAP GRADATION LIMITS FOR STEEP SLOPES

$$\frac{D_{max}}{D_{50}} = 1.25$$

$$\frac{D_{50}}{D_{10-20}} = 2-3$$

TABLE 10-9

THICKNESS REQUIREMENTS FOR GRANULAR BEDDING

Riprap Designation	Minimum Bedding Thickness (Inches)		
	Fine Grained Soils* (Two Layer System)		Course Grained Soils**
	Type I	Type II	Type II
L, G, SM	4	4	6
M	4	4	6
H	4	6	8
VH	4	6	8

*May substitute one 12 inch layer of Type II bedding. Substitution of one layer of Type II bedding shall not be permitted at drop structures. Use of a combination of filter fabric and Type II bedding at drop structures is acceptable.

** Fifty percent or more by weight retained on the #40 sieve.

TABLE 10-10

GRADATION FOR GRANULAR BEDDING

U.S. Standard Sieve Size	Percent Weight By Passing Type I	Square Mesh Sieves Type II
3"	-	90 - 100
1-1/2"	-	-
3/4"	-	20 - 90
3/8	100	-
#4	95 - 100	0 - 20
#16	45 - 80	-
#50	10 - 30	-
#100	2 - 10	-
#200	0 - 2	0 - 3

10.10.4 Granular Bedding

Granular bedding, according to the Urban Drainage and Flood Control District, is determined according to the general classification of the base (soil) material. For fine-grained soils, a two-layer system (Type I and Type II gradations) is required for the thicknesses presented in Table 10-9, with an optional single 12-inch layer of Type II material, as noted. For coarse-grained soils, a single layer of Type II gradation of the specified thickness is required.

The bedding material gradations are presented both in tabular and graphical form in Table 10-10 and Figure 10-22. These gradations were developed to conform to Colorado Division of Highways standards for AASHTO M-6 concrete sand (Type I), and Class A Filter Material (Type II).

10.10.5 Filter Fabrics

Filter Fabrics may be used in conjunction with or in lieu of granular bedding material if adequate design evaluation is given to the erosive properties of the supporting soils, piping, long term durability of the fabric materials and installation requirements.

Riprap that is 12 inches in diameter or larger shall not be dumped directly onto plastic filter cloth since it may tear or displace the filter cloth. Instead, at least a 4-inch blanket of Type II granular filter material shall be placed over the filter cloth, or riprap shall be placed directly on the filter cloth by hand or by an equipment bucket. Side slopes shall be 2.5H:1V or flatter in order for the gravel not to slide down the filter cloth before placing the riprap. Typical filter fabric installations are shown on Figure 10-21. Filter Fabric types shall be specified by the design engineer based on site specific conditions.

10.11 Scour

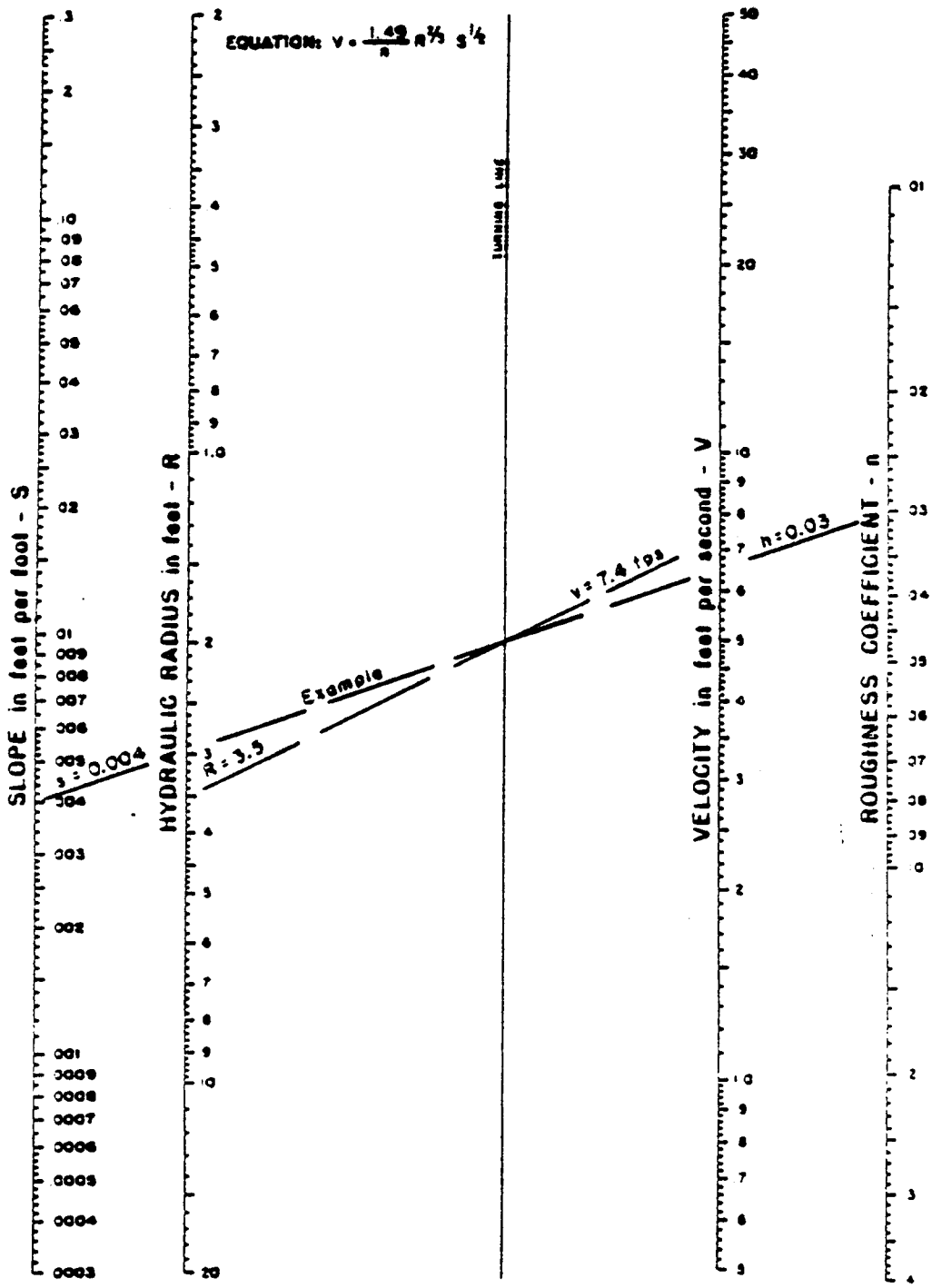
Basically, scour is the net result of an imbalance between the capacity of the flow to transport sediment out of an area and the rate of supply of sediment to that area. At a bridge crossing, for instance, the area of interest is the immediate vicinity of the bridge foundation, the piers and abutments. The imbalance of this capacity and supply can arise from a variety of causes which can be generally categorized as (1) those characteristics of the stream itself, and (2) those due to the modification of the flow by the bridge piers and abutments.

Because of the overall complexity of the hydrodynamic forces existing in a natural stream channel, the detailed flow pattern in an unobstructed stream cannot be predicted over time with great accuracy. Reasonable estimates can be made, however, on the basis of observations along reaches of similar streams, and in some cases, actual records and measurements for the particular reach of the stream under investigation.

Scour which occurs because of modification of the flow patterns by a bridge crossing can be further divided into two distinct types of scour depending upon whether or not sediment is supplied to the scour hole. Equilibrium is attained when a scour hole is enlarged to a size where the capacity to remove material from the scour hole is balanced by the rate at which sediment is supplied to the scour hole. During floods, a scour hole located in the main channel will be supplied with sediment at a rate characteristic of the stream. Ignoring the complexities of material stratification that may exist below the stream bed, the material supplied will be essentially the same as the material removed.

If no sediment is supplied to the scour hole, equilibrium is not attained until the configuration of the bed is such that the scouring capacity of the flow is zero. This condition is most likely to occur in overbank areas where vegetation reduces flow velocities, causing the coarser material to drop out of suspension, resulting in a great degree of scour in overbank areas than would otherwise occur in the main channel.

Reference (11) is made to the Simons, Li and Associates, Inc. report titled "Design Guidelines and Criteria for Channels and Hydraulic Structures on Sandy Soil", prepared for the City of Aurora and Urban Drainage and Flood Control District in June, 1981 for procedures to be used for estimating scour.



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Manning's Equation Nomograph

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Figure
10-1

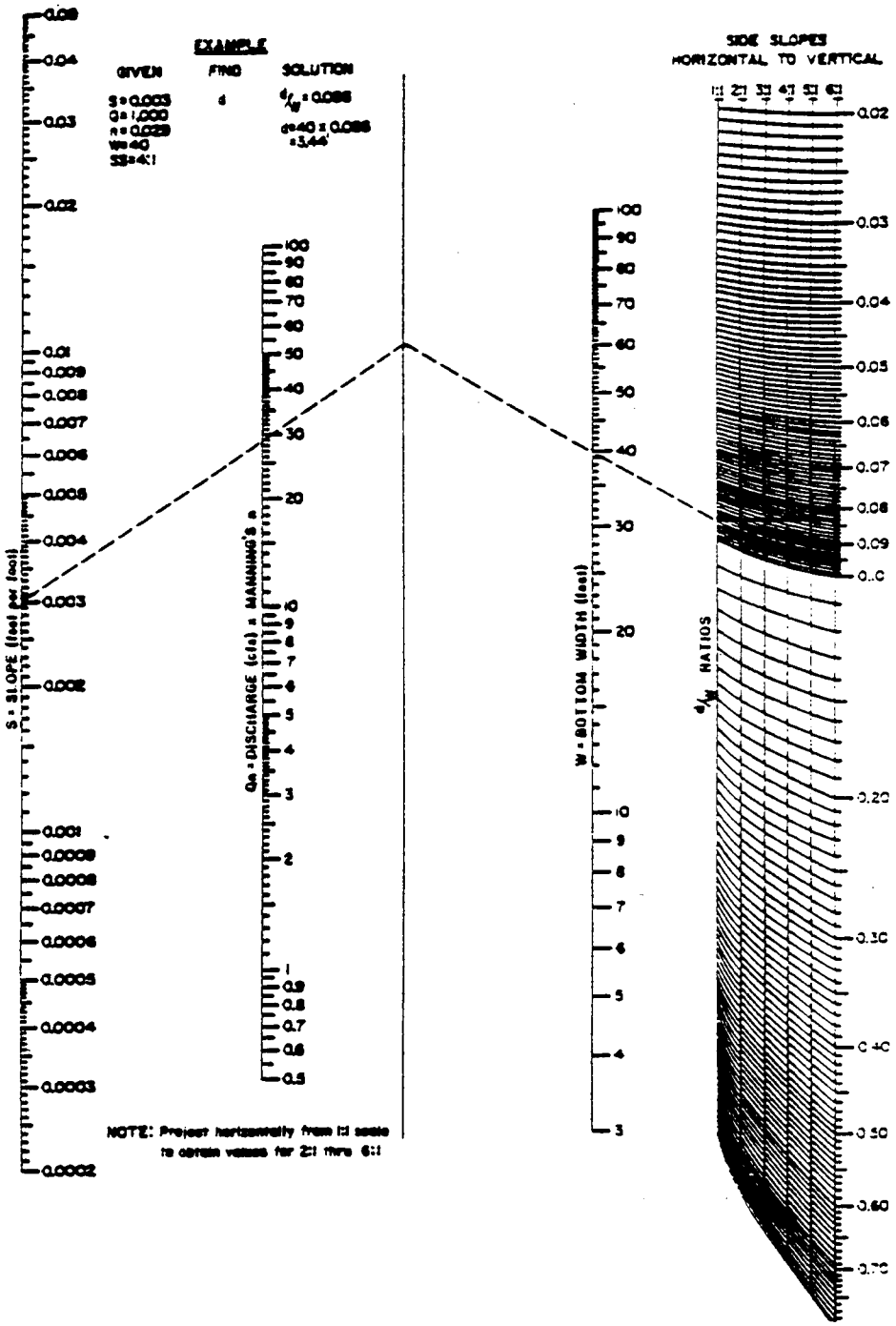

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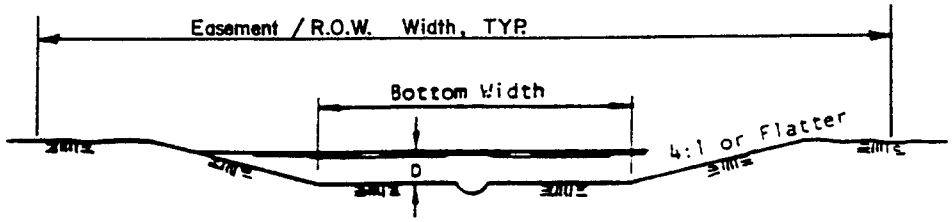
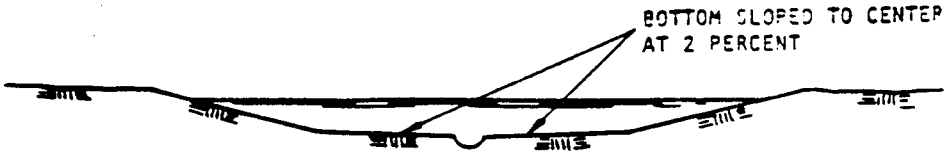
Figure
10-2

Uniform Flow for Trapezoidal Channels



FLAT BOTTOM WITH TRICKLE CHANNEL



OVAL OR SLOPED BOTTOM WITH TRICKLE CHANNEL

INITIAL DRAINAGE CHANNEL WITH OVERFLOW
AREA FOR MAJOR DRAINAGE RUNOFF

RIPRAPPED INITIAL DRAINAGE CHANNEL WITH
OVERFLOW FOR MAJOR DRAINAGE RUNOFF

NOTE: Provisions for erosion protection shall be included for the entire Easement / R.O.W.

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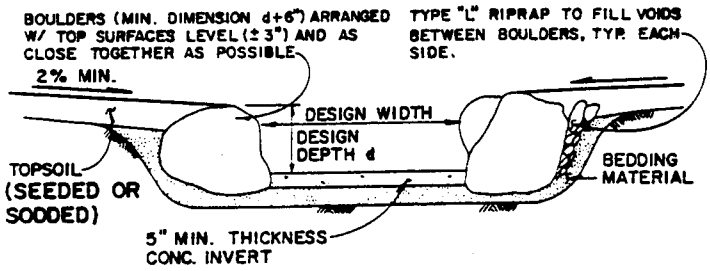
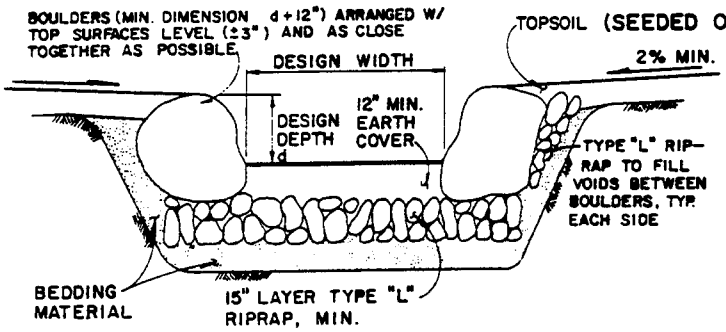
Figure
10 - 3a

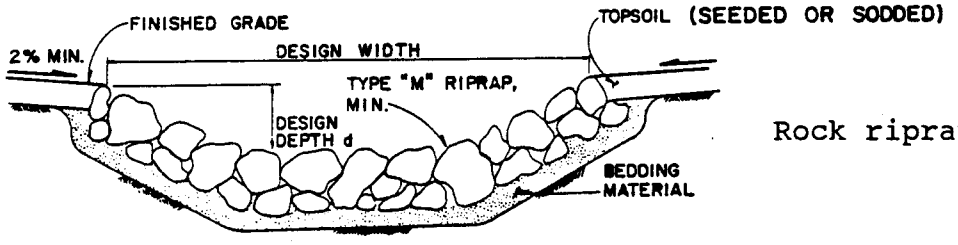
Typical Grassed - Lined Channels



Channel with boulders edge and concrete invert.



Channel with boulders edge and rock/soil invert.



Rock riprap channel.

REFERENCE : McLaughlin Water Engineers, Dec. 1986;

Drop Structures in the Denver Metropolitan Area

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Low Flow or Trickle Channel

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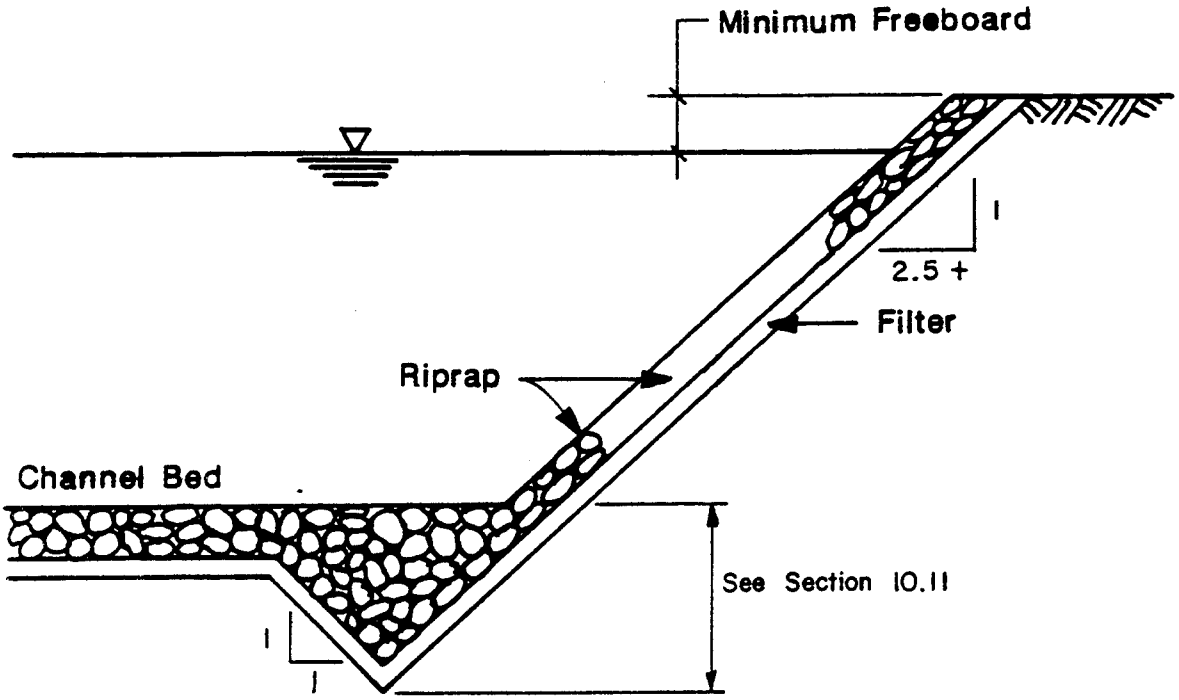
Figure
10 - 3b



Reference : Urban Drainage & Flood Control District, Urban Storm Drainage Criteria Manual, November 15, 1982.

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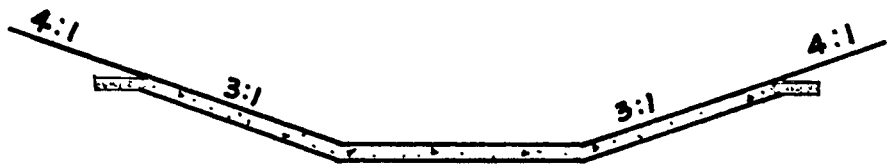
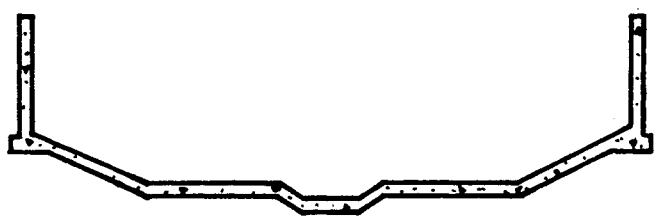
Figure
10. - 4

Ordinary Riprap

V-Bottom Type



Trapezoidal Channel with Combination Lining



Modified Trapezoidal

Reference: U.S. Army Corps of Engineers, Hydraulic Design Criteria

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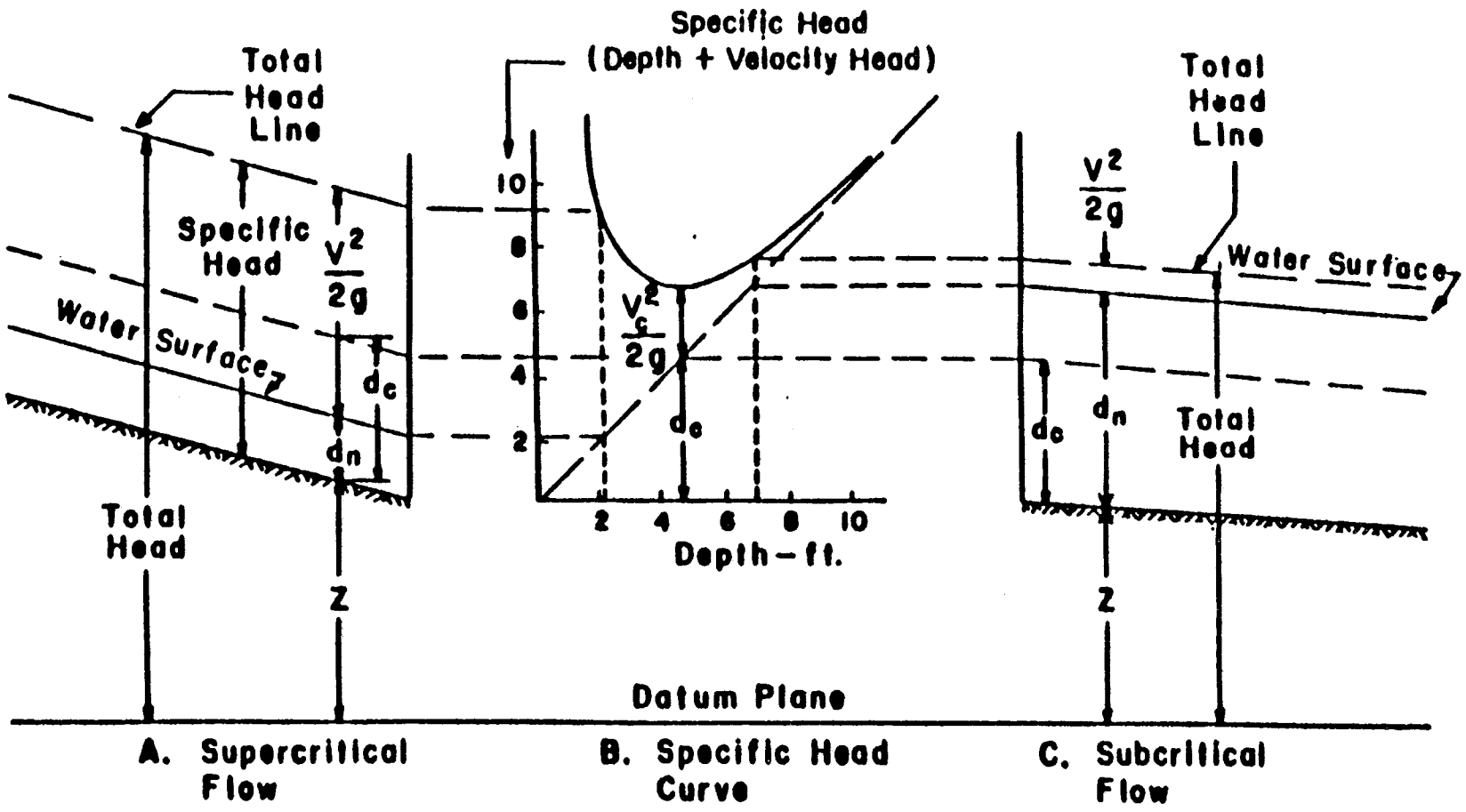
Figure
10 - 5

Special Concrete Channel Cross Sections



Reference: U.S. Department of Transportation Hydraulic Series #3 Design Charts For Open Channel Flow, 1973.

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Definition Sketch of Specific Head

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Figure
10-6

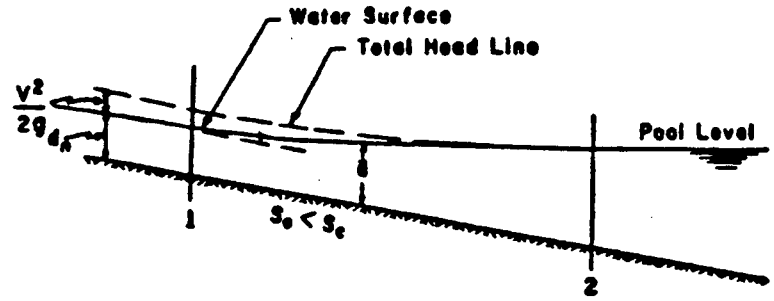

Figure 10–7a *Water-surface profile in flow from a channel to a pool.*

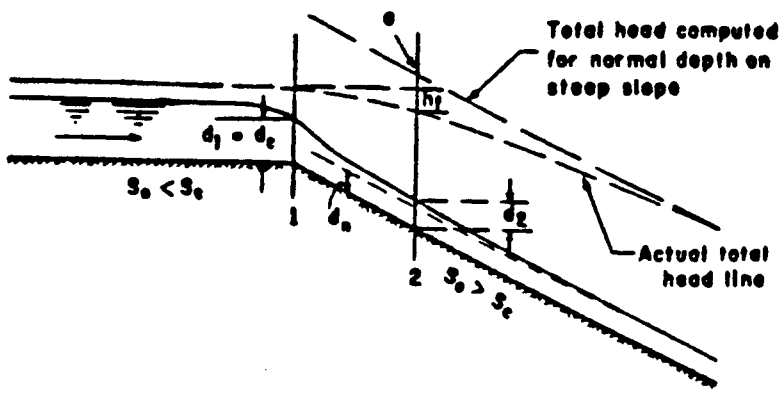

Figure 10–7b *Water-surface profile in changing from subcritical to supercritical channel slope.*

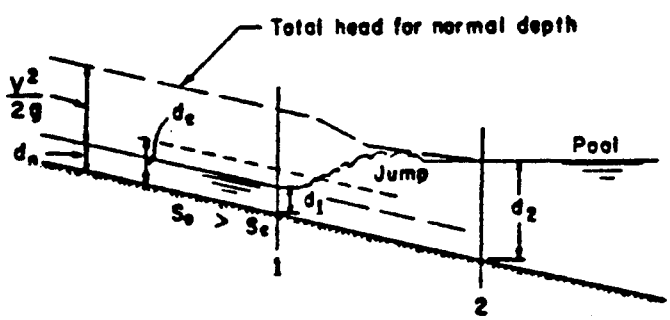

Figure 10–7c *Water-surface profile illustrating hydraulic jump.*

Reference:

U.S. Department of Transportation Hydraulic Design Series # 3 Design Charts For Open Channel Flow, 1973

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Water – Surface Profiles

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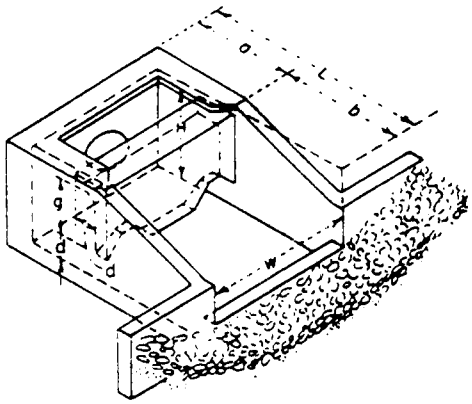
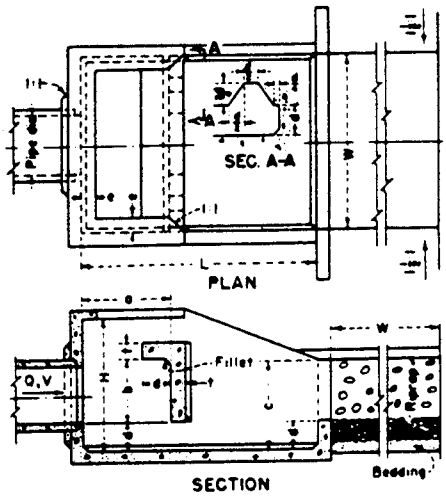
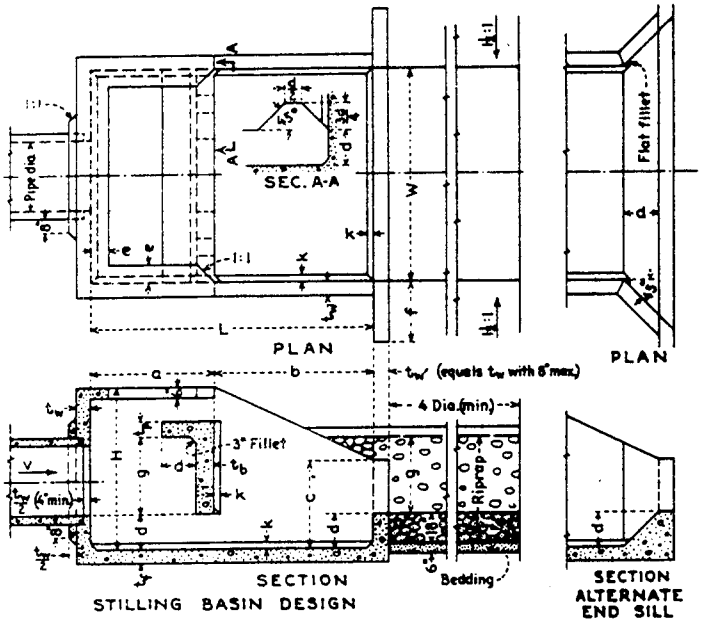
Figure
10–7



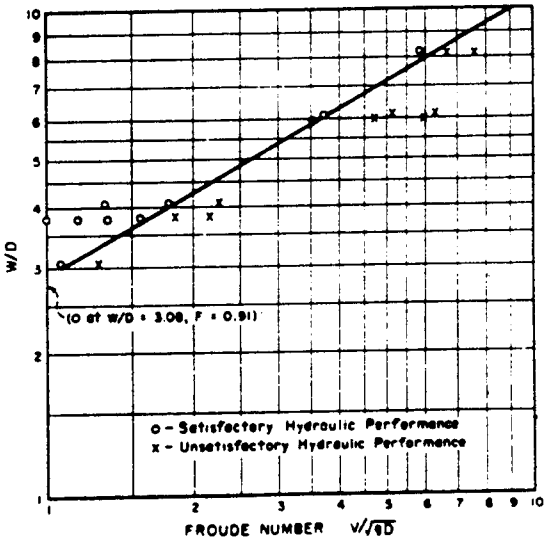


H = 5/6 (W)
L = 4/3 (W)
a = 1/2 (W)
b = 5/6 (W)
c = 1/2 (W)
d = 1/4 (W)
e = 1/2 (W)
t = 1/12 (W), suggested minimum
Riprap stone size diameter = 1/10 (W)

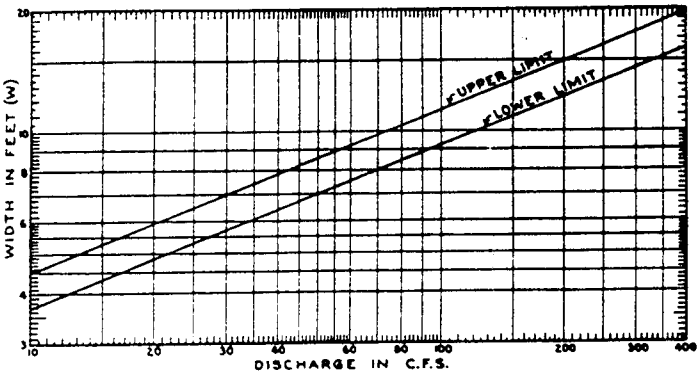
STILLING BASIN DESIGN



STILLING BASIN DESIGN



DESIGN WIDTH OF BASIN



DISCHARGE LIMITS

NOTES:

W is the inside width of the basin.
D represents the depth of flow entering the basin and is the square root of the flow area.
V is the velocity of the incoming flow.

Reference: U.S Bureau of Reclamation, Design of Small Dams, 1977

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Figure
10-8a

Dimensional Criteria For Impact Type Stilling Basin

Suggested pipe size[1]		Max discharge Q	Feet and inches										Inches					
Dia in. (1)	Area (sq ft) (2)	(3)	W (4)	H (5)	L (6)	a (7)	b (8)	c (9)	d (10)	e (11)	f (12)	g (13)	t_w (14)	t_f (15)	t_h (16)	t_p (17)	K (18)	Suggested riprap size (19)[3]
18	1.77	[2] 21	5-6	4-3	7-4	3-3	4-1	2-4	0-11	0-6	1-6	2-1	6	6½	6	6	3	4.0
24	3.14	38	6-9	5-3	9-0	3-11	5-1	2-10	1-2	0-6	2-0	2-6	6	6½	6	6	3	7.0
30	4.91	59	8-0	6-3	10-8	4-7	6-1	3-4	1-4	0-8	2-6	3-0	6	6½	7	7	3	8.5
36	7.07	85	9-3	7-3	12-4	5-3	7-1	3-10	1-7	0-8	3-0	3-6	7	7½	8	8	3	9.0
42	9.62	115	10-6	8-0	14-0	6-0	8-0	4-5	1-9	0-10	3-0	3-11	8	8½	9	8	4	9.5
48	12.57	151	11-9	9-0	15-8	6-9	8-11	4-11	2-0	0-10	3-0	4-5	9	9½	10	8	4	10.5
54	15.90	191	13-0	9-9	17-4	7-4	10-0	5-5	2-2	1-0	3-0	4-11	10	10½	10	8	4	12.0
60	19.63	236	14-3	10-9	19-0	8-0	11-0	5-11	2-5	1-0	3-0	5-4	11	11½	11	8	6	13.0
72	28.27	339	16-6	12-3	22-0	9-3	12-9	6-11	2-9	1-3	3-0	6-2	12	12½	12	8	6	14.0

[1] Suggested pipe will run full when velocity is 12 feet per second or half full when velocity is 24 feet per second. Size may be modified for other velocities by Q=AV, but relation between Q and basin dimensions shown must be maintained.

[2] For discharges less than 21 second-feet, obtain basin width from curve of Fig. 10-8B Other dimensions proportional to W; $H=\frac{3W}{4}$, $L=\frac{4W}{3}$, $d=\frac{W}{6}$, etc.

[3] Minimum size of individual stones which will resist movement when critical velocity occurs over the end sill.

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Dimensions for Impact - Type Stilling Basins

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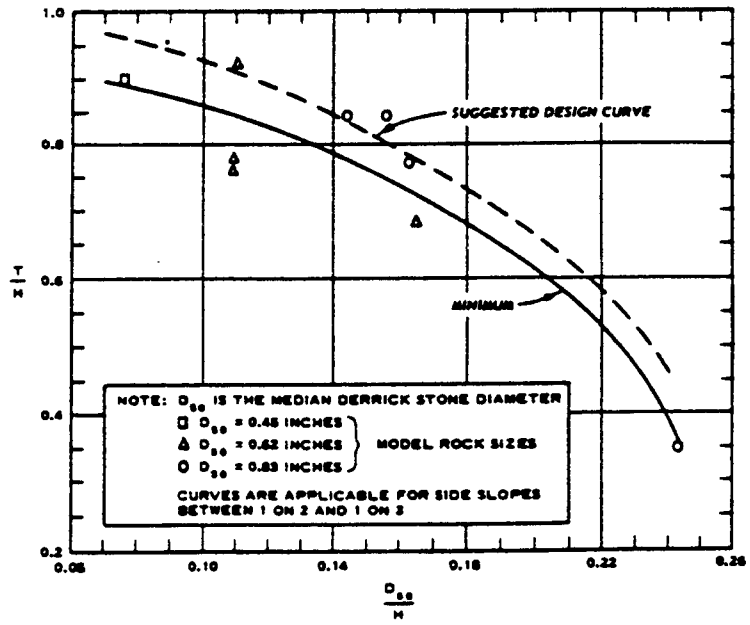
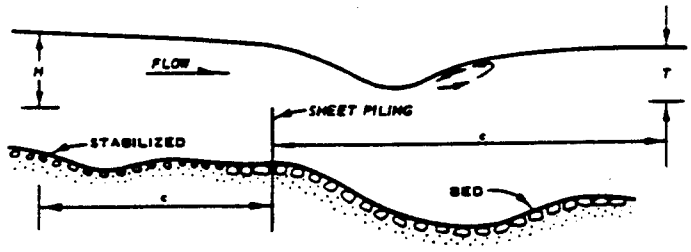
Figure
10 - 8b





DEFINITION SKETCH

GRAPH LIMITED TO

$\frac{T}{c} \geq 0.8$

c = CRITICAL DEPTH

SHEET PILING STABILIZER

DERRICK STONE SIZE

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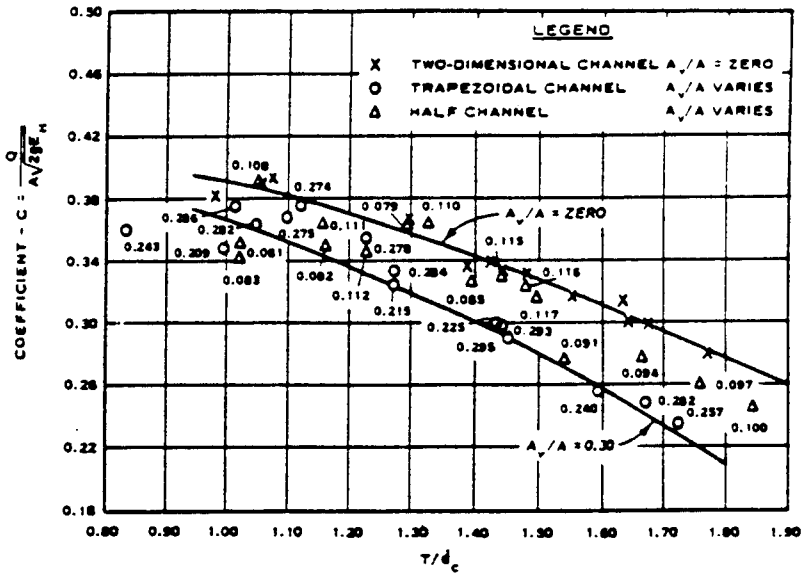

NOTE: $E_H = \frac{Q^2}{2A^2C^2g}$

Q = TOTAL DISCHARGE

E_H = ENERGY $\left(\text{TOTAL HEAD, } H + \frac{V^2}{2g}\right)$ ABOVE THE CREST $5d_c$ UPSTREAM OF THE CREST

T = TAILWATER DEPTH ABOVE THE CREST $10d_c$ DOWNSTREAM OF THE CREST

$_c$ = CRITICAL DEPTH FOR THE TRAPEZOIDAL CREST SECTION

CURVE IS APPLICABLE FOR SIDE SLOPES FROM VERTICAL TO 1 ON 3

A = TOTAL AREA ABOVE THE CREST AT $5d_c$ UPSTREAM OF THE CREST

A_v = AREAT IN THE END SECTIONS OF CREST $5d_c$ UPSTREAM OF THE CREST

NUMBERS BESIDE THE PLOTTED POINTS REPRESENT VALUES OF A_v/A

SHEET PILING STABILIZER
ENERGY LOSS

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Figure
10-9b

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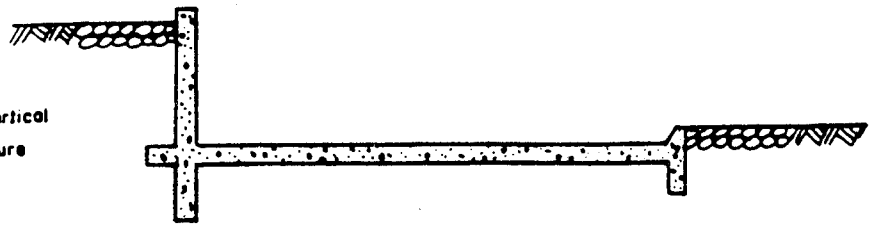
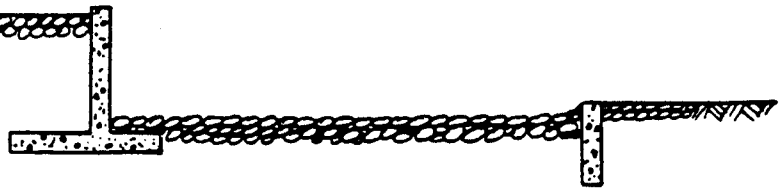
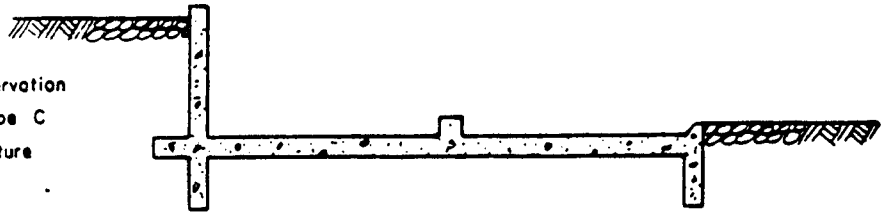
V-1. Concrete vertical drop structure



V-2. Concrete retaining wall with riprap



V-3. Soil Conservation Service Type C drop structure



V-4. Vertical drops with preshaped riprap basin

Reference :

Simons, Li & Associates , June , 1981 ; Design Guidelines & Criteria, Channels & Hydraulic Structures on Sandy Soils

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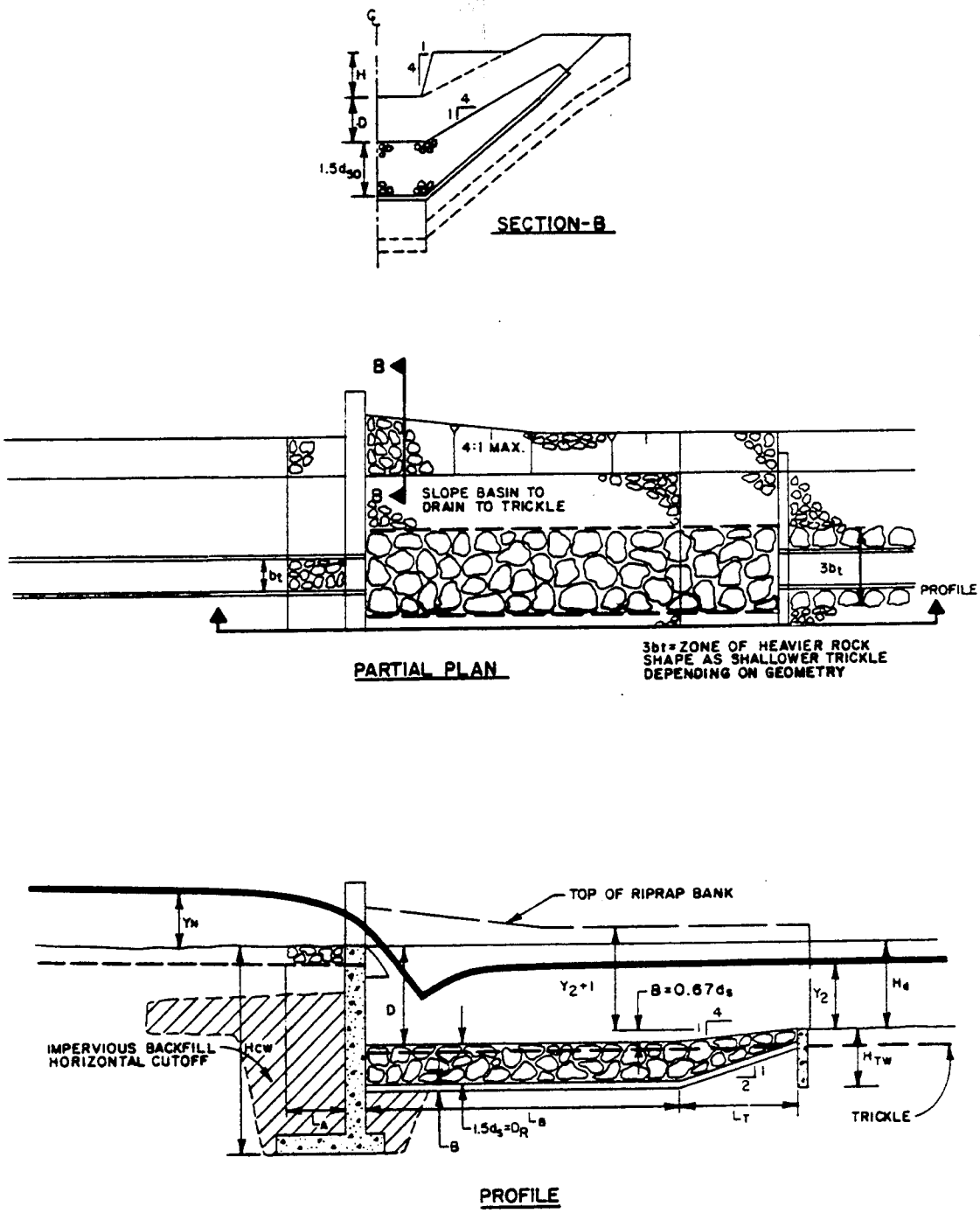
Figure
10-10

Vertical Drop Structure Types



VRR - Vertical Riprap Drop

REFERENCE : McLaughlin Water Engineers, Dec. 1986;Drop Structures in the Denver Metropolitan Area

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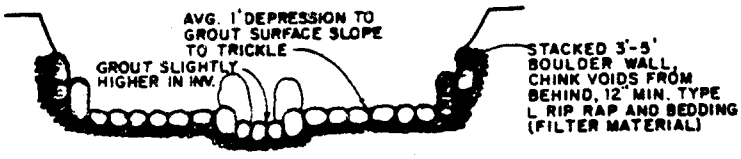
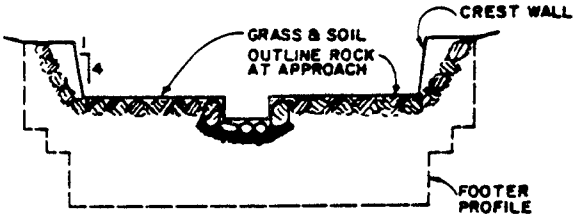
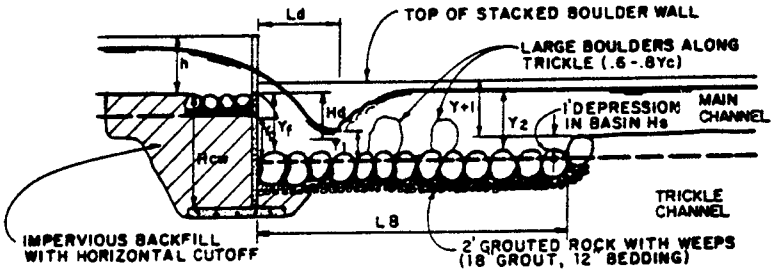
Figure
10-11



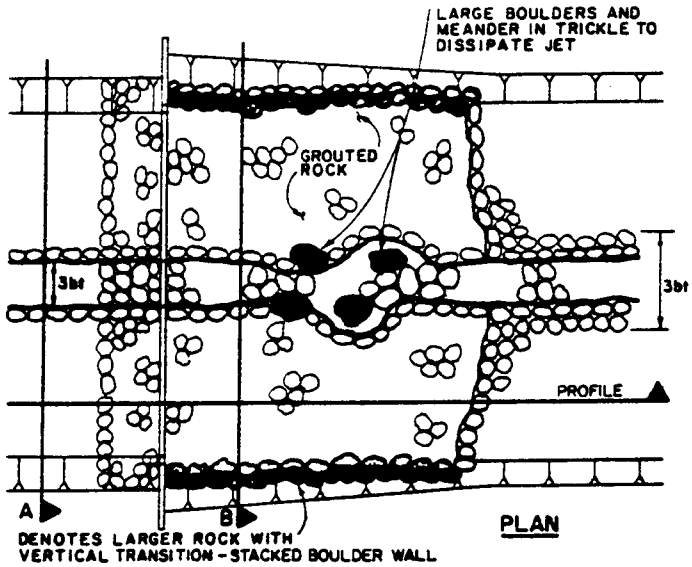
SECTION-B



SECTION-A



PROFILE



PLAN

VHB - Vertical Hard Basin Drop

REFERENCE : McLaughlin Water Engineers, Dec. 1986;Drop Structures in the Denver Metropolitan Area

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Figure
10-12

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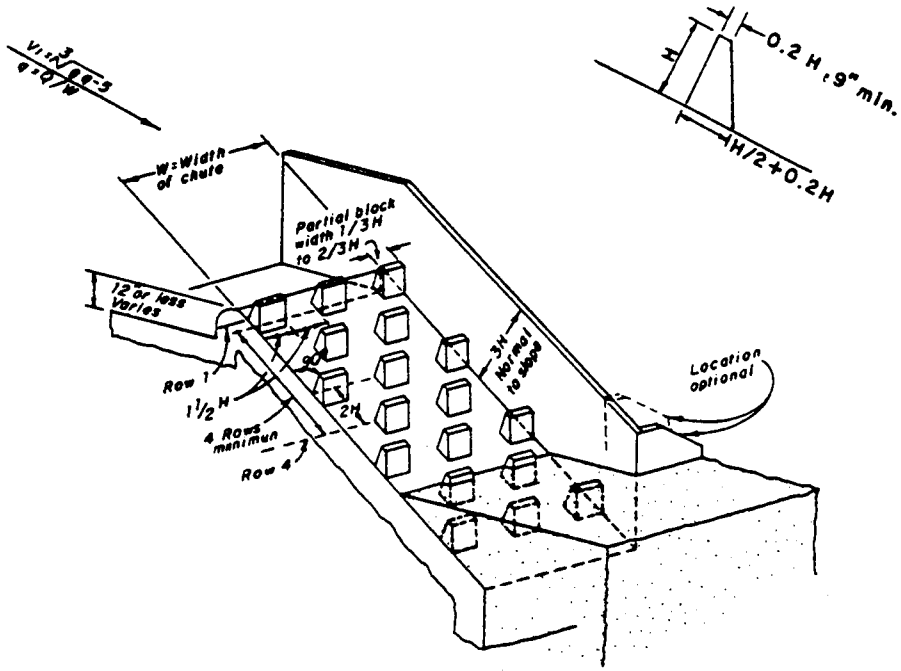
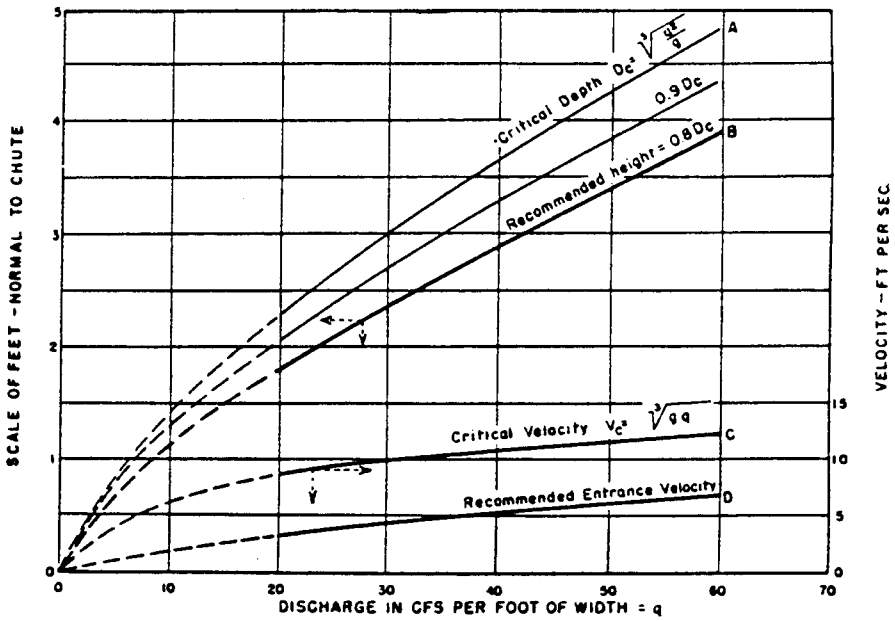


BC - Baffled (Apron) Chute Drop



REFERENCE :

McLaughlin Water Engineers, Dec. 1986; Drop Structures in the Denver Metropolitan Area

Recommended Baffle Pier Heights and Allowable Approach Velocities

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Drainage Criteria Manual

Date
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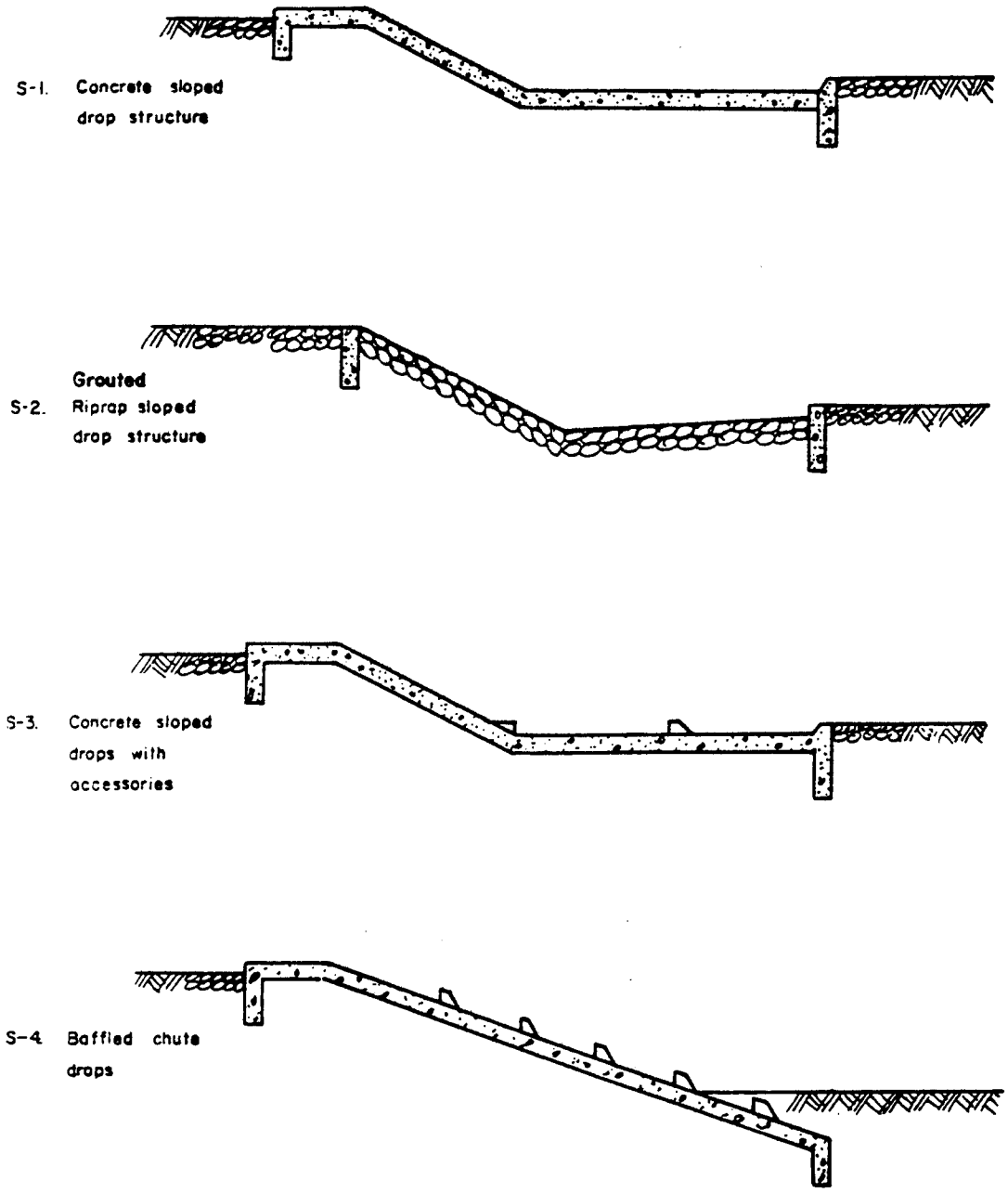
Figure
10-13



Reference :

Simons, Li & Associates, June, 1981; Design Guidelines & Criteria, Channels & Hydraulic Structures on Sandy Soils

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Sloped Drop Structure Types

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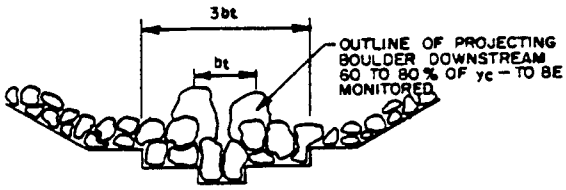
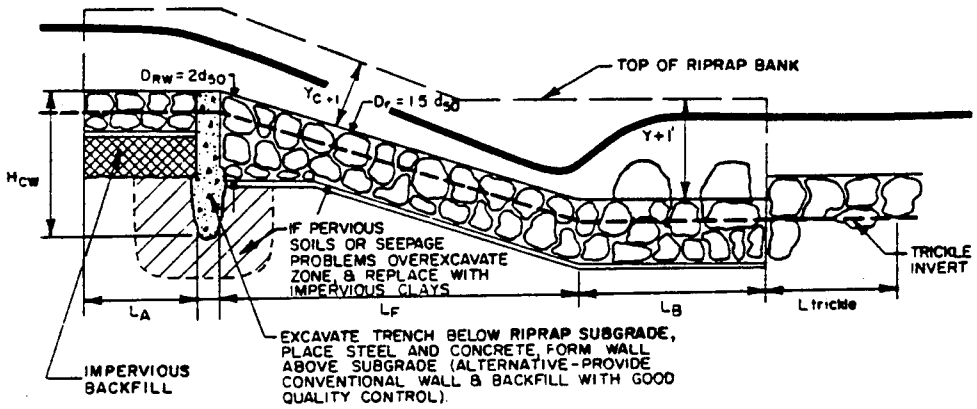
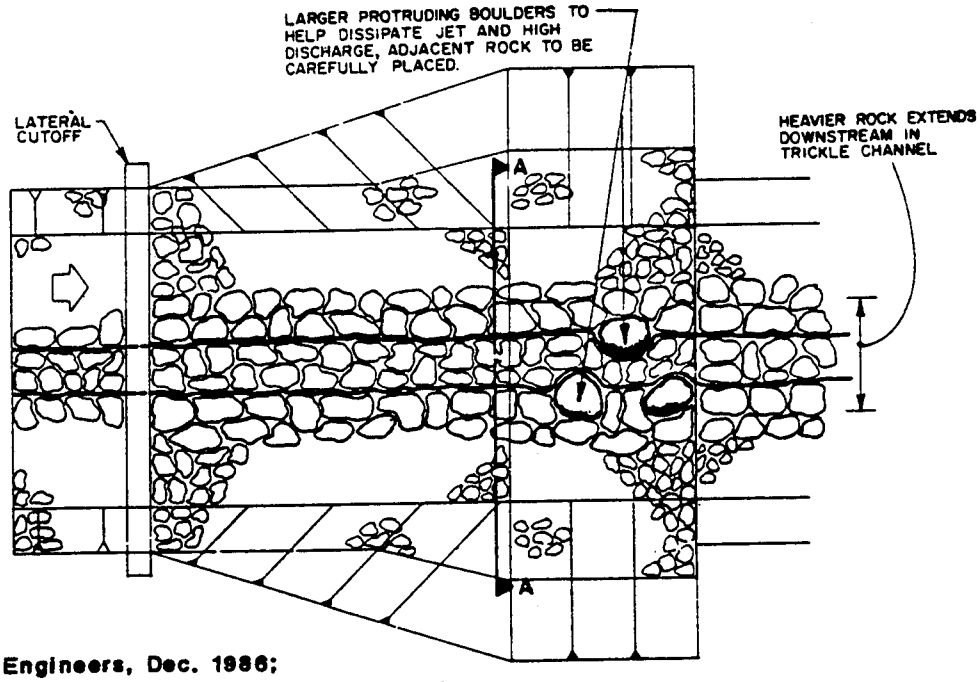
Figure
10 - 14



SECTION "A"



PROFILE



PLAN

REFERENCE :

McLaughlin Water Engineers, Dec. 1986; Drop Structures in the Denver Metropolitan Area

SLR - Sloping Large Riprap Drop

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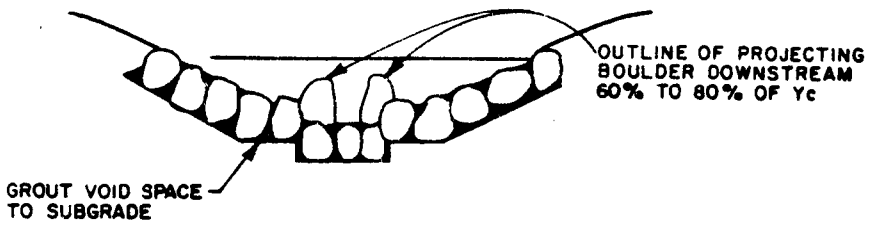
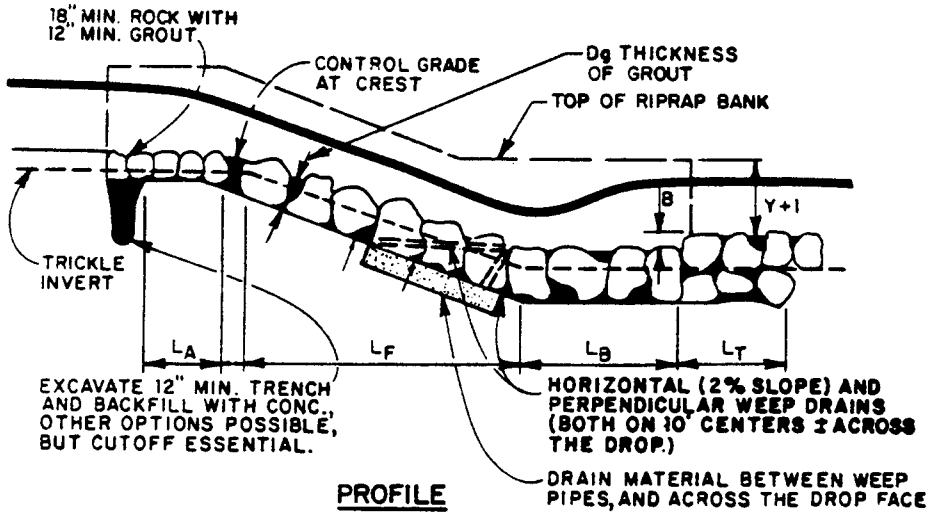
Figure
10-15



SECTION-A



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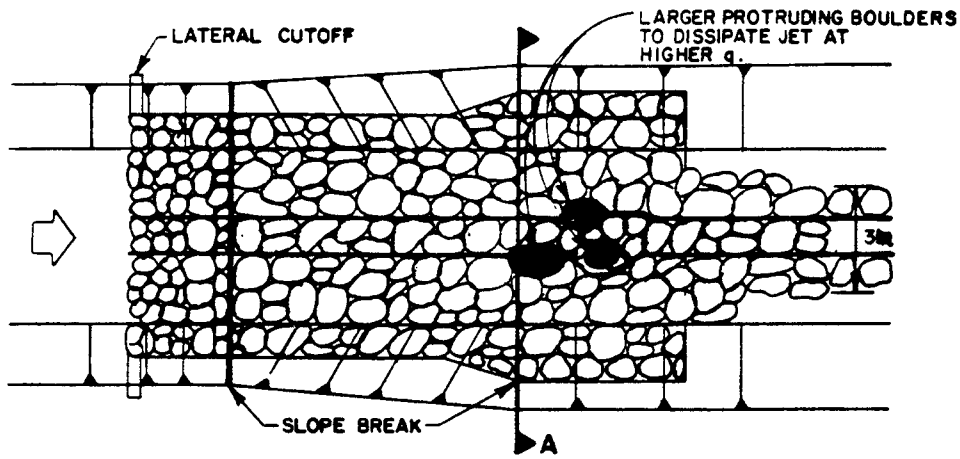

PLAN

REFERENCE :

McLaughlin Water Engineers, Dec. 1986; Drop Structures in the Denver Metropolitan Area

GSB - Grouted Sloping Boulder Drop

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Figure
10-16

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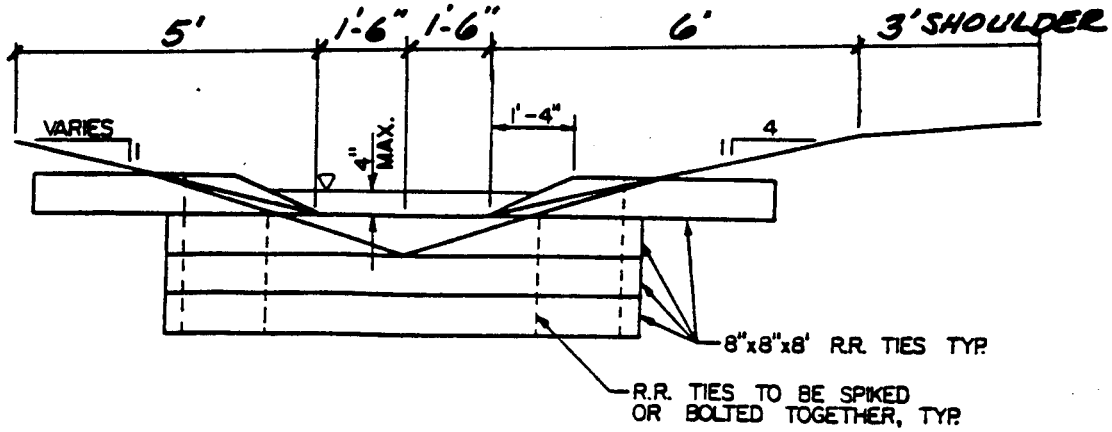

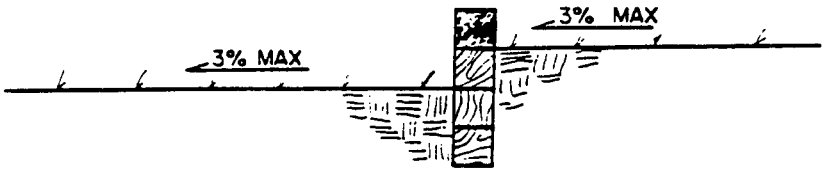

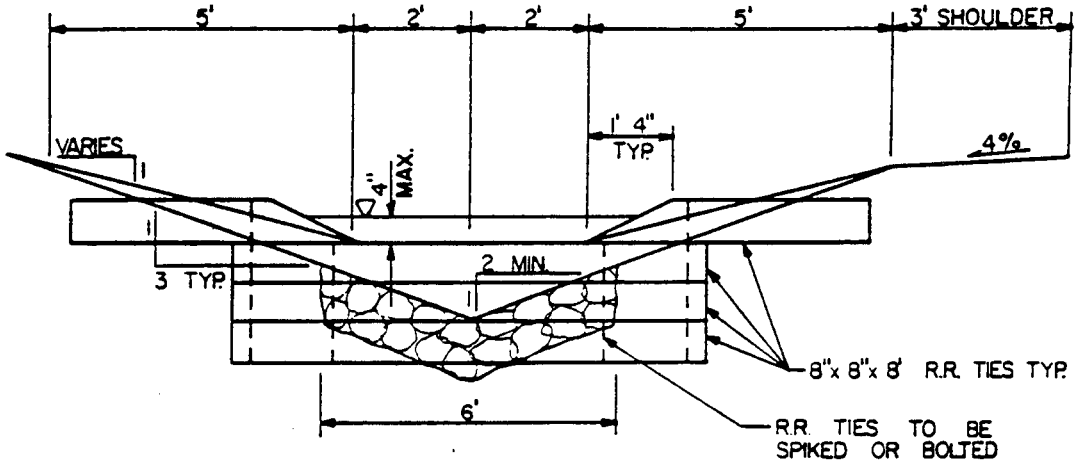

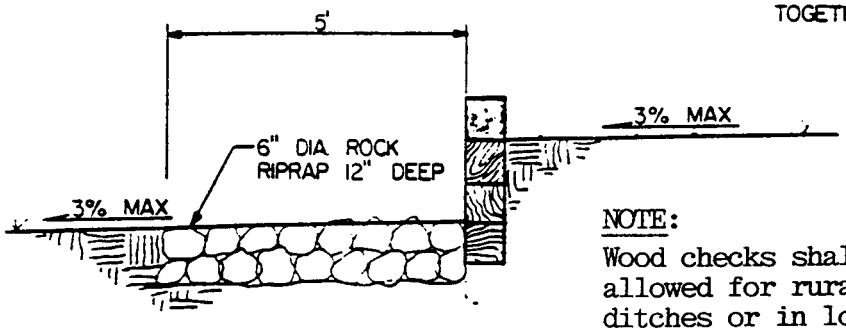

NOTE:
Wood checks shall only be allowed for rural roadside ditches or in locations as otherwise specified.

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Ditch Check Detail - Wood Ties

Date
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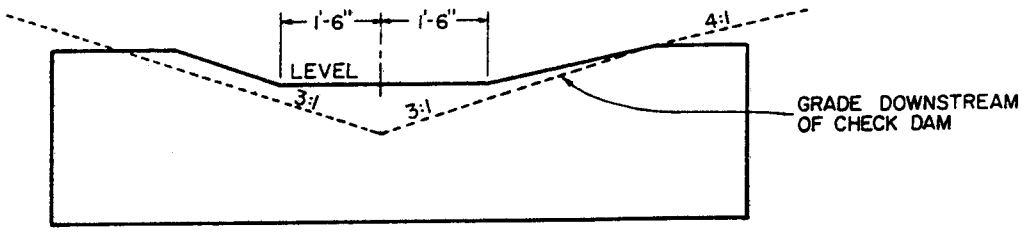
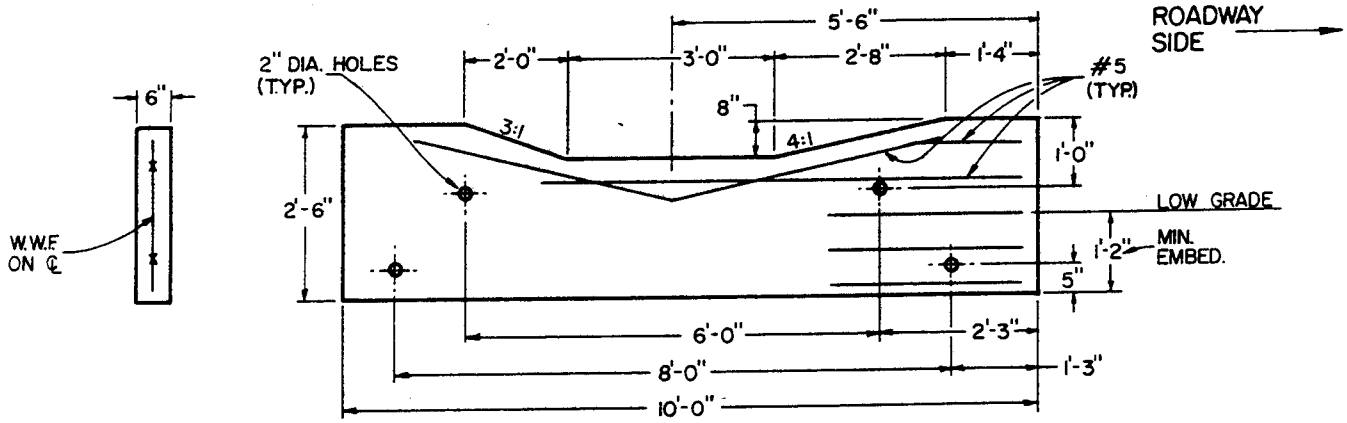
Figure
10-17



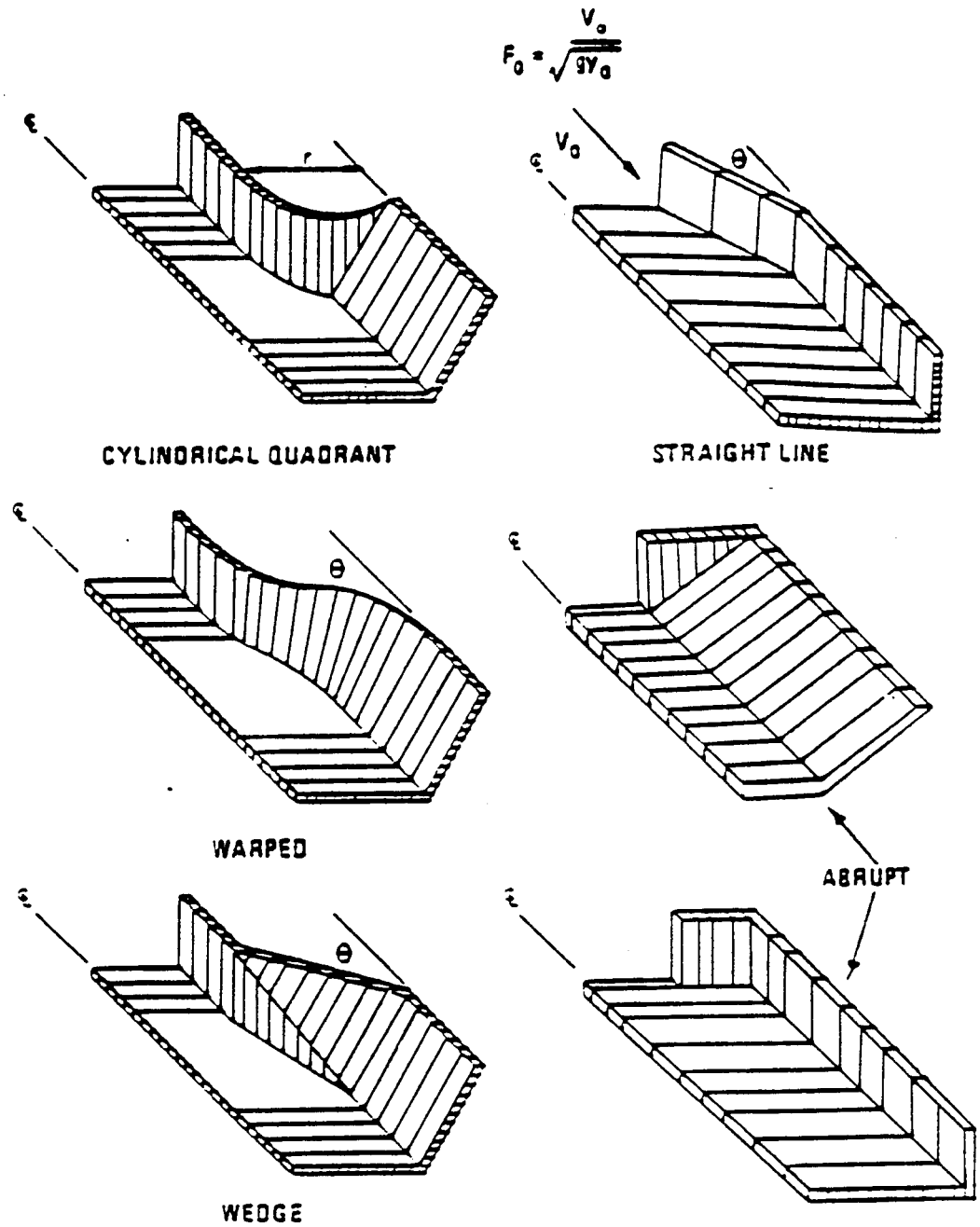


NOTE:

* CONCRETE 3000 PSI 28 DAY COMP. STRENGTH, TYPE IIA
* 6x6-6/6 WWF ASTM A-185, ON CENTER
* 4-#5, 9'-6" LONG ON CENTER, GRADE 40 OR 60
* 2" DIA. THROUGH HOLES (4 EA)
* INSTALL 6" d50, RIPRAP DOWNSTREAM OF CHECK 5'LONG, 6' WIDE

REFERENCE: EL PASO COUNTY ROADWAY CHECK DAM DESIGN PRECAST CONCRETE DETAIL CHARLES BARBER CONSTRUCTION INC. PRECAST 12-26-84, REVISED 4-23-91 CITY/CO.

The City of Colorado Springs / El Paso County Drainage Criteria Manual	Date OCT. 1987
Ditch Check Detail Concrete 10-89	Figure 10-18



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Date
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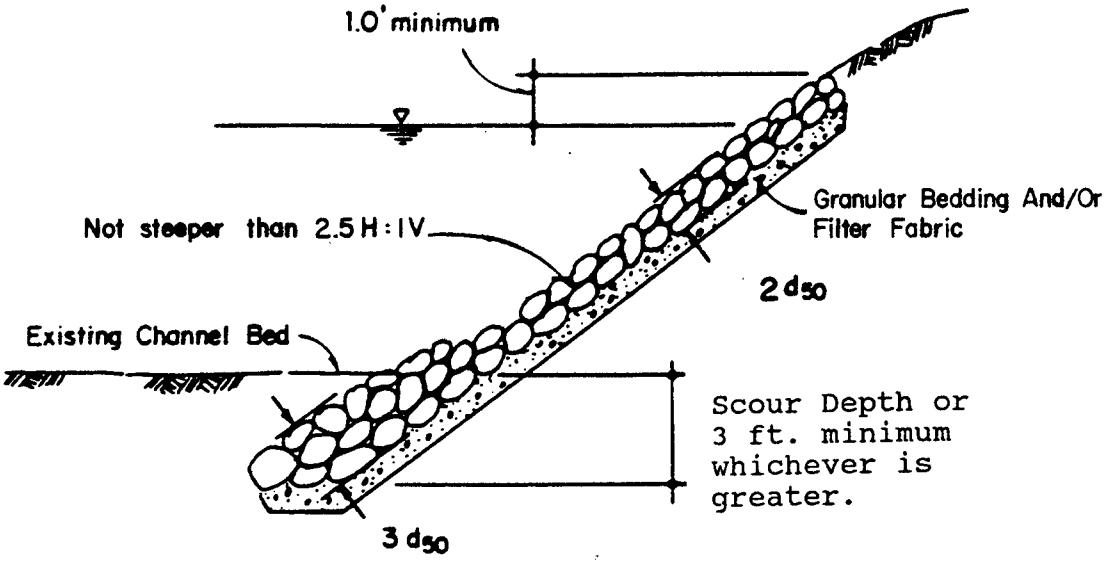
Figure
10-19

Channel Transition Types



Reference: Urban Drainage & Flood Control District, Urban Storm Drainage Criteria Manual, November 15, 1982.

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Toe Protection Riprap Channel Lining

Date
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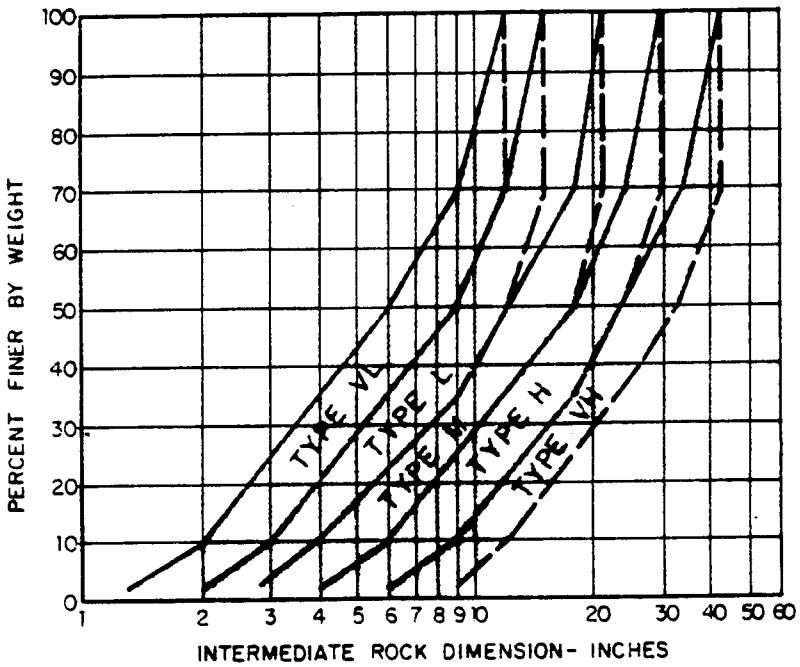
Figure
10-20



Reference : Urban Drainage & Flood Control District, Urban Storm Drainage Criteria Manual, November 15, 1982

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Date
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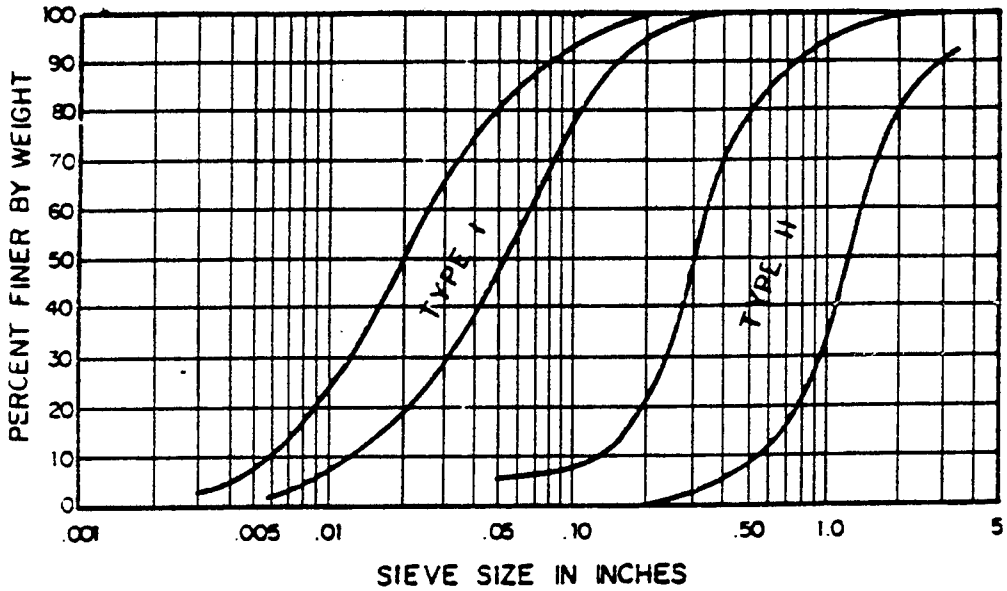
Figure
10-21

Riprap Sizing Curves



Reference : Urban Drainage & Flood Control District, Urban Storm Drainage Criteria Manual, November 15, 1982

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Date
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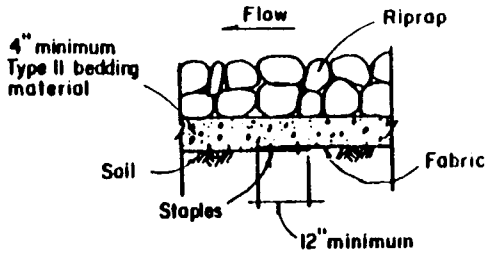
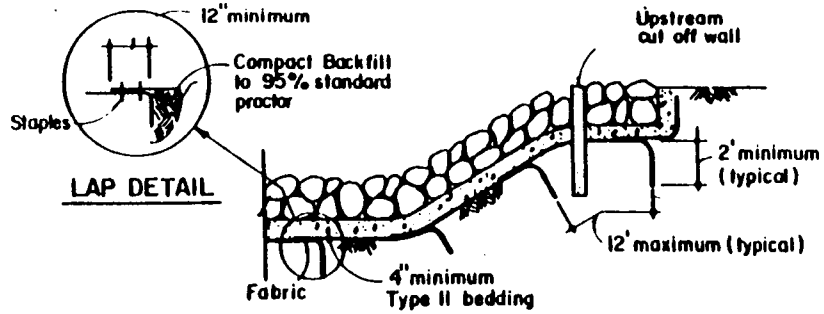
Figure
10-22

Gradation Curves For Granular Bedding

NOTE: Always place fabric starting downstream and work upstream

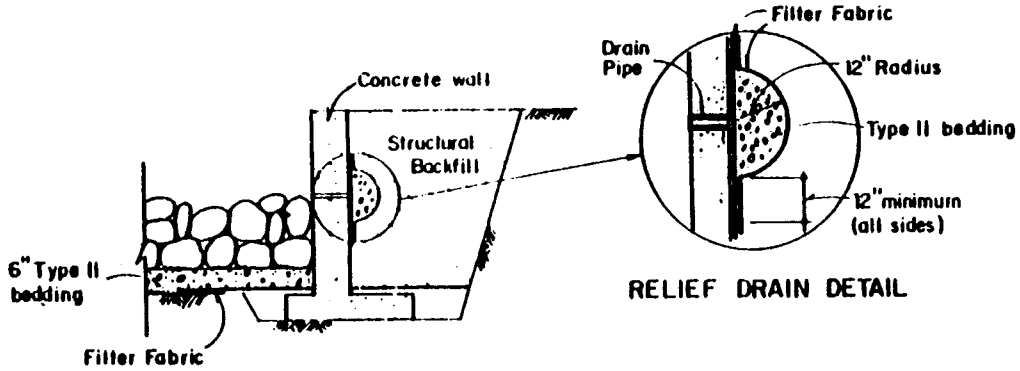


TYPICAL LAP DETAIL AND FILTER FABRIC PLACEMENT



FILTER FABRIC AT RIPRAP CHUTE DROP

NOTE: Some soils may require a hydraulic cut off to prevent piping failure of drops



RELIEF DRAIN DETAIL

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Drainage Criteria Manual

Typical Filter Fabric Placement

Date
OCT. 1987

Figure
10-23

10.12 Bibliography

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12. Resource Consultants, Inc., Storm Water Management Manual, Larimer County, CO., 1979.

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SECTION III

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CHAPTER 11 DETENTION STORAGE

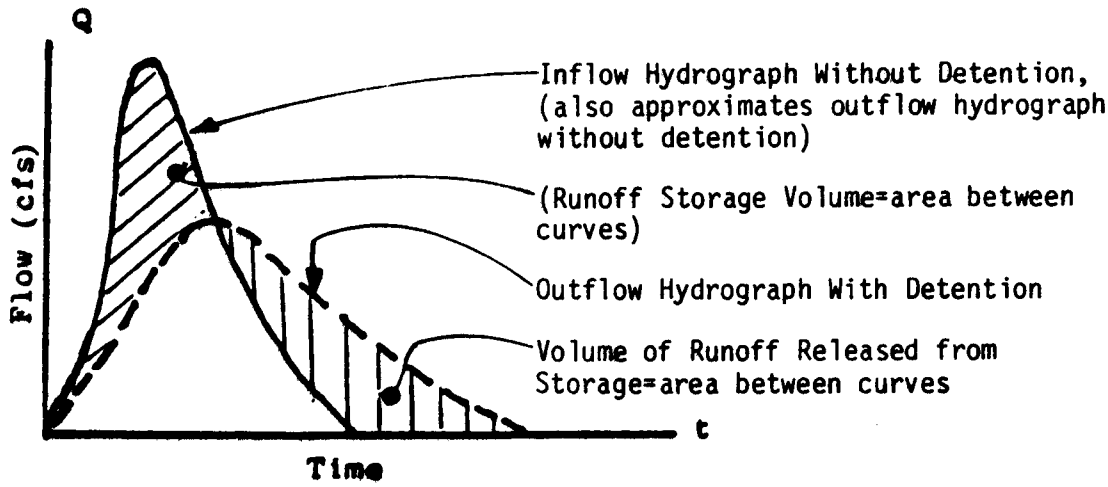
11.1 General Statement

Detention storage of runoff is an alternative to other methods of urban storm water management. Storage can often be an effective and economical means of reducing peak flow rates and mitigating downstream problems including flooding, pollution, soil erosion and siltation. Detention storage, however, also has disadvantages in terms of high maintenance costs, owners liability and potential aesthetic impacts.

The primary purpose of a storm water detention basin is to collect and store excessive runoff and thus, mitigate local flooding hazards to downstream areas. Discharge from this facility may be limited to a rate equal to (1) capacity of downstream drainageway (natural and/or man made) and/or (2) adopted governmental regulations and ordinances such as maintaining historic runoff release rates. Since accurate assessments of flooding conditions in an urban area would have an important bearing on public safety and costs, an engineer has to use considerable judgement in sizing a storm water detention basin and associated drainage facilities. Overall, the available pertinent hydrologic and basin data should provide the basis for a designer's choice of method, but yet be in compliance with local policy. The criteria for design of storm water detention facilities in the City/County is outlined in Chapter 2.5.

The purpose of this chapter is to introduce the design procedures and methods of application of detention pond facilities. The criteria and procedures for the design and evaluation of storm water detention basins are based on currently acceptable synthetic hydrologic methods. The applicability of these methods for design purposes can only be established on the basis of available basin data and reliability in the set of inherent assumptions made and used in their respective applications. Design considerations for sizing storm water detention basins require an inflow hydrograph and an outflow hydrograph, the latter of which is usually limited by local policy and State Law. The following illustration compares hydrographs with and without storm water detention.

On-site detention of storm water refers to storage of excess runoff on the site of a development prior to its entry into a public drainage system or outfall. Regional detention facilities, however, which can be more cost effective or which may be the only feasible alternative to mitigate downstream flooding problems may also be utilized. Regional detention facilities should be identified in a Drainage Basin Planning Study or Master Development Drainage Plan. Small on-site detention facilities are not encouraged by the City/County.



Note: Area between inflow and outflow hydrographs equals volume of excess runoff stored and released. The two volumes are ordinarily equal, except for local infiltration into the subsurface and evaporation (or evapotranspiration).

The benefits usually derived by detention storage are that: runoff is spread over a longer time period; peak flows to downstream areas can be controlled in magnitude; and total runoff volume may, or may not, be reduced (dependent upon rate of infiltration into soils).

Regional detention facilities can provide runoff control for many square miles of tributary area. They are commonly designed for the 100-year storm rainfall event, and are designed to provide sufficient storage capacity. Dual outlet works to also accommodate lower frequency storm events such as the 10-year storm event may be required where downstream conditions warrant.

Other beneficial features for regional facilities include the following elements.

- All of the storage is on publicly-owned land which overcomes the issue of "who will operate and maintain the facility." Responsibility is well defined.

- Economics of construction and operation and maintenance (O&M) feasibility favor the regional reservoirs.

- They are less likely to develop nuisance problems (i.e. weeds, erosion, mosquitoes, water pollution, etc.

- Satisfactory operation and maintenance is not usually a problem when such facilities are properly planned, designed and constructed and when an adequate and assured source of funding is available on a continued basis.

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- Multiple-purpose use of detention storage facilities is more feasible with larger detention facilities (i.e., football and baseball fields). Such use may be allowed by the City/County through special multi-use and maintenance agreements.

Key factors for identifying and selecting detention storage sites include locations both on and off stream of drainages. On-stream detention provides attenuation of runoff in a principal drainage system; i.e., in receiving streams. Off-stream detention provides storage accomplished "off line"; i.e., not along a principal drainage course.

The advantage for <u>on-stream</u> detention is that storm rainfalls of short duration will be well damped.

The potential disadvantages, for <u>on-stream</u> detention, include the following:

- Long-duration rainstorms may produce a condition in which inflow and outflow are comparable in magnitude, and the water level (stage) stabilizes. The increased flows caused by urbanization can, therefore, be more costly to prevent from being conveyed into downstream reaches.

- Increases stream stages in upstream areas.

The advantages for <u>off-stream</u> detention include the following:

- Adds flexibility to design:
  - location choices
  - depth and area alternatives
  - layout alternatives
  - hydraulic alternatives

- Opportunities for multiple-purpose use may be enhanced.

Disadvantage for <u>off-stream</u> detention include the following:

- Land required for storage may be prohibitively expensive, or land may not be available.

- Operation and/or maintenance expenses can be high.

- Peak flow rates and storage may not be sufficient to have significant peak reduction when combined with downstream hydrographs.

The advantages for <u>on-site</u> detention (private) include the following:

- Mitigates flooding problems at their sources.

- Construction cost of storm water collection and transport system is less.

- Aesthetics can be improved if properly constructed and maintained.

- Can sometimes be incorporated into other local programs neighborhood parks, etc.

Disadvantages for <u>on-site</u> detention (private) include the following:

- Design and Construction is not usually consistent with Drainage Basin Planning Studies.

- Sometimes requires use of expensive, desirable building sites.

- May result in a proliferation of randomly located detention facilities which often create storm water management problems for local agencies.

- Maintenance of many scattered facilities is frequently neglected and is often a serious problem for local public agencies or property owners. Environmental detriments and safety hazards can develop when maintenance is lacking.

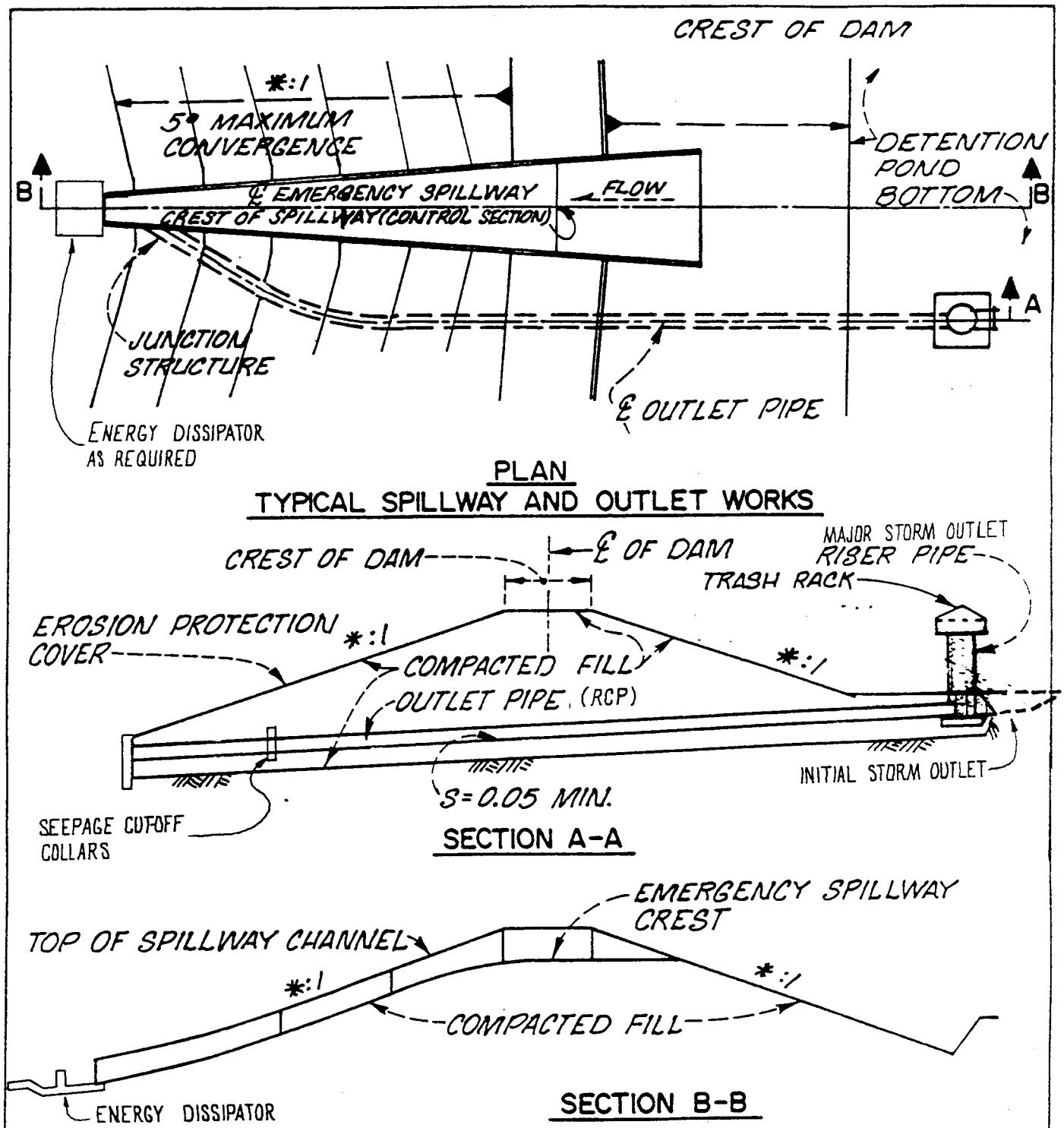
- Overall, initial construction costs are higher when compared to regional facilities with the same hydraulic function.

Embankment-type storage areas may fall under the purview of State of Colorado jurisdictional dams. The designer of such structures shall refer to the applicable draft Colorado Office of the State Engineer "Design Review Manual" for dams and dam safety (Office of the State Engineer; effective September 30, 1988) for detailed design criteria which is beyond the scope of this manual (also see Chapter 6.6).

A typical detention pond facility, with appurtenant structures, is illustrated on Figure 11-1.

11.2 Detention Storage Planning

Detention storage utilization is usually controlled by the planner in the early stages of development. However, architects, engineers, home builders, land developers and governmental officials all have a responsibility to work towards implementing storage facility siting whenever feasible. Storage in the upper portion of the basin offers the greatest potential for reducing the cost of urban drainage facilities construction. Detention storage in downstream areas can often have adverse peak flow impacts to drainageways due to the adjustment of peak times in hydrograph combinations.



* TO BE VERIFIED BY GEOTECHNICAL INVESTIGATION / DESIGN (4:1 REQUIRED BY CITY/COUNTY)

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EARTH EMBANKMENT

Typical Detention Pond Configuration

Date: OCT. 1987

Figure: II-1

11.2.1 Recreational Areas/Multi-use Facilities

Recreational areas are generally characterized by large grassed fields which often have high surface infiltration rates. Storm runoff from such fields is typically low. An alternate use of such recreational fields can be made by providing for the ponding of runoff from adjacent areas. Grassed recreational fields can be utilized for detention if the fields do not adversely affect the areas primary function as a detention facility.

Most urban areas include parks, both the local neighborhood type and the large central type. Parks, like recreational fields, can provide the possibility for detention storage of runoff.

The responsibility for maintenance must be established with the City/County for multi-use facilities.

11.2.2 Detention Facility Construction

The construction of detention facilities which multi-use benefits can provide significant benefits when properly planned and designed. Controlled outlets for flood surcharge storage should be provided, and it is required that such outlets be designed to release at a rate that does not exceed the peak rate estimated under natural conditions for the design storms, or other discharge established by policy and/or the drainage basin planning study.

Adequate safety measures shall be provided with all detention facilities. A minimum 15-foot maintenance easement shall be provided around the perimeter of the impoundment and embankment areas. Access to the bottom of the pond from a public road shall be provided via a minimum 15-foot wide ramp at a slope no greater than twelve (12) percent.

The geologic conditions of the site shall be investigated in sufficient detail to determine the suitability for impoundment of surface water. Ground water level increases downstream of the geologic investigation should be consistent with the class of structure and the complexity of the local site geology.

Guidelines for conducting geotechnical investigations for State of Colorado jurisdictional dams are presented in the draft "Design Review Manual" for dams and dam safety (Colorado Office of the State Engineer, July 31, 1986).

A design engineer check list for State of Colorado jurisdictional dams is included as Attachment A of this chapter. For non-jurisdictional dams i.e., those that do not or would not fall under State of Colorado purview, the designer must evaluate the appropriate factors identified, in the engineer check list, for the hazard rating presented as Attachment A and as otherwise required by the City/County.

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11.3 Hydraulic Design Considerations

11.3.1 Principal Outlet Works

For permanent public and private facilities, reinforced concrete pipes or boxes shall be utilized for the principal outlet works and shall be designed to completely drain the flood runoff volume in the storage area. The minimum pipe diameter shall be 18 inches with inlet orifice modifications as necessary to control discharge rates.

To prevent the formation of flow vortices at the inlet, a antivortex baffle shall be installed. Vortices can significantly reduce the discharge for a given headwater condition because of high energy losses.

Depending on the geometry of the outlet structure, discharge for various headwater depths can be controlled by the inlet crest (weir control), barrel opening (orifice control), or barrel pipe (pipe control). Each of these flow controls shall be evaluated when determining the rating curve of the principal outlet.

The various outlet works flow conditions can be evaluated as follows:

a) Weir flow may be computed by the following equation:

$$Q = CLH^{1.5}$$

where:

C = weir coefficient (dimensionless); for riser pipes, C = 3.1 may be used;

L = length of weir, in ft. For circular riser pipes, L is the pipe circumference;

H = the depth of flow over the riser pipe crest, in ft; and

Q = discharge, in cfs.

b) Orifice flow may be computed by the following equation:

$$Q = CA(2gH)^{0.5}$$

where:

C = orifice coefficient (dimensionless); for sharp-edged orifices, C = 0.6;

A = cross-sectional area of the pipe, in ft^2;

g = gravitational acceleration constant, 32.2 ft/sec^2; and

H = head above the centerline of the pipe, in ft.

c) <u>Pipe flow</u> may be computed by the following equation:

$$Q = A \quad 2g(H-S_f L)/(1+k_b+k_e)$$

where:

H = the difference between headwater and tailwater elevations, in feet;

S_f = friction slope, in feet per foot (ft/ft);

L = length of pipe, in ft;

k_b = bend or junction loss coefficient(s); and

k_e = entrance loss coefficient

11.3.2 Emergency Spillways

The location, profile and length of an emergency spillway are influenced by the geologic and topographic features at a given site. The optimum design of the spillway cross-section dimensions are best determined by performing reservoir routing of the design flood. The emergency spillway shall be designed to pass the maximum peak discharge for the 100 year design storm without damage to the structure assuming the outlet structure is totally inoperative and the upstream watershed is fully developed. Adequate provision shall be made for additional embankment safety by providing minimum freeboard above the potential maximum design flood water surface elevation (minimum 2.0 ft).

All detention facilities shall be designed and constructed with erosion resistant emergency spillways. Emergency spillways for detention ponds may be designed as grass-lined open channels only as allowed by the City/County. Vegetated spillway design velocities shall be limited by the criteria listed in Table 10-4.

Discharges from an emergency spillway shall be so directed that flows return to the downstream channel while limiting erosion potential along the downstream toe of the dam. Emergency spillways proposed for the protection of earthen embankments shall be in undisturbed earth, if possible, to avoid flows against the constructed fill. The side slopes of the excavated channel in earth shall be no steeper than 4H:1V for ease of maintenance. Where the site conditions prevent a full channel cut, a wing dike shall be provided to direct spillway flows away form the downstream toe of the dam. Ready access to emergency spillway system shall also be provided.

The slope of the exit channel usually follows the configuration of the abutment, unless exit velocities are excessive. A flatter slope with a widening of the spillway channel bottom width, should reduce exit velocities to a level where erosion will not pose a problem. In cases of highly erodible soils, it may be necessary to use other means of protection such as rip rap, grouted rock or concrete paving to form the exit channel. Especially adapted grasses can also provide a stabilizing effect and reduce erosion in the exit channel.

The outlet channel slope should be greater than the critical slope to assure that flow will be controlled at the spillway section. Otherwise, both weir control and open channel control should be evaluated over the range of headwater depths.

11.3.3 Embankment Structures

The width of the top of the embankment structure shall be a minimum of 12 feet for embankments less than 25 feet in height. Also, side slopes on embankment structures will vary with materials types used and shall be designed to produce a stable and easily maintained structure. A slope stability analysis shall be required on all Class 1 structures.

An allowance for settlement shall also be factored into the design for all embankment structures. Consideration shall also be given to limiting excessive seepage through the embankment and foundation that may lead to embankment erosion and structure instability for all Class 1 structures.

A geotechnical analysis and report prepared by a Colorado Professional Engineer with recommendations for the foundation preparation and embankment construction shall be submitted to the City/County Engineer with the complete design analysis for all permanent detention facilities.

11.4 Hydraulic Design Methods

Two methods suggested for estimating peak flows and runoff over time for use in detention pond sizing include the Rational Stored Rate Method and SCS Hydrograph Procedure. These methods are outlined in the following paragraphs. The principles of both methods are addressed in detail in the American Public Works Association (APWA) report "Urban Stormwater Management," 1981.

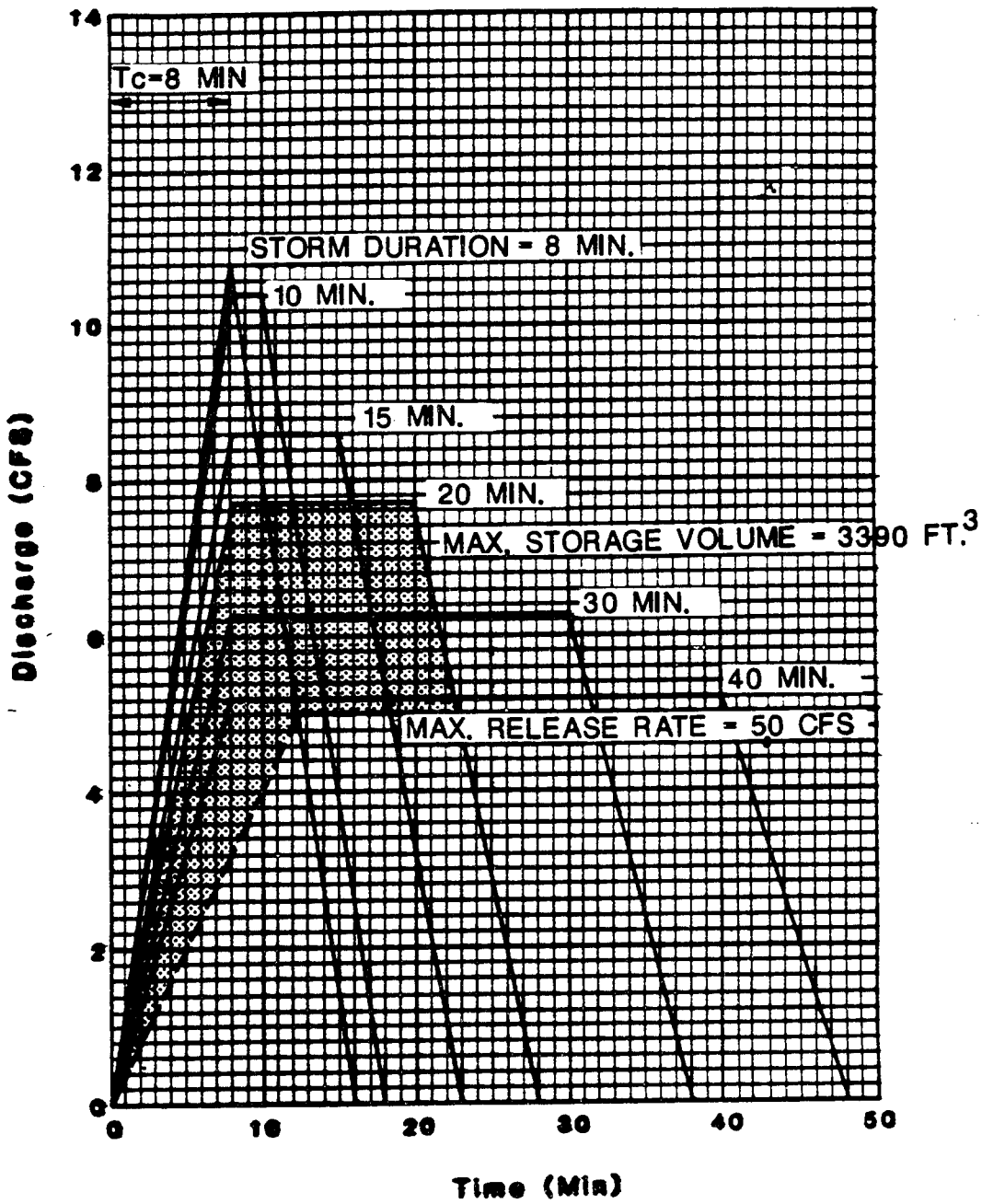
11.4.1 Rational Stored Rate Method

The Rational Stored Rate Method refers to a procedure for utilizing the basic Rational Method to reflect the fact that storms with durations greater than the estimated runoff time of concentration for a given drainage basin will result in a larger volume of runoff even though the peak discharge is reduced. This greater volume of runoff produced by longer storm durations must

be analyzed to determine the correct sizing for storm water detention storage facilities. The method considers net storage required for storms of various durations until maximum storage requirements are determined. The procedure should be limited to relatively small areas with contributing drainage areas 20 acres or less.

Figure 11-2, Rational Stored Rate Method Hydrographs, presents a family of curves for a theoretical basin. These hydrographs were developed by utilizing the basic Rational Method assumptions of constant rainfall intensity, runoff time of concentration for the longest flow path, and the coefficient of runoff. The typical Rational Method hydrograph with the peak discharge coinciding with the runoff time of concentration for the basin (T_c) is first calculated using the formula Q = CiA.

Following this, a family of hydrographs representing storms of greater duration are developed. The peak runoff rate for each hydrograph is equal to CiA where i is the rainfall intensity for the design storm duration under consideration. The rising limb and falling limb of the hydrograph are, in each case, equal to T_c for the basin. The area under the hydrograph is also equal to the peak discharge rate for that particular rainfall multiplied by the duration of the rainfall.



Reference: American Public Works Association, 1981; Urban Stormwater Management, Special Report No. 49.

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Drainage Criteria Manual

Modified Rational Method Hydrographs

Date
OCT. 1987

Figure
11-2

The following example presents the calculation method for a hypothetical, yet typical two (2) acre basin.

EXAMPLE 1

Given: Location: Anywhere, U.S.A.
Area: A = 2.0 acres
Type of development: commercial parking lot, fully paved, C = 0.9
Runoff Time of Concentration: Tc = 8 minutes
Design Storm Frequency = 10 years
Use applicable Rainfall-Intensity-Duration Curves

Required: Develop family of curves representing Rational Stored Method hydrographs for the 8-, 10-, 15-, 20-, 30- and 40-minutes storm rainfall durations for sizing proposed detention facility.

Storm Rainfall Duration (min)	Storm Rainfall Intensity (in/hr)	Flood Peak Runoff Rate (cfs)
8	6.0	10.8
10	5.8	10.4
15	4.8	8.6
20	4.3	7.7
30	3.5	6.3
40	2.9	5.2

Answer: The estimated storm hydrographs are shown on Figure 11-2

The next step in determining the necessary storage volume for the proposed detention facility is to establish a design maximum pond release rate and determine the volume of storage necessary to accommodate this release rate.

Figure 11-2, Rational Stored Rate Method Hydrographs, represents a method for detention pond sizing in small drainage areas. The assumed maximum release rate curve approximates a formal reservoir routing in much the same way the Rational Method Hydrograph approximates a true storm hydrograph. The curve allows for a low release rate at the beginning of a storm increasing to the assumed maximum release rate as the storage volume increases.

In normal flood routing procedures, the maximum release rate will always occur at the point where the outflow hydrograph crosses the receding limb of the inflow hydrograph. For this reason, the design release rate is forced to coincide with that point on the falling limb of the hydrograph resulting from the storm of duration equal to the runoff time of concentration for the basin.

The release rate is held constant past this point. The storage volume is then found by determining the total area between the inflow and release rate hydrographs.

To determine and confirm the required storage volume, reservoir routing should be performed for several of the hydrographs, with the critical storm duration and required storage volume being determined. The importance of the particular project should govern the type of routing model utilized. For small areas requiring repetitive calculations, such as in bays of a parking lot, an assumed nominal release rate curve (i.e., 0.1-0.5 cfs) is commonly utilized. For larger areas, such as a pond in a small park with 5 acres or more of contributing drainage area, a reservoir routing procedure would be in order.

Example 2 continues the calculations initiated in Example 1 to determine the required storage volume.

EXAMPLE 2

Given: Drainage basin characteristics and other hydrologic information presented in Example 1.

Allowable maximum release rate: Q_o = 5.0 cfs

Required: Determine the required storage volume.

Storm Duration (min)	Required Required Storage Volume (ft^3)
8	2780
10	3240
15	3315
20	3390 - maximum
30	2640
40	1005

The required storage volume is estimated at 3,390 ft^3 occurring for a 20-minute storm rainfall duration.

The limitations in the assumptions for this method are evident. The approach becomes more valid on progressively smaller basins. The procedures should, therefore, be limited to small drainage areas 5 acres or less where no major damage would result from overtopping or failure of the proposed detention facility.

11.4.2 SCS Hydrograph Procedure for Storage Analysis

The SCS Hydrograph Procedure is used to develop a hydrograph which provides a reliable solution for sizing detention storage facilities. Procedures provide the designer greater flexibility for the representation of actual conditions to be modeled.

The development of a SCS total runoff hydrograph is presented in Section II of this manual. A flood routing procedure may be used to determine the volume requirements for a given detention basin. Several flood routing procedures are available in published texts. The data needed for the routing computations are the inflow hydrograph, the physical dimensions of the storage basin, the maximum outflow allowed, and the hydraulic characteristics of the outlet structure or spillway.

After the inflow hydrograph, depth-storage relationship, and depth-outflow relationship have been determined, they are combined in a routing routine.

The results of the routing analysis include the outflow hydrograph, the depths of storage, and the volumes of storage at each point in time of the flood duration. The Modified Puls routing model, commonly utilized in practice, is based upon the principles discussed in the following paragraphs.

The routing period, or time interval, Δt, is selected small enough so that there is a good definition of the hydrograph and the variation in the hydrograph during the period Δt is approximately linear. This can be accomplished by setting $\Delta t = 5$ minutes, or to the same time interval as in the SCS Hydrograph procedure.

Several assumptions are made in using this model and include the following:

1. The entire inflow hydrograph is known.

2. The storage volume is know at the beginning of the routing.

3. The outflow rate is known at the beginning of the routing.

4. The outlet structures are such that the outflow is uncontrolled and the outflow rate is dependent only on the headwater.

The derivation of the routing equation begins with the conservation of mass which states that the difference between the average inflow and average outflow during some time period, Δt, is equal to the change in storage during that time period. This can be written in equation form as:

$$\bar{I} - \bar{O} = \Delta S / \Delta t \qquad (11\text{-}5)$$

If inflow during the period is greater than outflow, then S is positive and the pond gets deeper. If inflow is less than outflow during the period, then S is negative and the pond gets shallower. Using the assumptions made previously, this equation can be rewritten as:

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{S_2 - S_1}{\Delta t} \qquad (11\text{-}6)$$

Multiplying both sides by two and separating the right-hand side yields:

$$I_1 + I_2 - O_1 - O_2 = \frac{2S_2}{\Delta t} - \frac{2S_1}{\Delta t} \qquad (11\text{-}7)$$

Rearranging so that all the known terms are on the left-hand side and all the unknown terms on the right-hand side yields the final routing equation:

$$(I_1 + I_2) + \frac{(2S_1}{\Delta t} - O_1) = \frac{(2S_2}{\Delta t} + O_2) \qquad (11\text{-}8)$$

However, the last equation has two unknowns, S_2 and O_2. Therefore, a second equation which relates storage and outflow is needed. If outflow is a direct function of reservoir depth (as it is with uncontrolled outflow), there is a direct relationship that exists among reservoir elevation, storage and outflow. Therefore, for a given elevation, there is an answer for storage and outflow (S and O). A relationship between O and $(2S/\Delta t)+O$ is determined for several elevations and plotted on logarithmic graph paper. The routing equation is solved by adding all the known terms on the left-hand side. This yields a value for $(2S/t)+O$. This value is found on the log-log plot of $(2S/\Delta t)+O$ versus O and the value of O_2 is obtained from the graph.

The $(2S/\Delta t)+O$ versus O relationship is derived by combining the depth-storage relationship and the depth-outflow relationship, as previously discussed. This is presented in Table 11-2. Columns 1, 2 and 3 are tabulations of the depth-storage and depth-outflow relationships for a specific detention facility.

In column 4, the units of $2S/\Delta t$ and O must be identical. If O is in cfs, then $2S/\Delta t$ must be changed to cfs. For a routing time interval of 5 minutes:

$$\frac{2S \text{ ac-ft}}{5 \text{ min}} \times \frac{1 \text{ cfs-day}}{1.98 \text{ ac ft}} \times \frac{1440 \text{ min}}{1 \text{ day}} = 291S$$

Thus, $$\frac{2S + O}{\Delta t} = 291S + O \quad \text{for} \quad t = 5 \text{ min}$$

where: S has units of acre-feet, O has units of cfs, and $2S/\Delta t$ has units of cfs.

TABLE 11-1

Development of a (2S/Δt) + O vs. O Relationship

Depth (ft) (1)	Storage, S (ac-ft) (2)	Outflow, O (cfs) (3)	(2S/Δt) +O (cfs) (4)
0	0.0	0	0
2	0.1	40	69
4	0.6	138	313
6	3.0	274	1,147
8	11.0	426	3,627
10	32.0	560	9,872
12	72.0	671	21,623
14	131.0	765	38,886

The sizing of detention pond outlet works is a matter of judgement depending upon the actual conditions for the specific case. The designer may approach the sizing of the outlet works on a trial and error basis with the objective being the optimum use of the available storage capacity of the ponding area. In addition, all ponding areas must be carefully analyzed in regard to the major storm runoff conditions. In many cases, it will be found that the initial storm runoff should be routed through the outlet works with only minimal ponding. Thus, available storage would then be utilized to reduce the major storm runoff impact.

If other provisions are made for the major storm runoff, the designer may be primarily concerned with the initial storm drainage system. The outlet capacity would then be substantially less than the inflow from the initial storm runoff. It cannot be overemphasized, however, that the use of detention pond storage requires competent planning and design to avoid the creation of an unnecessary hazard.

11.5 Bibliography

1. Poetner, Herbert G., Practices in Detention of Urban Storm Water Runoff, American Public Works Association Special Report No. 43, Chicago, IL, 1974

2. Ward, Andy, Haan, Tom, and Tapp, John, The DEPOSITS Sedimentation Pond Design Manual, University of Kentucky Institute for Mining and Minerals Research, Lexington, KY, 1979.

3. Wright-McLaughlin Engineering, Urban Storm Drainage Criteria Manual, Denver Regional Council of Governments, CO, 1969.

4. Colorado Office of the State Engineer, "Design Review Manual" for Dams and Dams Safety, July 31, 1986

5. American Public Works Association (APWA), Urban Stormwater Management, Special Report No. 49, 1981.

6. U.S. Department of Commerce, "Nation Engineering Handbook, Hydrology, Section 4," August 1972.

7. Office of the State Engineer, Division of Water Resources, "Rules and Regulations for Dam Safety and Dam Construction," August, 1988.

STORAGE ROUTING FORM

1. **INFLOW HYDROGRAPH GENERATION**
   a. Hydrograph method used:
   b. Time interval selected for routing:
   (inflow hydrograph attached)

2. **APPROXIMATE FLOW REDUCTION DUE TO ROUTING**
   a. Peak inflow: Q_p = _____ ft^3/s
   b. Upstream storage: S = _____ ft^3
   c. Time to peak: t_p = _____ min

$$Q_r = Q_p - \frac{S}{80\, t_p} =$$

3. **ELEVATION - DISCHARGE RELATIONSHIP FOR TRIAL CULVERT**

ELEVATION ft								
DISCHARGE ft^3/s								

4. **ELEVATION - STORAGE RELATIONSHIP FOR UPSTREAM PONDING**

ELEVATION ft	AREA ft^2	INCREMENTAL VOLUME ft^3	ACCUMULATED VOLUME ft^3

5. **STORAGE - OUTFLOW RELATIONSHIP**

Elevation ft	Discharge (O) ft^3/s	Storage (S) ft^3	$2S/\Delta t$ ft^3/s	$2S/\Delta t + O$ ft^3/s

6. **STORAGE - INDICATION ROUTING TABLE**

(1) TIME min	(2) INFLOW (I) ft^3/s	(3) $2S/\Delta t - O$ ft^3/s	(4) $2S/\Delta t + O$ ft^3/s	(5) OUTFLOW (O) ft^3/s

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Figure
11 - 3

ATTACHMENT A

DESIGN ENGINEER CHECK LIST

FOR STATE OF COLORADO JURISDICTIONAL DAMS

I. SUBMITTAL REQUIREMENTS

A. General list of items to be submitted for preliminary review. (A hydrology report will be reviewed on a time available basis if not accompanied by the remaining items.) The following items must be on file to begin the 180 day time limit:

_____1. Hydrology report including spillway design, incremental damage analysis as required, and hazard classification.

_____2. Two copies of the geotechnical report.

_____3. Two complete sets of blue/black line drawings.

_____4. Two complete sets of technical specifications.

_____5. An engineer's cost estimate in sufficient detail to evaluate the cost of construction.

_____6. A filing fee as required by statute.

_____7. Design Engineer Check List and Project Information Record.

_____8. Engineer's Design Report showing all calculations, references, and assumptions.

_____9. Application for Construction or Enlargement.

B. Rejected Plans and Specification: Project re-submittals subject to 180 day time limit.

_____1. Two complete sets of plans with corrections based upon comments from the previous review.

_____2. Two complete sets of specifications with corrections based upon comments from the previous review.

_____3. A transmittal letter explaining location and character of changes in the plans and specifications which were made by the engineer and not in response to the previous review comments.

_____4. Corrected or updated reports as required.

_____5. New Filing Fee

_____6. Revised Cost Estimate

REFERENCE: Colorado Office of the State Engineer; July 31, 1986.

C. Final submittal of plans and specifications.

_____1. Three complete sets of blue/black line prints.

_____2. Minimum of four complete sets of technical specifications.

a. At least one copy must have original engineer's signature and seal.

b. Three sets will be retained by the state engineer. The remaining copies will be sent to the design engineer.

_____3. The original mylar cover sheet with original signatures and a rubber stamped engineer's seal.

a. If the engineer wishes to keep the original mylar for his record, a high quality, reproducible mylar cover sheet with original signatures and seal, must also be submitted.

b. All mylar cover sheets will be returned by the State Engineer for future use with the "AS-BUILT" mylar drawings.

II. <u>HYDROLOGY AND HYDROLOGIC DAM BREAK ANALYSIS</u>

A. The hydrology portion of the report should contain the following:

_____1. Topo map of drainage area showing:

a. Bearing and distance from the dam's Station 0+00 to a section corner.

b. Name of tributary or drainage impounded.

c. Elevation of dam crest.

_____2. Drainage area in acres and square miles.

_____3. Raise in basin along the longest water course.

_____4. Length of longest water course.

_____5. Method and computations for determining the time of concentration.

_____6. Description of basin hydrologic conditions and soil conditions.

_____7. References and methods used to determine design precipitation.

_____8. References and methods used to determine the unit hydrograph.

_____9. Computations of the general (areal) and thunderstorm hydrographs as applicable.

____10. Sketch of proposed spillway showing elevations dimensions, slope(s) of spillway chute.

____11. Equation(s) and spillway discharge capacity table.

____12. Reservoir elevation area/storage table from lowest level to the crest of the dam.

____13. Outlet discharge capacity table and equations for each foot of head beginning at inlet and continued to the crest of the dam.

B. The Hydrologic Dam Break Analysis for spillway design less than PMF should include the following:

_____1. Flood plain map of inundated critical areas showing pre and post failure high waterlines and the waterlines of the inflow flood without the dam.

_____2. Cross-sections to scale with elevation and channel widths. Showing pre and post failure and the no dam flood waterlines including discharges and average velocities.

a. Just below dam.
b. Intermediate channel sections.
c. Critical sections within the inundated critical area.

_____3. Hydraulic parameters assumed.

_____4. Sensitivity analysis of assumptions; i.e., time to complete failure, depth of overtopping, channel roughness (mannings "n").

_____5. Estimated probability of the failure of the dam by overtopping based on reoccurrence interval of flood.

_____6. Discussion of why the damage resulting from an overtopping dam failure is insignificant.

_____7. Computer printout of the dam break flood complete with explanation of parameters and numbers.

III. HAZARD RATING DAM BREAK

A. As a separate report or a report section of the hydrology report, the engineer shall evaluate the hazard class of the dam.

Criteria shall be as follows:

_____1. The non-hydrologic failure shall consider the reservoir full at the normal high water line.

_____2. Several reasonable failure rates shall be evaluated to determine the sensitivity of assumed parameters.

_____3. Analysis of flood inundation shall continue downstream until the flood stage is equal to or less than the 100-year floodplain.

B. The report shall include the following:

_____1. Floodplain maps showing the inundated floodplain areas.

_____2. Cross-sections to scale showing elevations and channel width. The discharge and average velocity shall be shown for each section.

_____3. Tabulation of parameters assumed.

_____4. Sensitivity analysis of assumed parameters.

_____5. Computer printout of the dam break flood.

_____6. Include a reference to all programs, data, and documents used.

_____7. Conclude with a recommended hazard rating for the dam based upon criteria shown in the Technical Guidelines, Section I.

IV. GEOTECHNICAL REPORT

A. For any dam which has a hazard rating above Class 3, the following must be evaluated and included in the geotechnic investigation:

1. Foundation:

_____a. In situ samples (undisturbed)

_____b. Field classification

_____c. Logs of test holes

_____d. Penetration tests

_____e. Ground water level

_____f. In situ permeability

_____g. Density

_____h. Dispersive clays

_____i. Rock quality

_____j. Piping potential between geologic layers; i.e., evaluation of gradation

_____k. Bedrock jointing, RQD, etc.

2. Borrow:

_____a. Disturbed samples

_____b. Gradation curves

_____c. Lab classification

_____d. Consolidation, compressibility, and expansion

_____e. Permeability

_____f. Strength tests of materials

_____g. Moisture density curves

_____h. Dispersive clay assessment

_____i. Verified quantity of borrow materials to complete the proposed dam as planned includes all zones and drain materials. Alternative borrow areas if estimates of materials available are incorrect.

_____j. Evaluation of compatibility of materials against piping.

3. Geologic Assessment: Class I and Class II dams

_____a. Local and dam foundation

_____b. Stability of land mass around reservoir perimeter

_____c. Faults

_____d. Seismicity of area

4. Stability analysis of embankment design:

_____a. Rapid drawdown.

_____b. Steady state.

_____c. Earthquake analysis as required. Class 1 - designed to withstand maximum credible earthquakes.

_____d. Discussing of conditions which infer stability if U.S.B.R. Design of Small Dams criteria is used and allowed for the structure.

B. All Class 3 and Class 4 dams shall have at least the following items included in the geotechnical report:

_____a. Foundation drill logs to competent foundation

_____b. Standard penetration tests

_____c. Soil classification

_____d. Gradation analysis of borrow materials

_____e. Proctor density curves

V. PLANS AND SPECIFICATIONS CHECK LIST

A. Plans Check List

1. Plan Identification

_____ Title Block on each sheet of plans

_____ Approval Statements on cover sheet of plans

_____ Drawings: Consecutive sheet/page numbers and design engineers seal/signature

_____ Space for State Engineer's File Number 1/2" X 2" in lower right-hand corner on each sheet.

2. Location and Vicinity Maps Showing:

_____ Bar Scale and North Arrow

_____ Project location and shipping points

_____ Public Land Grid

_____ Drainage area and topography

_____ Streams and Gaging Stations

_____ Roads, Utilities

3. <u>Plan of Reservoir Area Showing:</u>

_____ Bar Scale & North Arrow

_____ M.S.L. Elevations

_____ Bearings of dam centerline

_____ Public Land Grid

_____ Topography

_____ Cultural Features

_____ N.W.L. Traverse and Dead Storage Area & Volume

_____ Spillway Location to scale

_____ Reservoir Capacity at Spillway elevation (A.F.)

_____ Clearing Areas/Limits

_____ Embankment Borrow Areas

_____ Riprap Borrow

_____ Waste Areas

_____ Drill Hole & Test Pit Locations

_____ Roads, Utilities, Streams, etc.

_____ Land Ownership & Boundaries

4. <u>Dam and Spillway Plan</u>

_____ Scale & North Arrow

_____ M.S.L. Elevations

_____ Section Corner Tie & Bearings

_____ Topography

_____ Drill Holes & Test Pit Locations

_____ Drainage Blanket Limits

_____ Location of Cross-Section(s)

_____ Spillway Alignment

_____ Centerline Stationing

_____ Structure Locations

_____ Riprap Placement Limit

_____ Logs of Drill Holes & Test Pits

_____ Location of Instrumentation

5. Outlet, Maximum, and Typical Cross-Sections of Dam

_____ Crest Width

_____ Crest Elevation

_____ High Water Elevation

_____ Normal Water Elevation

_____ Upstream Slope

_____ Downstream Slope

_____ Riprap Thickness

_____ Bedding Thickness

_____ Computed Wave Height

_____ Additional Sections for Special Construction

_____ Outlet Pipe Intake Elevation

_____ Outlet Pipe Discharge End Elevation

_____ Outlet Pipe Materials

_____ Outlet Pipe Slope, Length and Diameter

_____ Cutoff Trench Depth & Width

_____ Crest Slope

_____ Location Cutoff Collar

_____ Embankment Zones

_____ Toe drain, drainage blanket, & chimney drain

6. Longitudinal Section Through Dam (Profile)

_____ Crest Elevation

_____ Cutoff Trench

_____ Stationing

_____ Drill Holes & Test Pit Locations

_____ Soils Logs on Profile

_____ Camber

7. Outlet Pipe

_____ Diameter

_____ Length

_____ Slope

_____ Material

_____ Gage or class

_____ Trenching & Structural Backfill Details

_____ Capacity Curve with Equation(s)

_____ Maximum Velocity (N.W.L.)

_____ Vented

_____ Bedding Requirements

_____ Cavitation

8. Cutoff Collars

_____ Size

_____ Material

_____ Number

_____ Spacing

_____ 20 Percent Increase in Seepage Path

_____ Molded Filler

_____ Acceptable Location

_____ Batter

9. Intake and Lift Structure

_____ Material

_____ Trashrack

_____ Trashrack Velocity

_____ Gate Type & Head Rating

_____ Gate Lift Mechanism Accessible

_____ Reinforcing Clear Distance

_____ Gate Lift or Wheel

_____ Size Gate Stem & Housing

_____ Adjustable Stem Support

_____ Stem Protection From Ice

_____ Gate Lift Lockable

10. Outlet Structure & Channel

_____ Type

_____ Material

_____ Stilling Basin

_____ Exit Velocity

_____ Riprap Protection

_____ Downstream Channel Capable of Carrying Maximum Outlet Pipe Discharge

11. Spillway

_____ Type

_____ Size

_____ Material

_____ Slopes

_____ Riprap Thickness

_____ Bedding Thickness

_____ Gross Freeboard

_____ Net Freeboard

_____ Profile to Natural Channel

_____ Cross-Sections of Channel

_____ Cavitation Potential

_____ Reinforcing Clear Distance

_____ Debris Barrier or Trashrack

_____ Stilling Basin

_____ Capacity Curve with Equation

12. Other

_____ Area in Acres & Storage Capacity Curve(s) in Acre-feet

_____ Instrumentation Details

_____ Reservoir Gage Details

B. Specifications

1. Front Cover

_____ Title or Name of Dam (identical to plans)

_____ Water Division _____, Water District _____

_____ County

_____ Design Engineers Seal and Signature

_____ State Engineer's approval statement

2. Index to specifications

3. Do specifications include the following under general conditions?

_____a. Provision for an engineer overseeing quality control of construction.

_____b. Provision that the plans and specifications may not be materially changed in any particular except with the written consent of the State Engineer.

_____c. Provisions for developing AS-BUILT drawings.

_____d. Provision that construction cannot be considered complete until the state engineer has approved the same in writing.

4. Technical

_____ a. Are the specifications compatible with the plans?

_____ b. Are all materials specified and include reference to:

_____ (1) Quality of materials, type

_____ (2) Installation/Workmanship

_____ (3) Reference to standards

_____ (4) Action to be taken for unsatisfactory materials or workmanship

_____ (5) Required tests and estimated frequency of testing

PROJECT INFORMATION RECORD

Name of Dam _____ Class: _____

Name of Impoundment _____

Other Names of Record _____

Location: _____, Sec. _____, Twnshp. _____, Rng. _____ P.M. _____

Water Division _____, Water District _____

Latitude _____, Longitude _____

County _____

Congressional District _____

Nearest Town Downstream _____

Flood Thalweg miles to nearest Town _____

Population of nearest Town _____

Owner Name and Agent _____

Owner Address, Street or Post Office Box _____

_____

City, State, ZIP Code _____

Owner Phone Number (____) _____

Engineer Name _____ Colorado P.E. # _____

Company _____

Address, Street, or Post Office Box _____

_____

City, State, ZIP Code _____

Engineer Phone Number (_____) _____

Stream _____ Tributary _____

Major Drainage Basin _____

BASIC DATA

Type of Dam

(CHECK AS APPLICABLE)

Concrete _____ Gravity _____ Arch _____

Earth _____ Zoned _____ Homogeneous _____

Rock _____ Facing or Membrane _____

Other _____

Comments: _____

_____

_____

Structural Dam

Height of Dam _____ ft. (Valley floor to crest)

Length of Dam _____ ft.

Crest Width _____ ft.

Crest Elevation _____ ft. M.S.L.

Freeboard _____ ft.

Volume of Dam _____ Cubic Yards

Structural Height _____ ft. (bottom of cutoff trench to crest)

Hydraulic Height _____ ft. (downstream toe to spillway)

Maximum Capacity _____ A.F. (to crest of dam)

Normal Reservoir Cap. _____ A.F.

Total Spillway(s) Cap. _____ CFS

Outlet Description _____ inches _____ type pipe

Maximum outlet capacity _____ cfs (head at spillway elevation)

Reservoir surface area N.H.W.L. _____ acres

Hydrologic Data

Drainage Area: _____ Acres _____ sq. mi.

Design Flood _____ Percentage of PMP _____

Peak Discharge _____cfs. Volume _____A.F.

Design Flood routes through reservoir with _____ ft. freeboard

SECTION III

Chapter 12 Flood Proofing

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CHAPTER 12 FLOOD PROOFING

12.1 General Statement

Flood proofing is defined by FEMA as any combination of structural and nonstructural additions, changes or adjustments to structures which reduce or eliminate flood damage to real estate or improved real property and sanitary facilities, structures, and their contents. Primary purposes of flood proofing structures are to reduce potential excessive property losses and to allow early return to normal occupation by residents soon after a flood has receded. The intent of flood proofing structures is not to provide self-contained structures for occupancy during a flood. Also, flood proofing may be permanent or contingent on some action at the time of flooding.

FEMA has published several references that provide detailed criteria and design procedures for flood-proofing structures. These and other key references are listed in the bibliography of this Chapter and should be consulted for detailed design information.

12.2 Flood-proofing Requirements

Any proposed non-residential structures located within the 100-year flood plain should either have the lowest floor elevated to one foot above 100-year flood elevation or should be flood-proofed to that elevation. The 100-year floodplain and corresponding maximum elevations of the 100-year flood can be determined by reviewing the Flood Insurance Rate Maps published by FEMA and available for review in the City/County and FEMA offices (Regional Building Department - Regional Floodplain Administrator).

12.3 Types of Flood Proofing

There are two basic systems for flood proofing for residential and commercial structures. One system consists of the use of conventional construction methods along with a total drain, sump and pump operation to keep water away from the lower portions of the structure. This type of system is feasible in regions with soils having low coefficients of permeability (i.e., clay soils). This Drain or Sump System accepts the inevitability of some subsurface infiltration from a high head of water into a reasonably economical basement construction via the sump. The sump type basement protection system is adaptable to low and medium volumes of inflow of flood waters within the capability of its drainage and pumping system. This system is only applicable to structures having a grade line above the 100-year maximum flood elevation on all sides.

The other system utilizes a watertight wall and slab treatment for all portions of the structure beneath the 100-year maximum flood elevation. This type of system is required for soils having a high coefficient of permeability (i.e., sandy soils). This is termed an Undrained or Barge System and is required where inflow is beyond the capacity of a sump drain system or where the grade line of the proposed structure cannot be located above the 100-year maximum flood elevation. An undrained structure is designed to be watertight. All openings below the 100-year maximum flood elevation are equipped with flood-proofed, watertight closures.

Both of these systems will allow flood proofing of structures up to five (5) feet above the bottom of the lowest floor slab. This height restriction is due mainly to buoyancy considerations. Special design features will allow flood proofing to higher elevations.

12.4 Flood-proofing Procedures

Several general procedures to achieve flood proofing are discussed in this chapter. Some of these procedures, such as laying out sites and raising buildings, are intended primarily for new construction which would represent a proper use of flood plain sites. Other procedures such as those to keep the water out of structures or those to minimize losses if the water gets in, would apply to both new and existing structures.

12.4.1 Site Layout

The practice of clustering buildings is prevalent in planned unit developments. This clustering permits buildings to be attractively grouped on parts of a site which are above flood levels and reserves the low-lying sections as landscaped green areas and parking facilities.

Where natural high ground does not exist, sites can be raised by filling, providing the fill does not interfere with the flow of flood waters (i.e., the structure is not located within the floodway). When this is done, a structure's appearance can be preserved and in some cases even enhanced by flood proofing.

12.4.2 Elevated Structures

The practice of elevating a building on "stilts" to provide an "open" effect at ground level can also reduce the flood hazard. If some means of access is maintained and utilities can continue to function, activities would not be interrupted during floods.

Where land is at a premium, as in central business districts, buildings are often placed on stilts with parking facilities on the ground level.

12.4.3 Waterproofing Structures

The design procedures discussed above either achieve flood proofing through site planning and development or incorporate flood proofing in the initial construction of a building(s). The location and environment of structures in urban areas may make these solutions impractical. These flood-proofed buildings can incorporate many contemporary design features such as large window areas, pedestrian arcades, open floor space, and curtain wall panels.

In designing new structures, or in altering existing one's, consideration should be given to the use of recessed flood shields which are normally hidden from view, but can be easily lowered or slid into place once a flood warning is issued. When flood shields must be mounted on the street side of an opening, the brackets to which they would be bolted can be concealed with easily removed aluminum strips or "skins."

12.4.4 Internal Flood-proofing Measures

Owners of buildings which are subjected to flooding inundation but which cannot be easily altered to keep the water out can consider the use of water-resistant construction materials to reduce flood damage. Even owners of flood-proofed structures generally able to withstand flooding would also be well advised to consider the use of such materials to reduce losses when flooding exceeds the protection level.

12.5 Engineering Considerations of Flood Proofing

Engineering design considerations in flood-proofing structures are presented in the several references identified in the bibliography of this Chapter. These include structural-related problems and structural engineering, subsurface drainage, and flood-proofing operations. Because of the complexity of the related engineering design factors and potential problems, building owners who are contemplating flood proofing should engage the services of a professional engineer who has a working knowledge of structures and who has had experience with hydraulic structures and flood proofing.

12.6 Bibliography

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9/30/90

APPENDIX

DESIGN EXAMPLES

The following examples represent a simplified version of a typical Subdivision Drainage Design. The examples are intended to demonstrate the specific design methods presented in Chapters 7 through 11. These examples also show the interrelationship between the various methods in design applications. For a detailed discussion of these methods, consult the appropriate sections of this manual and their respective bibliographies.

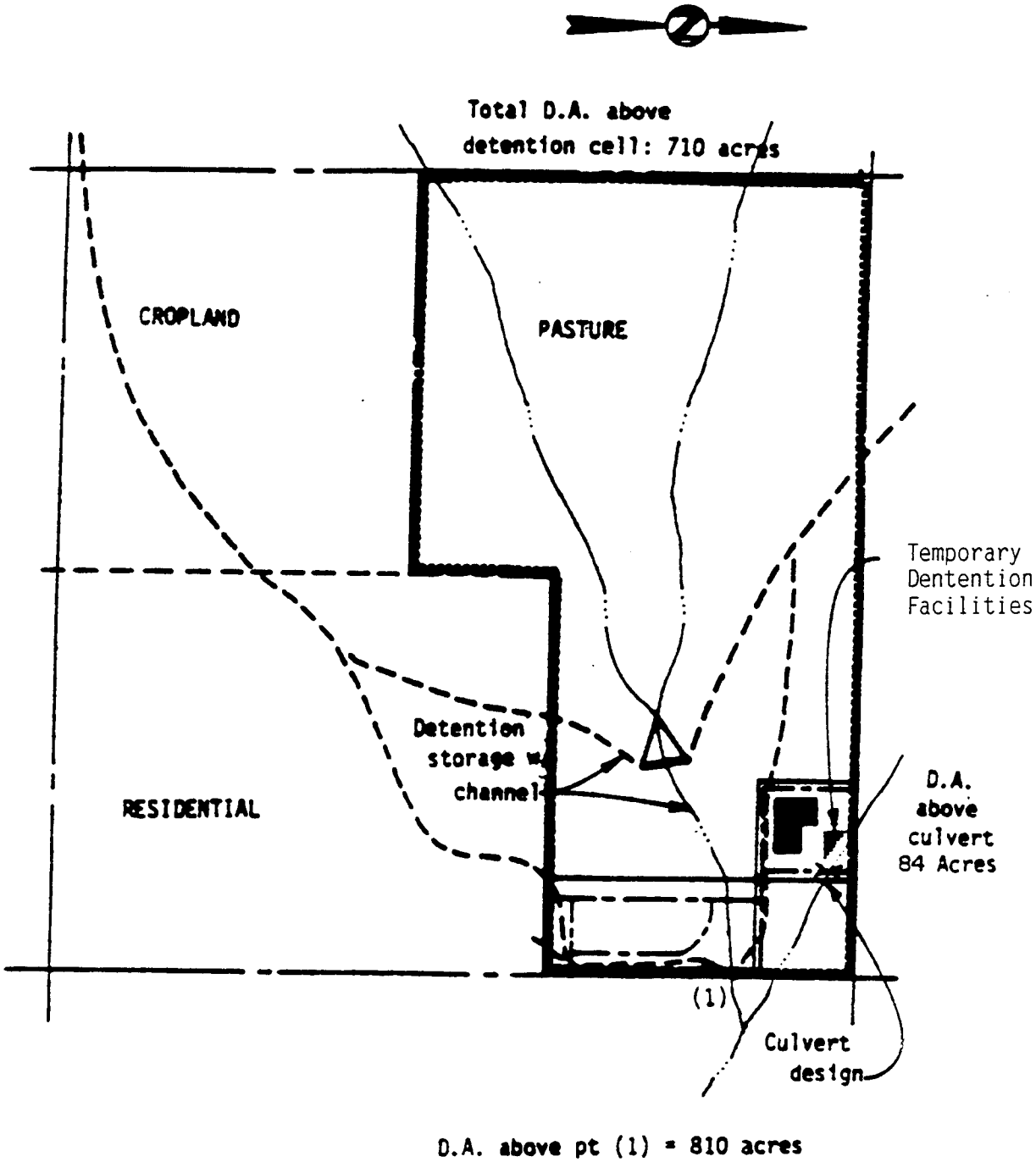
GENERAL

A developer owns 300 acres which he intends to develop in segments over the next 10 years. The first area to be developed will be a 30-acre tract containing 20 acres of single-family residential lots of approximately 11,250 square feet and a 10-acre shopping center.

Due to limited resources, the developer intends to use storm water management techniques to limit runoff and minimize his overall expenditures for drainage facilities.

Figure A shows the drainage basin, the proposed development, and drainage facilities. The drainage plan utilizes an on-site detention pond to reduce the peak runoff from the majority of the basin, thus reducing the size of required trunk sewer through the proposed development. The proposed shopping center will utilize temporary on-site detention to reduce peak runoff until the regional detention facilities are completed. Temporary detention had to be used to provide the peak reduction as no other practical method was available.

Figure B shows the proposed residential development layout, with appropriate dimensions and design street grades.



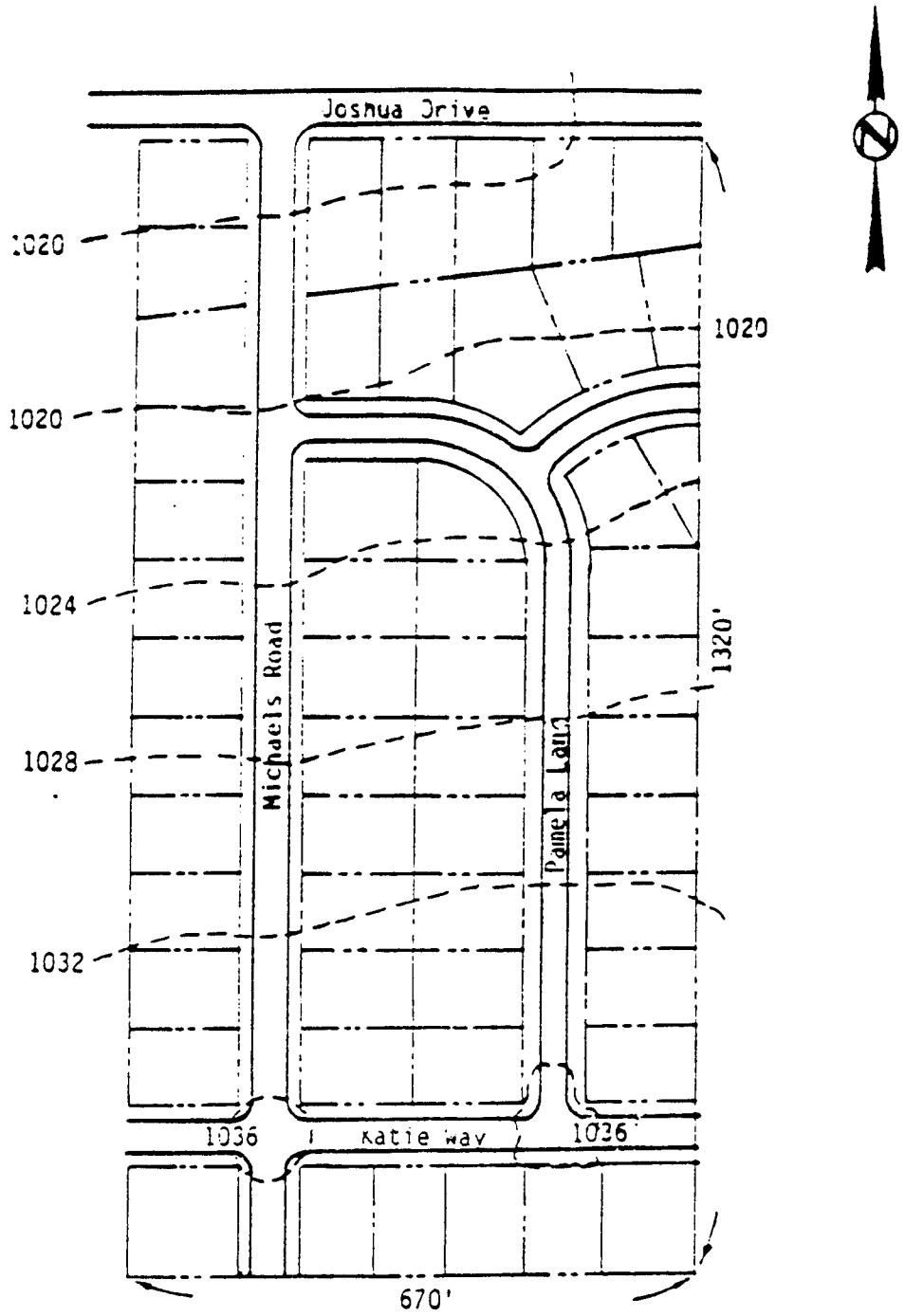
HDR Infrastructure, Inc.
A Centerra Company

The City of Colorado Springs / El Paso County
Drainage Criteria Manual

Date
12-22-86

Figure
A

Overall Site Plan A-3



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Sub-division Layout A-4

Date
12-22-86

Figure
B

STREET DRAINAGE

Michaels Road is a 60′ wide minor arterial street on a 2% grade, and drains a uniform width of 330′ along its length. Katie Way is also a 60′ wide minor arterial, but is graded at 0.8% and -0.8%, creating a sump curve in the middle of the block. It drains a total area of 1.82 acres, with 80% contributing to the east inlet. Pamela Lane is a 28′ wide hillside minor residential street, and drains a width of 280′ through its length.

Using the criteria of Chapters 6 and 7, design the surface drainage system for these streets.

ALLOWABLE GUTTER FLOW

Initial Storm

Michaels Road:

Given: 8" vertical curb
2% pavement cross slope
60′ street width
2% grade

Find: Allowable gutter flow for initial storm of 10-year frequency.

1. Determine allowable pavement encroachment.

   Use Table 6-1: 20′ maximum flow spread from each curb face.

2. Calculate theoretical capacity for each gutter.

   Use Figure 7-2
   $n = 0.016$
   $z = 1/Sx = 1/0.02 = 50$
   $y/n = 3{,}125$
   $y = 20 \times 0.02 = 0.40$
   $q = 21$ cfs

3. Calculate allowable gutter capacity -- no reduction necessary and $q = 1.0 \times 21$ cfs.

4. Therefore, allowable gutter capacity for Michaels Road is 21 cfs.

5. Using same procedures, allowable gutter capacity for Katie Way is 14 cfs.

Pamela Lane:

Given: 6" vertical curb
2% pavement cross slope
28′ wide street
2% grade

Find: Allowable gutter flow for initial storm.

1. Determine pavement encroachment.

   Use Table 6-1: Flow may not overtop curb
   (28′ ÷ 2) = 14′ = encroachment

2. Calculate theoretical capacity

   Use Figure 7-2
   n = 0.016
   z = 50
   z/n = 3,125
   y_a = 0.50′
   q_a = 40 cfs
   y_b = 0.50 - (14 x 0.02) = 0.22′
   q_b = 4 cfs
   q = qa - qb = 40 cfs - 4 cfs = 36 cfs

3. Allowable gutter capacity is 36 cfs.

<u>Major Storm</u>

Given: Property lines of 80′ R.O.W. are one foot higher than curb flow line, along all streets.

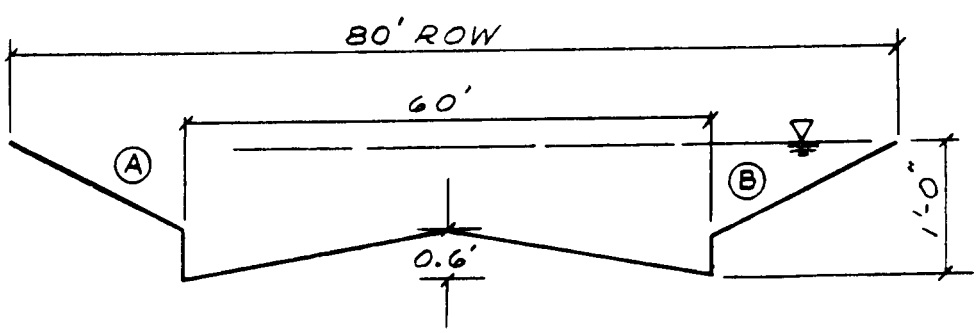
Find: Encroachment due to major storm.

1. Allowable depth of flow.

   Use Table 6-1: 12" above gutter flow line for local and minor arterial.

2. Street capacity: arterials

USE MANNING'S EQUATION



3. Assume areas A and B are non-effective for flow, due to obstructions.

4. Major storm capacity

$$Q = \frac{1.49}{n} AR^{2/3}S^{1/2}$$

$$A = (60' \times 1.0') - (60' \times \frac{0.6}{2}) = 42.0 \text{ sf}$$

$$R = \frac{A}{WP} = \frac{42}{2(1.0) + 60} = 0.68'$$

$$Q = \frac{1.49}{0.016} (42)(0.68)^{2/3} (0.02)^{1/2}$$

$$Q = 428 \text{ cfs}$$

5. Area needed to exceed major storm discharge - 100-yr, 5-min = Tc.

c = 0.62 (weighted)
i = 9.0 in/hr (Figure 5-1)

$$A = \frac{Q}{ci} = \frac{428}{0.62(9.0)} = 77 \text{ ac}$$

but, D.A. above streets <77 ac; therefore, street has sufficient capacity.

INLET DESIGN

Using the gutter capacities for Michaels Road, Katie Way and Pamela Lane developed previously, determine inlet locations for continuous grade and sag conditions. Use the criteria presented in Chapters 6 and 7.

Inlets on a Continuous Grade

Curb-opening Inlet

Given: Michaels Road gutter flow

21 cfs/gutter
n = 0.016
S_o = 0.02 ft/ft (2%)
g = 0.02 ft/(2%)
T = 20' (required)
W = 2'

Find: Inlet length for curb-opening inlet for various interception rates.

1. $F_W = 16.4 \left[(T-2)S_x\right]^{1/6} S_o^{1/2}$ (Table 7-2)

$= 16.4 \left[18 \times 0.02\right]^{1/6} (0.02)^{1/2}$

$= 1.96$

2. $F_W T = 39.12$

3. $L_1 = 30.12$

$L_2 = 18.09$

$L_3 = 64.55$

4. $Q_2 = (L_2/L_1) \times Q_1$

$= 12.61$

5. Try $Q_i Q = 1.00, 0.8, 0.6$

$$\frac{Q_i}{Q} = \left(\frac{L_i}{L_3}\right)^{0.4} \text{ for } Q_i > Q_2,\ L_i > L_2$$

rewritten as

$$\frac{L_i}{L_3} = \left(\frac{Q_i}{Q}\right)^{2.5}$$

$\frac{Q_i}{Q}$	$\frac{L_i}{L_3}$	L_i
1.0	1.0	64.55
0.8	0.572	36.92
0.6	0.279	$18.01 < L_2$
0.65	0.341	22.01
0.71	0.425	27.43

An 18.09′ inlet would provide the most efficient inlet; however, use 20′ inlet.

6. For $L_i = 20'$, $\frac{Q_i}{Q} = 0.63$

$Q_i = 0.63 \times 21 = 13.2$ cfs

<u>Grate Inlet</u>

Check grate inlet for same location.

1. Flow depth at curb = 0.4′

2. Flow depth at outside edge of grate:

$$0.4 - 0.02\ (2.0) = 0.36'$$

3. Gutter discharge at outside edge:

$$z/n = 3{,}125$$

4. Discharge captured by grate

$$21 - 16 = 5 \text{ cfs}$$

Grate efficiency: (5/21 = 0.24 or 24% vs. 63% for curb opening inlet

Inlet Spacing

Check inlet spacing for curb-opening inlet design using Rational Formula.

Assume t_c = 5 min., 10-year storm.

First inlet:

$A = \frac{Q_i}{ci}$ (Q_i is max. amount of gutter flow allowed between inlets); c = 0.52 (weighted)

$$A = \frac{21 \times 2}{0.52 \times 6.0} = 13.46 \text{ ac} = 586{,}384 \text{ sf}$$

$$\text{Spacing} = \frac{\text{area}}{\text{width}} = \frac{586{,}384 \text{ sf}}{330} = 1{,}777'$$

only one (1) inlet per 1/4 mile required.

Inlets in a Sag Condition

Initial Storm Grate Inlet

Katie Way:

Given: Allowable curb depth for initial storm = 0.40'
Total D.A. = 1.82 ac

Find: Inlet type and size

1. Total runoff

$$Q = ciA$$
$$= 0.52 \times 6.0 \times 1.82$$
$$Q = 5.7 \text{ cfs}$$

2. Try a double-grate inlet

$Q_i = 3.0 \times 12 \times 0.40^{1.5}/2$; Clogging factor = 2
$= 9.11/2$
$= 4.55$ cfs

Initial Storm Curb-opening Inlet

Try a standard 10′ inlet, W = 2′, a = 3"

$Q_i = (1.7\ L_i + 6.12)(d+.25)^{1.85}/F$
$= [1.7(10)+6.12](.4+.25)^{1.85}/1.25$
$= 8.3$ cfs

Major Storm

1. Provide for the passage of the 100-year flood (Q_{100}) from the sag area

$$Q_{100} = 0.62 \times 9.0 \times 1.82 = 10.2 \text{ cfs}$$

2. Subtract storm sewer design capacity

10.2 - 8.3 (assumed) = 1.9 cfs must go overland

3. Design property line swale

STORM DRAIN DESIGN

Given: Inlet locations as previously determined (assume 180′ for illustration purposes)

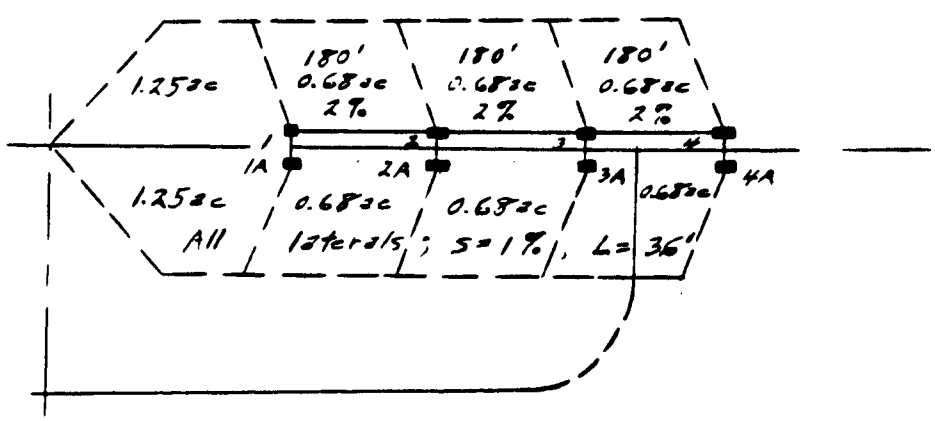
Runoff coefficient = 0.52 (weighted)

n = 0.013

Drainage areas as diagrammed below

15" minimum pipe size
10-year design frequency

Find: Storm sewer sizes for Michaels Road



A - acres
I - inches/hr
Q - cfs

t - min
V - ft/sec

COMPUTATION FORM

DRAINAGE & STORM SEWER DESIGN
RATIONAL METHOD

Project _____
Comp. by _____ date _____
Ck by _____ date _____

LOCATION			A		C	TIME OF CONCENTRATION		I	Q	DESIGN					PROFILE				
STREET	FROM	TO	INCRE-MENT	TOTAL	*	TO INLET	PIPE OR CHANNEL			SIZE	SLOPE %	n	CAP. FULL	V	LENGTH FT	FALL FT	OTHER LOSSES	UPPER INV. EL.	LOWER INV. EL.
Mains	1 A	1	1.25	1.25	.52	6	0	5.7	3.7	15"	1 2	.013	9.1	7.5 **	36	.72	—	1029.00	1028.28
"	1	2	1.25	2.5	.52	6	.1	5.6	7.3	18"	2	.013	14.9	8.5	180	3.6	—	1028.03	1024.43
"	2	3	1.36	3.86	.52	6	.5	5.5	11.0	21"	2	.013	22.1	9.2	180	3.6	—	1024.18	1020.58
"	3	4	1.36	5.22	.52	6	.8	5.4	14.7	21"	2	.013	22.1	9.2	180	3.6	—	1020.68	1016.98
"	4	Outfall	1.36	6.58	.52	6	1.1	5.3	18.1	24"	2	.013	31.8	10	170	3.4	—	1016.73	1013.33
Laterals	2A	2	.68		.52	5	—	6.0	2.1	15"	1	.013	6.5	5.2	36	.36	—	1025.29	1024.93
"	3A	3	.68		.52	5	—	6.0	2.1	15"	1	.013	6.5	5.2	36	.36	—	1022.69	1021.33
"	4A	4	.68		.52	5	—	6.0	2.1	15"	1	.013	6.5	5.2	36	.36	—	1018.09	1017.73

* Cf = 1 for 10 yr Storm
** Pipe size by Manning's Equation n = .013

1 Slope selected to match street grade
2 First inlet set at 3' depth
3 Match crown at pipe size changes

Check Velocity

$d_c/D = 0.93 > 0.78$
$v = 11$ fps < 20 fps (recommended maximum)

From #4 to trunk collector

Inlet time = 6.7 + 0.3 = 7.0 min

$i = 5.2$ in/hr
$A = 6.76$ ac
and $Q = 0.52 \times 52 \times 6.76 = 18.3$ cfs

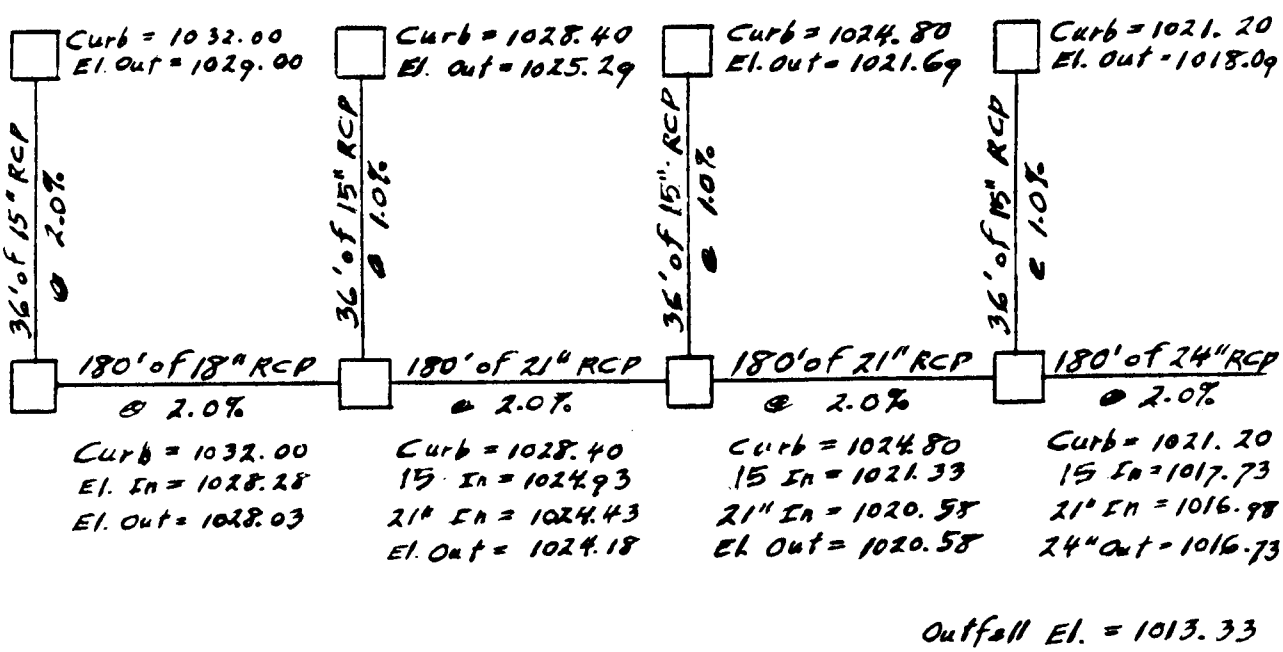
From Figure 8-1, use 24" pipe

($q_{full} = 35$ cfs)
$d/D = 0.68$

Check velocity

$v = 12.0$ fps

Final Design for Storm Sewer
System Along Michaels Road



OPEN CHANNEL DESIGN

Design an open channel to convey a 100-year design discharge through a planned development.

Given: Q_{100} = 170 cfs
Channel Slope = 0.9% or 0.009 ft/ft
Channel Side Slopes = 4H:1V

Find: Dimensions of trapezoidal channel to convey 100-year design discharge

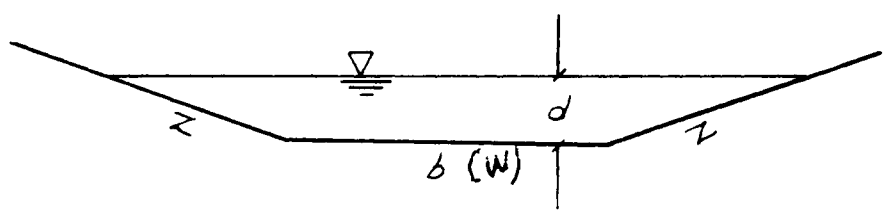
1. S = 0.009 ft/ft
   n = 0.030 from Table 10-2 for straight excavated grass-lined channel
   Qn = 170 (0.030)) = 5.10

2. From Figure 10-2, find bottom width for channel such that $(w/d) \geq 6$

   d/w = 0.17

3. Try b = 10′
   d/w = 0.165
   d = (d/w)(b)
   = 0.165(10) = 1.65′

4. Check with Manning's Formula



$$A = bd + zd^2$$
$$= 10(1/65) + 4(1.65)^2$$
$$= 27.39 \text{ ft}^2$$

$$R = A/(WP)$$

$$= \frac{A}{b + 2 \quad d^2 + (zd)^2}$$

$$= \frac{27.39}{10 + 2 \quad (1.65)^2 + [4(1.65)]^2}$$

$$= 1.16'$$

$$Q = vA = \frac{1.49}{n} R^{2/3} S^{1/2} A$$

$$= \frac{1.49}{0.030} (1.16)^{2/3} (0.009)^{1/2} (27.39)$$

= 142.5 cfs (v = 5.7 fps)<170 cfs
Try again ... d=1.81

5. Minimum Freeboard = $1 + .25\ Vd^{.33} = 1.16$

Use d = 3′
b = 10′

Side Slope = 4H:1V
s = 0.009 ft/ft

DETENTION BASIN DESIGN BY THE RATIONAL STORED RATE METHOD

Determine the required volume to control storm water runoff from an 8-acre site during a 10-year storm event. Assume the site is divided into 4 cells of 2 acres each, and that the slopes are 2%. The inlet time for each cell is estimated at 5 minutes.

Given: Drainage area = 2 acres
"c" runoff coef. = 0.90
tc = 5 minutes
Storm frequency = 10-year

Find:

1. Cell Dimensions

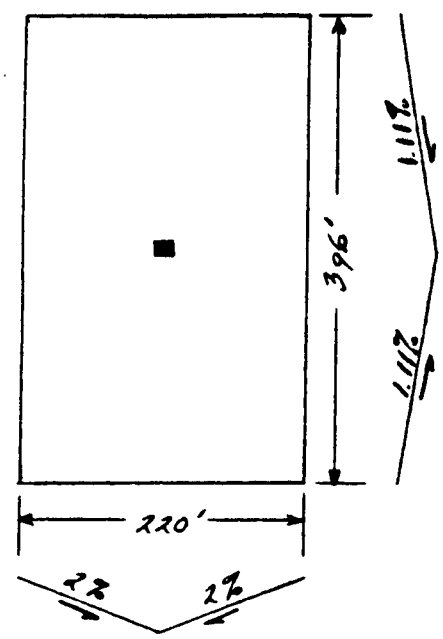


ILLUSTRATION ONLY

Parking Lot Detention Ponds are not allowed as part of the public drainage system.

Cell Volume
= 1/3 (220)(396)(2.2)
= 63,888 cf

9/30/90

2. Family of Curves

D, min.	i, in/hr	Qp, cfs
5	6.00	10.80
10	4.62	8.32
15	3.92	7.06
20	3.45	6.21
30	2.72	4.90
40	2.29	4.12

3. Critical Volume

Assume release rate = 3.5 cfs

D,min.	St. Vol., ft^3	Rel. Vol. ft^3	Storage, ft^3
5	3,240	1,050	2,190
10	4,992	2,100	2,892
15	6,354	3,150	3,204
20	7,452	4,200	3,252*
30	8,820	6,300	2,520
40	9,888	8,400	1,488

4. Check ponding depth

$3{,}252\ ft^3 = 1/3\ Ad$

try $d = 0.50$ ft
$A = 4{,}500\ ft^2$
$V = 750\ ft^3$

try $d = 0.82$ ft
$A = 12{,}115\ ft^2$
$V = 3{,}312\ ft^3$

10"

5. Inlet Design

$Q_i = 5.37\ Ad^{0.5}$ or $Q_i = 3.0\ Pd^{1.5}$

solve for A $\qquad P = \dfrac{3.5}{3.0\ d^{1.5}} = 1.6$ ft

$$A = \frac{3.5}{5.37d^{0.5}} = 0.72\ ft^2$$

Very small grate inlet required - 15" pipe

CULVERT DESIGN

A new culvert will be needed under the arterial street, which borders the proposed shopping center on the south side. The total drainage area above the culvert is 84 acres, and the estimated runoff time of concentration is 15 minutes. The composite runoff coefficient is 0.8.

Downstream of the culvert, the channel has been regraded and aligned with a channel slope of 1%. The outlet elevation will be Elev. 1010. The natural stream slope through the area was 2%. The proposed culvert will be 150 feet long, and the natural stream bed elevation at the inlet is Elev. 1016. The top of road elevation is Elev. 1024.

Design a culvert which will pass the 100-year flood without exceeding the top of road elevation. Determine the culvert size needed assuming inlet control. Also, determine the culvert size needed using an improved inlet. Use the forms and charts presented in Chapter 9. Discharge estimate using the Rational Formula is:

$$Q = 0.8 \times 5.9 \text{ in/hr} \times 84 \text{ acres} = 396 \text{ cfs}$$

SAY 400 cfs

PROJECT : _____

STATION : _____

SHEET ____ OF ____

CULVERT DESIGN FORM

DESIGNER / DATE : ____ / ____

REVIEWER / DATE : ____ / ____

HYDROLOGICAL DATA

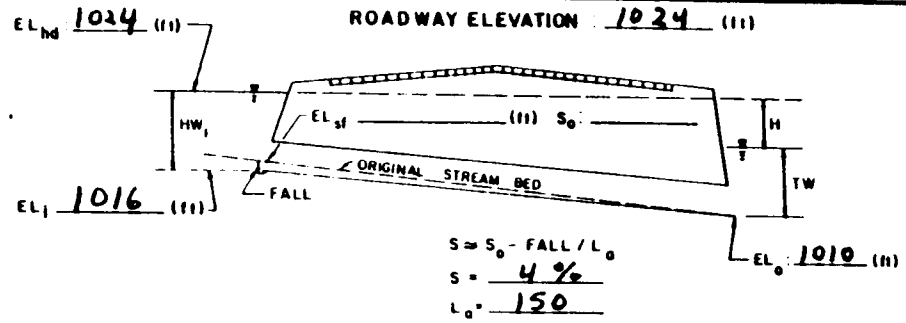
BEE ADD'L SHTS

- ☐ METHOD Rational
- ☐ DRAINAGE AREA: 84 acres ☐ STREAM SLOPE 1%
- ☐ CHANNEL SHAPE: _____
- ☐ ROUTING _____ ☐ OTHER _____

DESIGN FLOWS/TAILWATER

R I (YEARS)	FLOW (cfs)	TW (ft)
100	400	

ROADWAY ELEVATION 1024 (ft)



$S \approx S_o - FALL / L_a$

$S = 4\%$

$L_a = 150$

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW i/D (2)	HW i	FALL (3)	EL hi (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c + D}{2}$	h_o (6)	h_e	H (7)	EL ho (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
Groove Edge Proj. 96"	400	400	0.93	3.44				4.9	6.45	6.45	0.2	1.35		7.4		

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

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

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

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

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

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

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

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

SUBSCRIPT DEFINITIONS :

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

COMMENTS / DISCUSSION :

Inlet Control Governs : Use 96" RCP

CULVERT BARREL SELECTED :

SIZE 96"

SHAPE Circular

MATERIAL Concrete n

ENTRANCE _____

PROJECT: _____

STATION: _____

SHEET ____ OF ____

CULVERT DESIGN FORM

DESIGNER / DATE: ____ / ____

REVIEWER / DATE: ____ / ____

HYDROLOGICAL DATA

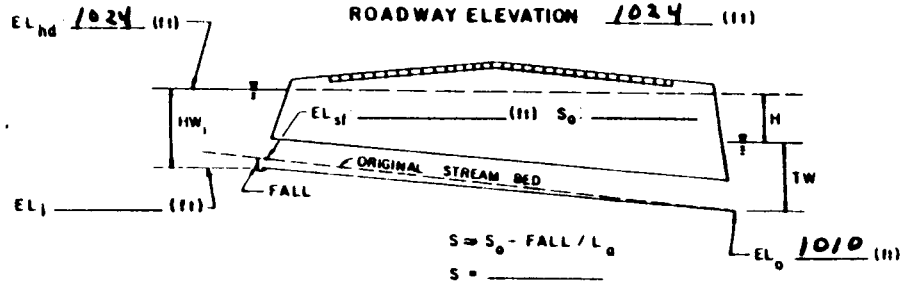
SEE ADD'L SHTS.

- ☐ METHOD: Rational
- ☐ DRAINAGE AREA: 84 acres ☐ STREAM SLOPE: 1%
- ☐ CHANNEL SHAPE: Trapezoidal
- ☐ ROUTING: _____ ☐ OTHER: _____

DESIGN FLOWS/TAILWATER

R I (YEARS)	FLOW (cfs)	TW (ft)
100	400	

ROADWAY ELEVATION 1024 (ft)



$S = S_o - FALL / L_a$

$S =$ _____

$L_a =$ _____

CULVERT DESCRIPTION: MATERIAL · SHAPE · SIZE · ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS — INLET CONTROL: HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL: TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	h_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
Trial-1 60" RCP w/Sq. Edge	400	400					3.8	>5	5	5	0.5	13	1010	1024		HW_o = 1028 Exceeds Allowable
Trial-2 66" RCP w/Bevel Edge	400	400					3.8	5.3	5.4	5.5	0.2	7	1010	1024		HW_o = 1022.4
	350	350					—	5.1	5.3	5.1	0.2	5.3	1010	1024		HW_o = 1020.4
	450	450					—	>5.5	5.5	5.5	0.2	8.8	1010	1024		HW_o = 1024.3

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

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

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

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

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

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

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

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

SUBSCRIPT DEFINITIONS:

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

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE _____

SHAPE _____

MATERIAL _____

ENTRANCE _____

PROJECT : _____

STATION : _____

SHEET _____ OF _____

CULVERT DESIGN FORM

DESIGNER / DATE : _____ / _____

REVIEWER / DATE : _____ / _____

HYDROLOGICAL DATA

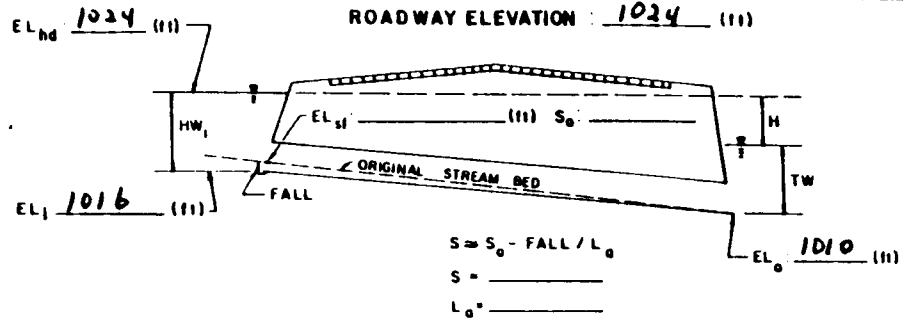
SEE ADD'L SHTS.

- ☐ METHOD Rational
- ☐ DRAINAGE AREA: 84 acres ☐ STREAM SLOPE 1%
- ☐ CHANNEL SHAPE: Trapezoidal
- ☐ ROUTING _____ ☐ OTHER _____

DESIGN FLOWS/TAILWATER

R I (YEARS)	FLOW (cfs)	TW (ft)
100	400	

ROADWAY ELEVATION : 1024 (ft)



$S \approx S_o - FALL / L_o$

$S =$ _____

$L_o =$ _____

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW i/D (2)	HW i	FALL (3)	EL hi (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c + D}{2}$	h_o (6)	h_e	H (7)	EL ho (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
Trial-1 66", Bevel Inlet (Type B)	400	400	2.1	1024	3.6	1012.4									16.8	Fall to large Try Tapered Inlet
Trial-2 66", Tapered Throat	400	400	1.8	1024	1.9										16	OK
	350	350	1.65	1023.2	1.65											
	45	450	2.20	1026.2	2.2											

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

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

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

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

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

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

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

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

SUBSCRIPT DEFINITIONS :

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

COMMENTS / DISCUSSION :

CULVERT BARREL SELECTED :

SIZE 66"

SHAPE Circular

MATERIAL RCP n _____

ENTRANCE Tapered Throat

PROJECT: _____	STATION: _____	**TAPERED INLET DESIGN FORM**
	SHEET ____ OF ____	DESIGNER / DATE: ____ / ____
		REVIEWER / DATE: ____ / ____

DESIGN DATA:

Q 100 - 400 cfs ; EL_{hi} _____ ft

EL. THROAT INVERT 1014.1 ft

EL. STREAM BED AT FACE _____ ft

FALL _____ ft TAPER _____ : 1 (4 : 1 TO 6 : 1)

STREAM SLOPE, S_o = _____ ft/ft

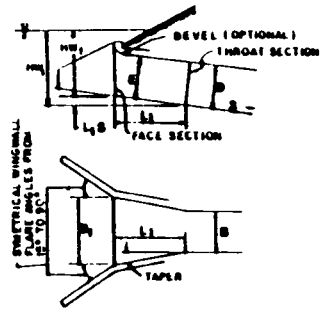
SLOPE OF BARREL, S = _____ ft/ft

S_f _____ : 1 (2 : 1 TO 3 : 1)

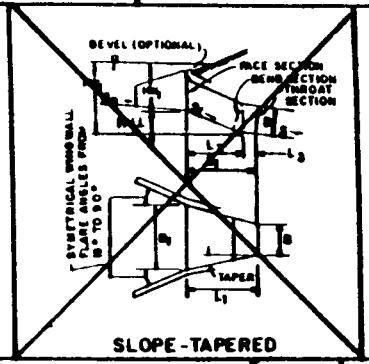
BARREL SHAPE AND MATERIAL: 66" RCP

H = _____, B = _____, D = _____

INLET EDGE DESCRIPTION 45° Bevel



SIDE - TAPERED



SLOPE - TAPERED

COMMENTS

Q (cfs)	EL_{hi}	EL. THROAT INVERT	EL. FACE INVERT (1)	HW_f (2)	HW_f/E (3)	Q/B_f (4)	MIN. B_f (5)	SELECTED B_f	SLOPE-TAPERED ONLY: MIN. L_3 (6)	L_2 (7)	CHECK L_2 (8)	ADJ. L_3 (9)	ADJ TAPER (10)	L_1 (11)	SIDE-TAPERED W/ FALL: EL. CREST INV.	HW_c (12)	MIN. W (13)
400		1014.1	1015.1	1024	1.62	40		10						9			

(1) SIDE - TAPERED: EL. FACE INVERT = EL. THROAT INVERT + 1 ft (APPROX.)
SLOPE - TAPERED: EL. FACE INVERT = EL. STREAM BED AT FACE

(2) $HW_f = EL_{hi} - EL.\ FACE\ INVERT$

(3) $1.1D \geq E \geq D$

(4) FROM DESIGN CHARTS

(5) MIN. $B_f = Q/(Q/B_f)$

(6) MIN. $L_3 = 0.5NB$

(7) $L_2 = (EL.\ FACE\ INVERT - EL.\ THROAT\ INVERT)\ S_f$

(8) CHECK $L_2 = \left[\frac{B_f - NB}{2}\right] \cdot TAPER - L_3$

(9) IF (8)>(7), ADJ. $L_3 = \left[\frac{B_f - NB}{2}\right] \cdot TAPER - L_2$

(10) IF (7)>(8), ADJ. TAPER $= (L_2 + L_3) \Big/ \left[\frac{B_f - NB}{2}\right]$

(11) SIDE - TAPERED: $L = \left[\frac{B_f - NB}{2}\right] \cdot TAPER$
SLOPE - TAPERED: $L_1 = L_2 + L_3$

(12) $HW_c = EL_{hi} - EL.\ CREST\ INVERT$

(13) MIN. $W = 0.35Q/HW_c^{1.5}$

SELECTED DESIGN

B_f 10'

L_1 9'

L_2 _____

L_3 _____

BEVELS ANGLE 45°

b = ____ in; d = ____ in

TAPER 4 : 1

S_f = _____ : 1

The hydraulic design of a storm sewer system should take into account the effect of backwater and allow for all energy losses in the system. It is easy to use only prepared nomographs and charts of flow equations in the design of sewer systems. However, using them alone to design a sewer system may result in erroneous conduit sizing and cause serious problems in the performance of the system.

Flow charts and nomographs provide quick answers for the friction head losses in a given run of straight conduit between structures (manholes, junctions). These design aids do not consider the additional head losses associated with the other structures and appurtenances common in sewer systems.

In most instances, when designing with common friction flow formulas such as the Manning equation, the hydraulic grade is assumed to be equal to the pipe slope at an elevation equal to the crown of the pipe. Consideration must therefore also be given to the changes in hydraulic grade line due to pressure changes, elevation changes, manholes and junctions. The design should then not only be based on the pipe slope, but on the hydraulic grade line.

A comprehensive storm sewer design must therefore proceed on the basis of one run of conduit or channel at a time, working methodically through the system. Only in this way can the free flow conditions be known and the hydraulic grade controlled, thus assuring performance of the system.

Making such an analysis requires backwater calculations for each run of conduit. This is a detailed process which is demonstrated on following pages.

When using the backwater curve approach the design should first establish the type of flow (sub-critical or super critical) in order to determine the direction his calculations are to proceed.

—Super critical flow - designer work downstream with flow
—Sub-critical flow - designer work against the flow
—Hydraulic jump may form if there is super and sub-critical flow in the same sewer

Backwater Analysis

Given is a flow profile of a storm drainage system where the hydraulic grade is set at the crown of the outlet pipe. Hydrological computations have been made and preliminary design for the initial pipe sizing has been completed.

In order to demonstrate the significance of form losses in sewer design, a backwater calculation will be performed in this example with helical corrugated steel pipe.

Solution

1) Draw a plan and surface profile of trunk storm sewer.
2) Design discharges, Q are known; A are known; Diameters of pipe, D have been calculated in preliminary design.
3) Calculate the first section of sewer line.
   Note: Normal depth is greater than critical depth, $d_n > d_c$, therefore calculations to begin at outfall, working upstream.
   At "point of control" set design conditions on profile and calculations sheet:

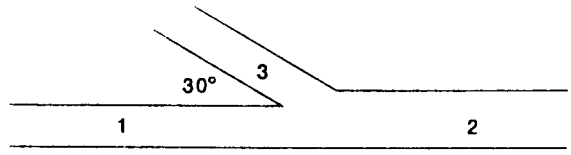
   Station 0 + 00 (outfall)

Design discharge	Q = 245 cfs	(9)
Invert of pipe	= 94.50'	(2)
Diameter	D = 66"	(3)
Hydraulic grade elevation,	H. G. = 100'	(4)
Area of pipe	A = 23.76 sq. ft.	(6)
Velocity = $\frac{Q}{A}$,	V = 6.1 f/s	(8)

Note: (1) Number in brackets refer to the columns on Table

Source: Modern Sewer Design by American Iron and Steel Institute, 1980

Station 4 + 55.5 to 4 + 65.5 (Junction)



$Q = 100$ cfs	$Q_2 = 145$ cfs	$Q_3 = 45$ cfs
$A_1 = 12.57$ ft.^{2}	$A_2 = 15.9$ ft.^{2}	$A_3 = 5.0$ ft.^{2}
$d_1 = 48''$	$d_2 = 54''$	$d_3 = 30''$
$D_1 = 4.0'$	$D_2 = 4'.5'$	$D_3 = 2.5'$
		$\Phi = 30°$

$\Sigma P = \Sigma M$ (Pressure plus momentum laws)

$$\left(H_j + D_1 - D_2\right)\left(\frac{A_1 + A_2}{2}\right) = \frac{Q_2^2}{A_2 g} - \frac{Q_1^2}{A_1 g} - \frac{Q_3^2 \cos \Phi}{A_3 g}$$

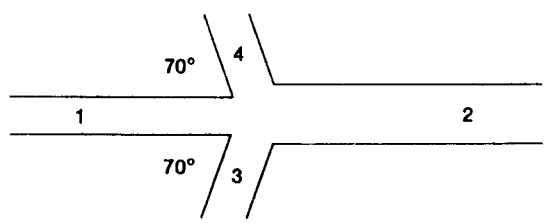
$$(H_j + 4 - 4.5)\ \frac{(12.57 + 15.9)}{2} = \frac{(145)^2}{15.9\,(32.2)} - \frac{(100)^2}{12.57\,(32.2)} - \frac{45^2 \cos 30°}{5\,(32.2)}$$

$$H_j - .5\,(14.24) = 41.07 - 24.71 - 10.89$$

$$14.24\,H_j - 7.12 = 5.47$$

$$H_j = .88$$

Station 5 + 65.5 to 5 + 75.5 (Junction)



$Q_1 = 20$ cfs	$Q_2 = 100$	$Q_3 = 60$	$Q_4 = 20$
$A_1 = 3.14$ ft.^{2}	$A_2 = 12.57$	$A_3 = 7.069$	$A_4 = 3.14$
$d_1 = 24''$	$d_2 = 48''$	$d_3 = 36''$	$d_4 = 24''$
$D_1 = 2'$	$D_2 = 4'$	$\theta_3 = 70°$	$\theta_4 = 70°$

$$(H_j + D_1 - D_2)\left(\frac{A_1 + A_2}{2}\right) = \frac{Q_2^2}{A_2 g} - \frac{Q_1^2}{A_1 g} - \frac{Q_3^2 \cos \theta_3}{A_3 g} - \frac{Q_4^2 \cos \theta_4}{A_4 g}$$

$$(H_j + 2 - 4)\ \frac{(3.14 + 12.57)}{2} = \frac{(100)^2}{(12.57)\,(32.2)} - \frac{(20)^2}{(3.14)\,(32.2)} - \frac{(60)^2 \cos 70°}{7.06\,(32.2)} - \frac{(20)^2 \cos 70°}{3.14\,(32.2)}$$

$$(H_j - 2)\,(7.86) = 24.71 - 3.96 - 5.41 - 1.35$$

$$7.86 H_j - 15.72 = 13.99$$

$$H_j = 3.78$$

Compute:

a) 'K' value (7): $K = \frac{(2g)\,n^2}{2.21}$ (Derived from Manning-Chezy Formula)

b) 'S_f' value (12): $S_f = K \frac{V^2}{2g} \div R^{4/3}$

The friction slope (S_f) may also be estimated from Table for a given diameter of pipe and with a known n value for the expected flow Q.

S_f (12) is a "point slope" at each station set forth by the designer. Therefore, the friction slope (Ave. S_f) (13) for each reach of pipe L, (14), is the average of the two point slopes S_f being considered.

c) Velocity Head (10) $H_v = \frac{V^2}{2g}$

d) Energy grade point, E. G. (11) is equal to H. G. (4) plus the velocity head (10).

e) Friction loss (15): Multiply Ave. S_f (13) by length of sewer section, L (14) = H_f (15).

f) Calculate energy losses: H_b, H_j, H_m, H_t, using formulae in text.

g) Compute new H. G. (4) by adding all energy loss columns, (15) thru (19) to previous H. G.

Note: If sewer system is designed under pressure (surcharging) then energy losses must be added (or subtracted, depending on whether you are working upstream or downstream) to the energy grade line, E. G.

h) Set new E. G. (20) equal to E. G. (11)

i) Conduit invert (2), in example designing for full flow conditions, therefore H. G. (4) is at crown of pipe and invert (2) is set by subtracting, D (3) from H. G. (4).

j) Continue to follow the above procedure taking into account all form head losses.

k) Complete profile drawing, showing line, grade and pipe sizes. This saves time and usually helps in spotting any design errors.

Energy Losses

Station 1 + 10 to 1 + 52.4 (Bend)

$$H_b = K_b \left[\frac{V^2}{2g}\right], \text{ where } K_b = 0.25 \sqrt{\frac{\Phi}{90}}$$

Φ, central angle of bend = 30°

$$K_b = 0.25 \sqrt{\frac{30}{90}} = 0.1443$$

$$\therefore H_b = 0.1433\,[0.58] = 0.08'$$

Station 2 + 48 to 2 + 55.5 (Transition)

$$H_t = 0.2 \left[\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right] \text{ Expansion}, V_1 > V_2$$

$$= 0.2\,[1.29 - 0.58]$$

$$= 0.14$$

Station 3 + 55.5 (Manhole)

$$H_m = 0.05 \left[\frac{V^2}{2g}\right]$$

$$= 0.05\,[1.29] = 0.06'$$

Station 6 + 75.5 (Manhole)

$$H_m = .05 \left(\frac{V^2}{2g} \right)$$

$$= 0.03$$

Total friction H_f throughout the system = 7.38

Total form losses = 4.97

In this example, the head losses at junctions and transition could also have been accommodated by either increasing the pipe diameter or increasing the slope of the pipe.

This backwater example was designed under full flow conditions but could also have been designed under pressure; allowing surcharging in the manholes; which would have reduced the pipe sizes. Storm sewer systems, in many cases, can be designed under pressure to surcharge to a tolerable level.

Table Energy-Loss Solution by Manning's Formula for Pipe Flowing Full

Diam. in Inches	Area in Square Feet A	Hydraulic Radius R	$R^{2/3}$	$AR^{2/3}$	$\left(\frac{n}{1.486AR^{2/3}}\right)^2 \times 10^{-7}$				
					$n=0.012$	$n=0.015$	$n=0.019$	$n=0.021$	$n=0.024$
6	.196	.125	.250	.049	271,600.	424,420.	681,000.	831,940.	1,086,350.
8	.349	.167	.303	.106	58,000.	90,703.	145,509.	177,730.	232,164.
10	.545	.208	.351	.191	17,879.	27,936.	44,802.	54,707.	71,455.
12	.785	.250	.397	.312	6,698.	10,466.	17,797.	20,605.	26,791.
15	1.227	.3125	.461	.566	2,035.6	3,180.8	5,102.5	6,234.4	8,144.6
18	1.767	.375	.520	.919	772.2	1,206.5	1,935.5	2,364.7	3,088.7
21	2.405	.437	.576	1.385	340.00	531.24	852.60	1,041.0	1,359.98
24	3.142	.50	.630	1.979	166.5	260.04	417.31	510.20	666.39
30	4.909	.625	.731	3.588	50.7	79.126	127.01	155.12	202.54
36	7.069	.75	.825	5.832	19.20	29.953	48.071	58.713	76.691
42	9.621	.875	.915	8.803	8.40	13.148	21.096	25.773	33.667
48	12.566	1.00	1.00	12.566	4.130	6.452	10.353	12.647	16.521
54	15.904	1.125	1.082	17.208	2.202	3.440	5.520	6.741	8.817
60	19.635	1.25	1.16	22.777	1.257	1.965	3.337	3.848	5.030
66	23.758	1.375	1.236	29.365	0.756	1.182	1.895	2.316	3.026
72	28.274	1.50	1.310	37.039	0.475	0.743	1.192	1.456	1.902
78	33.183	1.625	1.382	45.859	0.310	0.485	0.777	0.950	1.241
84	38.485	1.75	1.452	55.880	0.209	0.326	0.524	0.640	0.835
90	44.179	1.875	1.521	67.196	0.144	0.226	0.362	0.442	0.578
96	50.266	2.00	1.587	79.772	0.102	0.160	0.257	0.314	0.410
108	63.617	2.25	1.717	109.230	0.055	0.085	0.137	0.167	0.219
114	70.882	2.375	1.780	126.170	0.041	0.064	0.103	0.125	0.164
120	78.54	2.5	1.842	144.671	0.031	0.049	0.078	0.098	0.125

Manning Flow Equation: $Q=\left(A \times \frac{1.486}{n} \times R^{2/3}\right) \times S^{1/2}$

Energy Loss $= S = Q^2\left(\frac{n}{1.486\, AR^{2/3}}\right)^2$

To find energy loss in pipe friction for a given Q, multiply Q^2 by figure shown under proper value of n.

Table Hydraulic Calculation Sheet

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Sta.	Inv.	D	W. S.	Section	A	K	V	Q	$\frac{V^2}{2g}$	E. G.	S_f	Ave. S_f	L	H_f	H_b	H_i	H_m	H_t	E G
0 + 00	94.50	66	100	O	23.76	0.01678	6.1	145	0.58	100.58	0.0064	0.0064	110	0.70					100.58
1 + 10	95.20	66	100.70		23.76	0.01678	6.1	145	0.58	101.28	0.0064	0.0064	42.4	0.27	0.08				101.28
1 + 52.4	95.55	66	101.05		23.76	0.01678	6.1	145	0.58	101.63	0.0064	0.0064	95.6	0.61					101.63
2 + 48	96.16	66	101.66		23.76	0.01678	6.1	145	0.58	102.24	0.0064	0.0110	7.5	0.08				0.14	102.24
2 + 55.5	96.67	54	101.17		15.90	0.01410	9.1	145	1.29	102.46	0.0155	0.0155	100	1.55			0.06		102.46
3 + 55.5	98.28	54	102.78		15.90	0.01410	9.1	145	1.29	104.07	0.0155	0.0155	100	1.55					104.07
4 + 55.5	99.83	54	104.33		15.90	0.01410	9.1	145	1.29	105.62	0.0155	0.0150	10	0.15		0.88			105.62
4 + 65.5	101.66	48	105.66		12.57	0.01166	8.0	100	0.99	106.65	0.0115	0.0115	100	1.15					106.65
5 + 65.5	102.81	48	106.81		12.57	0.01166	8.0	100	0.99	107.80	0.0115	0.0118	10	0.12		3.78			107.80
5 + 75.5	109.06	24	111.06		3.14	0.00746	6.4	20	0.64	111.70	0.0120	0.0120	100	1.20			0.03		111.70
6 + 75.5	110.29	24	112.29	O	3.14	0.00746	6.4	20	0.64	112.93	0.0120								112.93

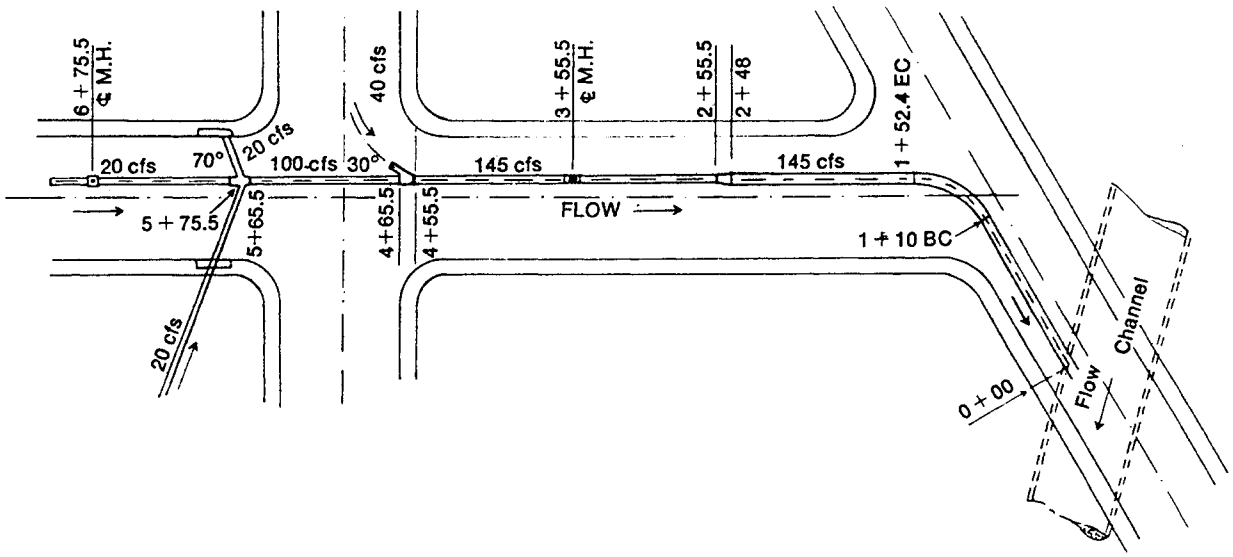
$\Sigma h_{friction} = 7.38$

$\Sigma h_{form} = 4.97$

n = Variable

$K = \frac{2g\,(n^2)}{2.21}$

$S_f = K\left(\frac{V^2}{2g}\right) + r^{4/3}$



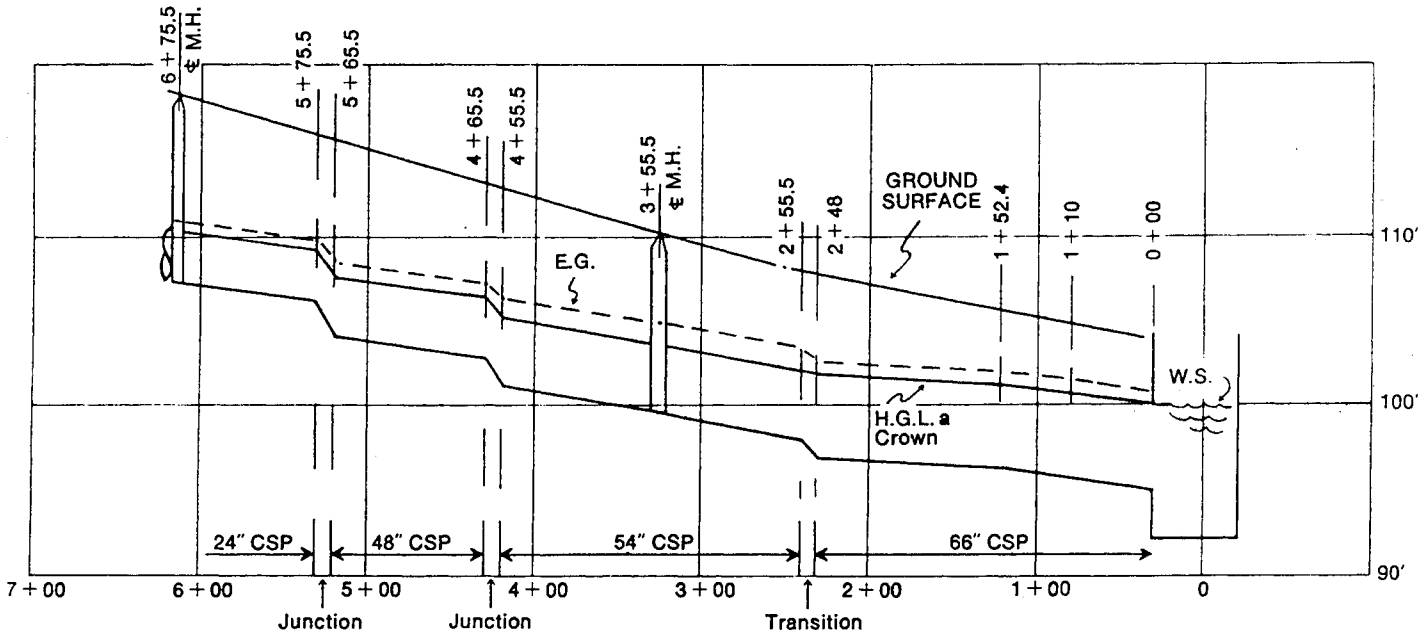


Figure Plan and profile for storm sewer.

GLOSSARY

ENGINEERING AND RELATED TERMS

Alternate Depths: For a given rate of flow and a given specific head two depths of flow are possible, one lower than critical and one higher than critical. These are known as alternate depths. See also Critical Depth.

Backwater: The water retarded above a dam or backed up into a tributary by a flood in the main stream.

Backwater Curve: The term applied to the longitudinal profile of the water surface in an open channel when flow is steady but non-uniform.

Baffle Chute: A drop structure in a channel with baffles for energy dissipation to permit the lowering of the hydraulic energy gradient in a short distance to accommodate topography.

Berm: A horizontal strip of shelf built into an embankment or cut, to break the continuity of an otherwise long slope, usually for the purpose of reducing erosion, improving stability, or to increase the thickness or width of cross section of an embankment.

Carry Over: The quantity of water which continues past an inlet.

Catch Basin: A basin combined with a storm drain inlet to trap solids.

Catchment Area: See Drainage Area.

CFS: Cubic feet per second (also known as second-foot) is a measure of discharge or flow. The basic units are volume per time. A cfs is equal to 7.48 gallons per second.

Check: A barrier placed in a ditch, canal or channel to decrease the velocity of the flow of water so as to minimize erosion of the bottom and banks or to raise the level of the water. Also used for diverting water from one channel to another, as in irrigation usage.

Chute: An inclined conduit or structure used for conveying water at a high velocity to lower levels. For vertical structures see Drop.

Coefficient of Roughness: A factor, in the Kutter, Manning, Bazin, or other formula for computing the average velocity of flow of water in a watercourse or conduit, which represents the effect of roughness of the confining material of the watercourse or conduit upon the energy losses in the flowing water.

Conduit: Refers to an underground storm drain, which is a pipe designed to carry storm water runoff. The runoff enters the pipe through Catch Basins, which are those inlets usually found in the gutters, identifiable by the curb opening or iron grating that covers them. The conduit is nearly always made from concrete.

Conduit with Open Channel or Surface Flow: Describes a combination of water flow devices. Conduit with open channel is a system of moving storm water runoff that consists of both conduits and open channels. (See Open Channels)

Critical Depth: The particular depth of flow in an open channel with a given discharge at which the specific energy is at a minimum; i.e., the depth at which a given discharge flows in a given channel with a minimum specific energy. The given discharge may flow at an alternate depth above or below critical in the given channel but the specific energy of the flow at either alternate depth will be greater than for flow at critical depth.

Critical Flow: Flow at critical depth.

Cross-Street Flow: Flow across the traffic lanes of a street from external sources, as distinguished from sheet flow of water falling on the pavement surface.

Culvert: A closed conduit for the passage of surface drainage water under or over a roadway, railroad, canal, or other impediment.

Culvert Box: Generally a rectangular or square section concrete structure for carrying large amounts of water under a roadway. This term is sometimes applied to long underground conduits.

DBPS-Drainage Basin Planning Study: The initial drainage plan for a designated major basin. The plan shall develop the hydrology for 100 acre sub-basin and hydraulics for the major streams in the basin. The plan shall establish the unit drainage fee for each basin.

Debris Basin: A basin formed behind a low dam, or an excavation in a stream channel, to trap debris or bed load carried by a stream. The value of a basin depends on cleaning-out of debris periodically to restore its capacity.

Detention Facility: Is designed for the storage of storm runoff water for controlled release during or immediately following storm. A typical facility consists of a detention pond area with an embankment (or dam) on the downstream side, and a pipe or concrete box outlet (see culvert). The size of the pond is based on a specific design storm and the amount of water that can be discharged through the outlet.

Drainage Area: (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 Way: A route or course along which water moves or may move to drain an area.

Drains: Usually a pipe, ditch or channel for collecting and conveying water. Sometimes used by lawyers in "Storm Sewers and Drains" when describing an urban storm drainage system to carry the initial runoff.

Drawdown Curve: The longitudinal profile at the water surface of an open channel as it approaches and crosses an overfall.

Drop: A vertical structure in a conduit or canal installed for the purpose of dropping water to a lower level.

Drop Inlet Culvert: A culvert installed with a drop inlet on one end and daylighted at the other end.

Energy Gradeline: The total energy level of water at all points along a longitudinal line. It is the sum of velocity head, pressure head and elevation of a flowing body of water.

Erosion: Wearing away of the lands by running water and waves, abrasion and transportation.

FDR-Final Drainage Report: A final drainage plan submitted which is required after approval of the Preliminary Drainage Report. The FDR shall reflect revisions directed by the City/County on the Planning Drainage Report (PDR).

Flood: Water from a river, stream, watercourse, ocean, lake, or other body of standing water that temporarily overflows or inundates adjacent lands and which may affect other lands and activities through stage elevation, backwater and/or increased ground water level.

Flood-Design: The amount of runoff generated from the 10-year or 100-year design storm for which the major project features are sized.

Flash Flood: A flood of short duration with a relatively high peak rate of flow, usually resulting from a high intensity rainfall over a small area.

Flood-Maximum Probable: The largest momentary flood discharge believed possible from a consideration of meteorologic conditions and snow cover on the watershed.

Flood Control: The elimination or reduction of flood losses by the construction of flood storage reservoirs, channel improvements, dikes and levees, by-pass channels, or other engineering works. Sometimes called the structural alternate.

Flood Frequency: The average interval of time, based on the period of record, between floods equal to or greater than a specified discharge or height. It is generally expressed in years. See Flood Probability.

Flood Plain: The relatively flat or lowland area adjoining a river, stream, watercourse, ocean, lake, or other body of standing water which has been or may be covered temporarily by flood water. For administrative purposes, the flood plain may be defined as the area that would be inundated by the "Standard Project Flood" (CofE) or the "Maximum Probable Flood (TVA).

Flood Plain Fringe: That portion of the flood plain that lies outside the regulatory area. Its hazard should be recognized although it is not great enough to make public regulations desirable.

Flood Plain Management: Control of use of land subject to flooding.

Flood Plain Regulations: A general term applied to the full range of codes, ordinances, and other regulations relating to the use of land and construction as influenced by water. The term encompasses zoning ordinances, subdivision regulations, building and housing codes, encroachment line statutes, open-area regulations, and other similar methods of control affecting the use and development of the areas.

Flood Probability: The probability of a flood of given size being equaled or exceeded in a given period. A probability of one percent would be the flood expected to be equaled or exceeded once in 100 years; a probability of ten percent would be a 10-year flood.

Flood Proofing: A combination of structural changes and adjustments to properties subject to flooding primarily for the reduction of flood damages.

Floodway: Floodway is that portion of the regulatory area required for the reasonable passage or conveyance of the design flood. This is the area of significant depths and velocities and due consideration should be given to effects of fill, loss of cross sectional flow area, and resulting increased water surface elevations.

Freeboard: The vertical distance between the normal maximum level of the surface of the liquid in a conduit, reservoir, tank, basin, canal, etc., and the top of the confining structure, which is provided so that waves and other movements of the liquid will not overtop such confining structures.

Froude Number: A flow parameter, which is a measure of the extent to which gravitational action affects the flow. A Froude number greater than 1 indicates supercritical flow and a value less than 1 subcritical flow. The simplest form of the Froude number is given by the equation.

$$F = \frac{V}{\sqrt{gD}}$$ where V is velocity

g the acceleration due to gravity (32.2 ft/sec/sec), and
D the depth

Gabion: A wire basket containing earth or stones, deposited with others to provide protection against erosion.

Gradient: The rate of change of any characteristic per unit of length. The term is usually applied to such things as elevation, velocity, pressure, etc.

Gutter Flow: Flow in a gutter.

Headgate: The gate at the entrance to an irrigation ditch, or other channel.

Headwater: (1) The upper reaches of a stream near its source. (2) The region where ground waters emerge to form a surface stream. (3) The water upstream from a structure.

Hydraulics: A branch of science that deals with practical applications of the mechanics of water movement.

Hydraulic Gradeline: A hydraulic profile of the piezometric level of the water, representing the sum of the depth of flow and the pressure. In open channel flow it is the water surface.

Hydraulic Jump: The hydraulic jump is an abrupt rise in the water surface which occurs in an open channel when water flowing at supercritical velocity is retarded by water flowing at subcritical velocity. The transition through the jump results in a marked loss of energy, evidenced by turbulence of the flow within the area of the jump. The hydraulic jump is often used as a means of energy dissipation.

Hydro Planing: The phenomenon of a vehicle tire being supported by a thin film of water between it and the pavement.

Hydrograph: A graph showing stage, flow, velocity or other property of water versus time at a given point on a stream or conduit.

Unit Hydrograph: See definition under U.

Hydrology: The science that deals with the processes governing the depletion and replenishment of the water resources of the land areas of the earth.

Impervious: A term applied to a material through which water cannot pass, or through which water passes with great difficulty.

Infiltration: (1) The entering of water through the interstices or pores of a soil or other porous medium. (2) The quantity of groundwater which leaks into a sanitary or combined sewer or drain through defective joints. (3) The entrance of water from a ground into a sewer or drain through breaks, defective joints, or porous walls. (4) The absorption of liquid water by the soil, either as it falls as precipitation, or from a stream flowing over the surface. See Surface Infiltration.

Infiltration of Rainfall: See Surface Infiltration.

Initial Drainage System: That storm drainage system is frequently used for collecting, transporting, and disposing of snowmelt, miscellaneous minor flows, and storm runoff up to the capacity of the system. The capacity should be equal to the maximum rate of runoff to be expected from the initial design storm which has a frequency of occurrence of once in 10 years.

The initial system may include many features ranging from curbs and gutters to storm drain pipes and open drainageways.

Inlet: (1) An opening into a storm drain system for the entrance of surface storm runoff, more completely described as a storm drain inlet. (2) A structure at the diversion end of a conduit. (3) The upstream connection between the surface of the ground and a drain, for the admission of surface or storm water.

Inlet Backwater: The surface water accumulated over and adjacent to an inlet that cannot enter the inlet for any one of various reasons.

Inlet Gratings:

a. Longitudinal Bar Grate: A grate in which the bars are oriented parallel to the direction of flow.

b. Transverse Bar Grate: A grate in which the bars are located at some angle, usually perpendicular to the direction of flow.

Inlet Types:

a. Combination Inlet: An inlet composed of a curb opening and a grated gutter opening inlet acting as a unit. Usually the gutter opening is placed directly in front of the curb opening. This arrangement is called a contiguous combination inlet, or more simply a combination inlet. When the curb and gutter opening are placed in an overlapping, or end to end position, the arrangement is called an overlapping, offset, or special combination inlet.

b. Curb Opening Inlet: A vertical opening in a curb through which the gutter flow passes. The gutter may be undepressed or depressed in the area of the curb opening.

c. Deflector Inlet: A special type of curb opening inlet which has flow deflector in the gutter adjacent to the opening.

d. Grated Inlet: An opening in the gutter covered by one or more grates through which the water falls. As with all inlets, grated inlets may be either depressed or undepressed and may be located on a continuous grade or in a sump.

e. Multiple Inlet: Two or more closely spaced inlets acting as a unit. The two inlets may be of any of the types mentioned above.

Intensity: As applied to rainfall is a rate usually expressed in inches per hour.

Intercepting Channel: A channel excavated at the top of earth cuts, at the foot of slopes, or at other critical places to intercept surface runoff or flow.

Interception: As applied to hydrology, refers to the process by which precipitation is caught and held by foliage, twigs and branches of trees, shrubs and buildings, never reaching the surface of the ground, and is lost by evaporation.

Interception Rate: As applied to storm water inlet design, interception rate refers to the ratio of flow intercepted by an inlet to the gutter flow delivered to an inlet.

Invert: The floor, bottom or lowest portion of the internal cross section of a conduit. Used particularly with reference to aqueducts, sewers, tunnels, and drains.

Inverted Sipon: Refers to conduit flowing freely up and down stream but which is pressurized in a depressed section to carry water underneath an obstacle.

Lag Time: The time difference between two occurrences, such as between rainfall and runoff or pumping of a well and effect on the stream. See Time of Concentration.

Lining: Impervious material such as concrete, stone, clay, grass, plastic, puddled earth, etc., placed on the sides and bottom of a ditch, channel, and reservoir to prevent or reduce seepage of water through the sides and bottom and/or to prevent erosion.

Major Drainage System: 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, gulches, and major drainage channels. Storm drain systems may reduce the flow in many parts of the major system by storing and transporting water underground.

The good planning and designing of a major system should eliminate major damage and loss of life from storms having a one percent chance of occurring in any given year.

Manhole: A hole through which a man may go, especially to gain access to an underground or enclosed structure.

MDDP-Master Development Drainage Plan: A drainage masterplan required for any phased residential,commercial or industrial development greater than 10 acres.

Orifice: (1) An opening with closed perimeter, and of regular form in a plate, wall, or partition, through which water may flow. (2) The end of a small tube, such as a Pitot tube, piezometer, etc.

Outfall: The point or location where storm runoff discharges from a drain. Also applies to the outfall storm drain or channel which carries the storm runoff to the point of outfall.

PDR-Planning Drainage Report: A preliminary drainage report submittal required for all residential commercial or industrial development applying for land annexation or preliminary platting process.

Peak Rate of Runoff: The maximum rate of runoff during a given runoff event.

Percolation: To pass through a permeable substance such as ground water flowing through an aquifer.

Permeability: The property of a material which permits movement of water through it when saturated and actuated by hydrostatic pressure.

Pervious: Applied to a material through which water passes relatively freely.

Pipes, Tile Drain: Pipes which are laid in covered trenches underground with either open joints or with perforations to collect and carry off excess ground water.

Precipitation: Any moisture that falls from the atmosphere, including snow, sleet, rain and hail.

Rainfall, Effective: As applied to runoff analysis, refers to the portion of rainfall which becomes surface runoff or runoff excess.

Rainfall Intensity Curve: A curve which expresses the relation between rate of rainfall and duration. Each curve is generally for a period of years during which time the intensities shown will not, on the average, be exceeded more than once.

Rational Formula: A formula for estimating the peak rate of runoff from a given drainage basin less than 100 acres in contributory drainage area. The formula is, in its basic form,

$$Q = CIA$$

where Q is the peak discharge in cubic feet per second,
C is the runoff coefficient
I is the intensity of rainfall expressed in inches per hour for a period equal to the time of concentration of the basin.
A is the drainage area in acres.

Reach: Any length of river or channel. Usually used to refer to sections which are uniform with respect to discharge, depth, area or slope, or sections between gaging stations.

Recurrence Interval: The average interval of time within which a given event will be equalled or exceeded once. For an annual series (as apposed to a partial duration series) the probability of occurrence in any one year is the inverse of the recurrence interval. Thus a flood having a recurrence interval of 100 years has a 1 percent probability of being equalled or exceeded in any one year.

Return Period: See Recurrence Interval.

Reynold's Number: A flow parameter which is a measure of the viscous effects on the flow. Typically defined as:

$$Re = \frac{VD}{v}$$ where V is velocity, D is depth and v is the kinematic viscosity of the fluid.

Riprap: Broken stone or boulders placed compactly or irregularly on dams, levees, ditches, dikes, etc., for protection of earth surfaces against the erosion action of water.

Routing Hydraulic: (1) The derivation of an outflow hydrograph of a channel or stream from known values of upstream inflow. (2) The process of determining progressively the timing and shape of a flood wave at successive points along a stream or channel.

Runoff: That part of the precipitation which reaches a steam, drain, etc., directly or indirectly.

9/30/90

a. Direct Runoff: The total amount of surface runoff and storm runoff which reaches stream channels.

b. Overland Runoff: Water flowing over the land surface before it reaches a definite stream channel or body of water.

Runoff Coefficient: A decimal number used in the Rational Formula which defines the runoff characteristics of the drainage area under consideration. It may be applied to an entire drainage basin as a composite representation or it may be applied to a small individual area such as one residential lot.

Runoff Total: The total volume of flow from a drainage area for a definite period of time such as a day, month, or a year, or it may be for the duration of a particular storm.

Scour: The erosive action of running water in streams or channels in excavating and carrying away material from the bed and banks.

SCS Hydrograph Procedure: The hydrologic analysis method recommended for basins greater than 100 acres for development of stormwater runoff hydrographs and peak discharges for the 10-year and 100-year storm event.

Sediment: Material of soil and rock origin transported, carried, or deposited by water.

Silt Basin: A basin or reservoir installed in a storm drainage system to retard velocity, causing sedimentation and providing storage for deposited solids.

Siphon: A closed conduit, a portion of which lies above the hydraulic grade line. This results in a pressure less than atmospheric in that portion, and hence requires that a vacuum be created to start flow.

Slope, Critical: The slope or grade of a channel that is exactly equal to the loss of head per foot resulting from flow at a depth that will give uniform flow at critical depth; the minimum slope of conduit which will produce critical flow.

Slope, Friction: The friction head or loss per unit length of channel or conduit. For uniform flow the friction slope coincides with the energy gradients, but where a distinction is made between energy losses due to bends, expansions, impacts, etc., a distinction must also be made between the friction slopes and the energy gradient. The friction slope is equal to the bed or surface slope only for uniform flow in uniform open channels.

Spillway: A waterway in or about a dam or other hydraulic structure, for the escape of excess water. Also referred to as By-Channel, By-Wash, and Diversion Cut.

9/30/90

Spillway, Automatic: Devices for wasting water into drainage courses. They may be uncontrolled overflow crests, crests equipped with automatic gates or enclosed crests designed for siphon operation.

Spillway, Side Channel: A spillway in which the initial and final flow are approximately at right angles to each other.

Statistical Analysis: A method of analyzing a series of past events using mathematical and probability techniques to predict future occurrences.

Storm Drain: A closed conduit for conducting storm water that has been collected by inlets or collected by other means (sometimes referred to as Storm Sewer). The various parts of a drainage system are defined as follows:

a. Lateral (Collection) Storm Drain: A storm drain that has inlets connected to it but has no other storm drain connected.

b. Trunk (Main) Storm Drain: A storm drain which receives the discharge from several branches (submains) and generally serves a relatively large area, and may or may not have inlet connections.

c. Outfall Storm Drain: A storm drain which receives the runoff from a collecting system, such system being lateral (collection) storm drains, branch (submain) drains, and trunk (main) storm drains, as are required, and carries such runoff to a point of final drainage.

Storm Drainage System: All facilities used for conducting the storm water through and from a drainage area to a point of final outlet, consisting of any or all of the following: conduits and appurtenant features, canals, channels, ditches, streams, gulches, gullies, flumes, culverts, inlets, streets, and pumping stations.

Storm, Initial Design: A storm of such magnitude that it can be expected to occur only once each 10 years.

Storm, Major Design: A storm of such magnitude that it can be expected to occur once every 100 years.

Storm Runoff: The water from precipitation running off from the surface of a drainage area during and immediately following a period of rain.

Street Classifications: (As used for drainage criteria)

a. Local Residential Street: A local residential street is a minor traffic carrier within a neighborhood which usually is characterized by two moving lanes and

parking along the curb. There is no through traffic moving from one neighborhood to another. Traffic control is by use of stop or yield signs, and many cases, there is no formal traffic control.

b. Collector and Minor Arterial Streets: The function of a collector street is to collect and distribute traffic between arterial and local streets. There may be two or four moving traffic lanes and parking is allowed adjacent to curbs. Traffic on collectors has right-of-way over traffic from adjacent local streets.

c. Arterial: An arterial street permits rapid and relatively unimpeded traffic movement throughout a city. Four to six lanes of traffic exist and parking adjacent to curbs may be prohibited. The arterial traffic normally has the right-of-way over collector streets which it crosses with intersections at approximate 1/4-mile intervals. Construction of an arterial will often include a median strip with traffic channelization and signals at numerous intersections.

d. Freeway: A freeway permits rapid and unimpeded movement of traffic through and around a city. Access to the freeway is completely controlled by interchanges at major arterial streets. There may be up to eight lanes of traffic and parking is not permitted on the freeway right-of-way.

Street Flow: The total flow of storm runoff in a street, usually being the sum of gutter flows on each side of the street. Also the total roadside ditch flow where there are no curbs and gutters.

Street Nomenclature:

a. Crown: In the pavement, it is the highest point in the paving cross-section.

b. Grade: The longitudinal slope measured along the crown.

c. Cross Slope: The slope of the pavement perpendicular to the crown.

d. Curb: The lateral side of the pavement terminated by either a vertical or a sloped concrete section.

e. Curb and Gutter Section: A curb section constructed integrally with the gutter.

f. Cross Fall: In lateral pavement cross-section, it is the difference in elevation between the gutter flow lines.

g. Cross Pan: A concaved paved surface crossing a street, usually at pavement intersections, for the purpose of carrying surface water across the street to continue the surface flow.

h. Gutter: A paved section designed to carry surface flow. Often the gutter is constructed integral with curb when located at the edge of a street section.

Subdrain: An underground conduit designed to permit infiltration for the purpose of collecting ground water.

Subgrade: (1) The bottom of a trench, or other excavation or structure which is to be placed therein, the intervening space being backfilled with some special material such as sand, gravel, broken stone, or tamped earth, or impervious lining, or occupied by the structure for which the excavation was made. The term is also applied to the elevation of such bottom. (2) The natural soil area beneath a street or road.

Subsoil: That portion of a normal soil profile underlying the surface or A-horizon. Its depth and physical properties control to a considerable degree the movement of soil moisture.

Sump Condition: Water restricted to an inlet area because the inlet is located at a low point.

Surface Detention: The storm runoff detained on the surface of the ground at or near where the rainfall occurred and which will run off later.

Surface Flow or Sheet Flow: The surface flow from rainfall on pavements, ground surfaces and other exposed surfaces until such flow reaches a gutter, ditch, water course, inlet, or other point of concentration.

Thalweg: (1) the lowest thread along the axial part of a valley. (2) The middle of chief navigable channel of a waterway.

Time of Concentration: The time required for storm runoff to flow from the most remote point of a catchment or drainage area to the outlet or point under consideration. It is not a constant, but varies with depth of flow, grades, and condition of conduit and/or channel.

Time of Flow: The time required for water to flow in a storm drain from the point where it enters to any given point or location beyond the inlet.

Time of Inlet: The time required for storm water to flow from the most distant point on a drainage area to the point where it enters a storm inlet drain. The time of concentration for the inlet.

Trash Rack: A barrier constructed to catch debris and exclude it from a downstream conduit. An improperly constructed or maintained trash rack may render a conduit useless.

Underdrain: A system of tile drains which are laid in covered trenches underground, in most cases with open joints, to collect and carry off excess ground water.

Uniform Flow: Uniform flow occurs only in a uniform channel when resistance and gravity forces are in exact balance. For uniform flow the energy gradient, water surface, and channel bottom are all parallel.

Unit Drainage Fee: A one time charge to the developer which represents the unit cost (per acre of remaining plattable land in a designated basin) of drainage improvements yet to be constructed in the basin.

Unit Hydrograph: A unit hydrograph is defined as the hydrograph of one-inch of direct runoff from the tributary area resulting from a unit storm. A unit storm is a rainfall of such duration that the period of surface runoff is not appreciably less for any rain of shorter duration. The unit hydrograph thus represents the integrated effects of factors such as tributary area, shape, stream pattern, channel capacities, and stream and land slopes.

Velocity: A time rate of change of position.

a. Critical: The velocity at which turbulent eddying motion in flow begins or ceases.

b. Mean: The average velocity of a stream flowing in a channel or conduit at a given cross section or in a given reach. It is equal to the discharge divided by the cross-sectional area of the section of the average cross-sectional area of the reach. Also called Average Velocity.

c. Permissible: (1) The highest velocity at which water may be carried through a structure, canal, or conduit without excessive damage. (2) The highest velocity throughout a substantial length of a conduit that will not cause scour.

d. Self Cleansing: The minimum velocity in drains necessary to keep solids in suspension and prevent their deposition and the subsequent nuisances from stoppage and odors on decomposition.

e. Velocity of Approach: The mean velocity in the conduit immediately upstream from a weir, dam, venturi tube, or other structure.

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New Firm Name _____

New Address _____

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New Phone No. _____

Mail To: El Paso County
Department of Public Works
3105 North Stone
Colorado Springs, Colorado 80907
Attention: Supervisor, Stormwater Division

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If Users of this manual change their address, this form must be completed and returned to the address below in order to receive future revisions and amendments.

PREVIOUS INFORMATION:

Old Firm Name _____

Old Address _____

_____

Old Phone No. _____

NEW INFORMATION:

New Firm Name _____

New Address _____

_____

New Phone No. _____

Mail To: Department of Public Works
City Engineering Division
P.O. Box 1575
Colorado Springs, Colorado 80901-1575
Attention: City Engineer