Drainage Design Manual

for Maricopa County, Arizona

2801 West Durango Street, Phoenix, Arizona 85009 (602) 506-1501

Acknowledgments

This manual was originally prepared by NBS Lowry Engineers and Planners, and McLaughlin Water Engineers, Ltd. under the direction of the Flood Control District of Maricopa County (FCDMC). Work groups composed of representatives from the FCDMC and various communities in Maricopa County were formed to advise the consultants about the applicability of technical criteria, special problem areas, and resolve conflicts over potential differences in drainage standards between communities.

The first edition of this manual was released in a draft format (November 1991) for review and comment by public and private sector engineers, and other interested parties. The FCDMC staff revised the manual using comments received from the public and reissued the manual with a date of September 1992.

The second edition of the manual was revised in January 1996 by FCDMC staff based on comments received from users. Letters were sent to all the Cities and Towns in Maricopa County informing them of the revisions to the manual, and inviting them to review the manual before the FCDMC released it.

The third edition of the manual was updated in a collaborative effort between the FCDMC and the City of Phoenix. Stantec Consulting Inc. performed the updating of the manual. Coincident with the updating of this manual, the Hydrology manual underwent revisions. This effort saw the creation of a third document to contain policies and standards that can be tailored to each community's needs. This manual along with the Hydrology Manual was completed in January 2004, but not released because of further pending revisions.

During 2005 and 2006, the policies and standards manual was reviewed and revised by a working committee composed of representatives from the Flood Control District, Maricopa County Planning and Development, Maricopa County Department of Transportation, and Maricopa County Environmental Services. Following completion of the policies and standards manual, the draft third edition of the Hydraulics manual was revised. A new chapter 7 was inserted and the following chapters renumbered. The completed third edition of the Hydraulics manual was released in March 2007 as a draft. Between March 2007 and April 2013, the manual was further refined, including a complete rewrite of Chapter 11 Sedimentation and fairly extensive technology updates to Chapter 6 Open Channels and Chapter 8 Hydraulic Structures.

The Flood Control District of Maricopa County wished to thank the many individuals who contributed to the preparation of this document. (1989, 1995).

Chapter 11 required extensive research to provide the additional depth and detail included in this new edition. It would not have been possible without the help of many individuals who contributed their time and experience to review and edit the chapter. The Flood Control District of Maricopa County wishes to express its appreciation for the efforts of everyone who contributed, and the following individuals in particular:

Gary Freeman, PhD, PE	George Sabol, PhD, PE	Weiming Wu, PhD
Yafei Jia, PhD	Aihua Tang, PhD, PE	Mike Zeller, MS, PE
Bob Mussetter, PhD, PE	Brian Wahlin, PhD, PE	Lyle Zevenbergen, PhD, PE
Hari Raghavan, PhD, PE	Sam Wang, PhD, PE	Shimin Zou, PhD, PE
Dennis Richard, MS, PE	David Williams, PhD, PE	

Comments

Users of this manual are welcomed to submit comments, suggestions, or findings of errors. This information should be addressed to:

Engineering Division Manager

Flood Control District of Maricopa County

2801 West Durango

Phoenix, AZ 85009

Revisions

Because of ongoing technical and administrative changes in the field of stormwater management, revisions to this manual will be required from time to time. Such revisions will take place on an ongoing, as needed, basis and will be posted on the FCDMC's Web page (www.fcd.maricopa.gov). The dates of revision and an overview of changes made are listed below.

1st Edition September 1, 1992

2nd Edition January 28, 1996

3rd Edition August 15, 2013

Overview of Changes Made in the Second Edition

The following is a summary list of the changes to the September 1, 1992 edition of the Drainage Design Manual, Volume II, Hydraulics. This summary of the revisions is only presented as an aid for users of the previous edition, it doesn't document every revision to the manual. Typically corrections for spelling, typographical errors, and revisions for readability are not documented here. When sections were moved, the renumbering of subsequent sections wasn't usually identified here. Due to the use of a Dew word processing program, there can be significant differences in the page numbering between this edition and previous editions. The sections or page numbers used in this list refer to the September 1, 1992 manual, unless otherwise stated.

Comments - Added this page requesting comments on the manual.

Acknowledgments - Added this page that identifies and thanks those individuals who have contributed to the manual in some official format.

Revisions - Added this section to summarize some of the significant changes to the September 1, 1992 edition.

Chapter 1 - Changed the use of the phrase "regulation" to "recommended uniform policy requirement." Revised the descriptions of the chapters to reflect the revisions made to the chapters. Revised the wording of the recommended uniform policy requirements to match with what is in the chapters.

Chapter 2 - In Table 2.1 changed the V_{min} from 2.5 to 3.0 fps for the 50 year peak frequency on cross road culvert collector and arterial streets. Revised the wording for the finish floor elevation for buildings within a FEMA floodplain area. In the footnote changed the minimum discharge for delineating a floodplain for submittal to FEMA from 1,000 cfs to 500 cfs. Also, added Section 2.4 (References).

Chapter 3 - In Section 3.1 made minor corrections to some of the definitions. In Table 3.1 revised the second footnote. Renumbered the equations to account for identifying a new equation 3.2. For equation 3.2 added a sentence on what terms were inserted into equation 3.1 in order to derive equation 3.2. Revised the wording in the recommended uniform policy box on page 3-6.

Chapter 4 - This chapter was divided into two chapters. The new Chapter 4 is titled Storm Drains, and the new Chapter 5 is titled Culverts and Bridges. In Section 4.1 deleted and added definitions as needed for the revisions to the new chapter. Replaced the whole method for analyzing storm drains (Section 4.2). This required the addition of several new sections and the complete revision to several old sections. Revised the wording slightly in the recommended uniform policy requirement on page 4-5. Added a section on minimum slope as Section 4.2.2.3. Had to revise the numbering for some of the sections because of the new Section 4.2.2.3. In Section 4.2.2.5 added a paragraph on minimum pipe size. Changed the title of Section 4.2.2.6 and revised some of the wording slightly. The methods to calculate the various losses are now all located in Section 4.3.3.

The new Chapter 5 on Culverts and Bridges begins with Section 4.3 from September 1, 1992 edition. Because of being broken out into a new chapter all the numbering for the sections changed. The wording in the recommended uniform policy requirement boxes on pages 4-73, 4-74, 4-81, 4-82, 4-83, and 4-85. In Section 4.3.2.2 revised the minimum velocity to 3 ft/s. In Section 4.3.2.7 the italic subsections were made into numbered sections. Sections 4.3.3.6 and 4.3.3.3 were relocated under Section 5.2.2. This was done in order to locate all the various losses together in one section.

Chapter 5 - The chapter had to be renumbered to 6. Section 5.2 was made into the first section of the chapter, which is consistent with the other chapters. Revised the recommended uniform policy requirement boxes on pages 5-12, 5-16, 5-30, 5-33, and 5-41. In Figure 5.1 changed the side slope of the riprap channel to 3:1. In Section 5.5.1.2 deleted the paragraph on the slope paving method and Figure 5.7. In Section 5.5.2.2 thickness of the lining is now determined using an ADOT reference. Table 5.4 was revised. On page 5-38 revised the thickness required for the riprap layer. In Section 5.5.3.3 revised the method for sizing riprap. Deleted Table 5.7. Revised Figure 5.10 to agree with the text. Changed the title of Table 5.10. In Section 5.6.3.1 changed the Q_{100} in the example problem to 565 cfs.

Chapter 6 - The chapter was renumbered to. Section 6.2 was made into the first section of the chapter, which is consistent with the other chapters. Revised the definitions of some of the symbols. Made the fourth paragraph on page 6-14 into a recommended uniform policy requirement. Moved the hydraulic jump analysis (Section 6.8.1) to just after Section 6.3.2.).

Chapter 7 - Made this part of the new Chapter 5 on Culverts and Bridges. Deleted Sections 7.3.1.1 to 7.3.1.4 because there wasn't enough information presented here to do a complete analysis of a bridge, and most designers will use a computer program for the analysis. From these sections only the recommended uniform policy requirement on page 7-7 needed to be kept. Revised the wording of the recommended uniform policy requirements on page 7-11. The minimum freeboard for a bridge was revised to two feet for the 100-year event. Section 7.3.2.1 was revised.

Chapter 8 - Created a new Section 8.1, which defines the symbols used in this chapter and modified the numbering of the other sections because of it. In Section 8.2.1.2 added an equation for determining the volume of retention required. Also, added a new recommended uniform policy requirement dealing with off -site flows. Revised the wording for the recommended uniform policy requirements on pages 8-4, 8-6, 8-7, and 8-18. Added a new section dealing with sedimentation right before Section 8.2.1.3. In Section 8.2.1.8 added a recommended uniform policy requirement about dry wells. The recommended uniform policy requirements on pages 8-18, 8-24, 8-25, and 8-30 were dropped although the text remains.

Chapter 9 - Revisions to this chapter were only to correct typographical errors.

Glossary - Revisions to this chapter were only to correct typographical errors

Index - A subject index was added to make it easier to find information in the manual.

Overview of Changes Made in the Third Edition

All Chapters: The policies and standards previously highlighted by boxes in each chapter were removed to a separate volume. This allows each jurisdictional entity to customize its policies and standards to meet its community’s needs.

Chapter 1 Introduction - The background section was changed to identify the history of the development of the third edition. The reasons for the updating the second edition were identified.

The sedimentation chapter summary was added. The summary of policies and standards was eliminated with a section on safety added. The Purpose section was revised to identify this document as a “Substantive Policy Document” as defined in A.R.S. 48-3641.6.

Chapter 2 Hydrology - Changes to this chapter were minimal, most of which were corrections for word selection. The table identifying hydrology design criteria was eliminated as this information is listed in a separate volume.

Chapter 3 Street Drainage - Chapter structure/format was revised to follow the following major sections:

1. Introduction: Intent of Chapter and source of information
2. Procedures: Technical guidelines for engineering analyses.
3. Instructions: Example problems.

Figures 3.9 through 3.19 (Curb Opening Inlet Capacity Curves for MAG Details) were removed. Chapter figures are revised/updated.

Chapter 4 Storm Drains - The following major sections were added/revised:

1. Introduction: Intent of Chapter and source of information
2. Procedures: Technical guidelines for engineering analyses.
3. Criteria: General criteria for hydraulic design and evaluation of storm drains.
4. Design Standards.
5. Design Examples.

The procedure for estimating losses that occur at a storm drain junction was replaced with the Thompson Equation (Los Angeles County Flood Control District, *Design Manual Hydraulic*, March 1982). The method for estimating the bend loss coefficient for curved and deflected sewers was changed. Procedures for estimating the hydraulic grade line for connector pipes (catch basin to trunk line) were added from the City of Phoenix, *Storm Drain Design Manual, Storm Drains With Paving of Major Streets*, July 1987. An appendix was added that provides a pressure plus momentum approach to estimate the hydraulic grade line through a storm drain junction.

Chapter 5 Culverts & Bridges - The introduction was revised to better identify the intent of the chapter. Discussions pertaining to trashracks was moved to Chapter 7, Hydraulic Structures. The discussion on scour hole geometry was eliminated. The procedure for Protection of Culvert Outlets was deleted and reference made to the procedure in Chapter 7. An equation that allows the estimation of scour depth was added to aid in the design of cutoff walls. The discussion on scour was eliminated with reference made to Chapter 10, Sedimentation.

Chapter 6 Open Channels - This chapter was re-organized in its entirety with several sections re-written, reorganized, or amended. Of particular note, the fundamentals of open channel hydraulics was expanded and relocated to the beginning of the chapter. The design procedures section and design checklist was removed while design guidelines remain.

Chapter 7 Hydraulic Structures - Discussions pertaining to the hydraulic analysis of trashracks and access barriers, spillways, side channel spillways (forthcoming), channel bifurcations, channel access ramps, grade control structures, groins, and guide dikes were added along with other design guidance related to these structures. The discussion on Low Flow Check Structures was eliminated.

Chapter 8 Stormwater Storage - The Detention/Retention chapter was renamed Stormwater Storage in order to eliminate confusion between the terms retention and detention. The lengthy discussion on safety was moved to Chapter 1. The discussion on trashracks was moved to Chapter 7. The section on flood routing was eliminated since it overlapped with other chapters of the Hydraulics Manual and Hydrology Manual. Design considerations for stormwater storage basins was expanded to elaborate multi-use concepts. The benefit of stormwater storage on water quality was described in more detail. The discussion on sedimentation was condensed with reference made to Chapter 10, Sedimentation.

Chapter 9 Pump Stations - The design criteria and checklist were revised and incorporated into the chapter. The remainder of the chapter was completely rewritten to add a basic discussion on pump station design and hydraulic analysis.

Chapter 10 Sedimentation - This chapter in its entirety was added.

Third Edition Dates of Revision

The following indicates the dates in which the draft third edition has been updated and summarizes revisions made after the draft release of this third edition in September 2003.

September 2003

1. The entire manual was reformatted for 2-sided printing.

2. Equation 6-14, which was missing, was added back in. Equation 6-15 was blank and was deleted. Subsequent equations were re-numbered.

December 2006

1. A new Chapter 7 Friction Losses in Open Channels was inserted and the following chapters renumbered. References throughout the document were revised to reflect the new chapter. Text revisions to accommodate the new Chapter 7 were added, particularly in Chapter 6.

2. Chapter 11 Sedimentation was revised to include comments received from the public and significant edits by Dr. Bing Zhao.

March 2009

1. Chapter 11 Sedimentation is currently under revision after a peer review by sediment mechanics experts from around the southwestern United States.

2. Chapter 5 Culverts and Bridges. Tables 5.2 and 5.3 revised to match the reference document intent.

3. Chapter 6. Revised Table 6.1 to remove duplicate items.

4. Chapter 6. Revised text under Section 6.6.3 to correctly address computing a combined correction factor, C, for adjusting the riprap size to arrive at a stable riprap size. The statement now matches HEC-11.

5. Chapter 6. Revised reference to Table 6.2 on page 6-62 to refer to Table 6.3 and Table 6.4.

6. Added Section 8.4.2 Riprap Aprons at Conduit Outlets.

April 2010

1. Entire Document. Miscellaneous revisions correcting references and typographical errors.

2. Chapter 6, Section 6.6.3. Revised riprap channel bank lining procedure.

3. Chapter 11. The entire chapter has been revised.

4. Additional revisions and corrections are in progress.

June 2010

1. Chapter 8. Corrected typographical errors in equation 8.20 on page 8-59, and in item 5 on page 8-62. Revised date in footer to June 2010.

2. Chapter 11. Revisions to text, references, and format. Revised date in footer to June 2010.

August 2013

1. Chapter 6. Design guidelines for channel linings were revised.

2. Chapter 7. Reorganization to make the chapter easier to read. Scanned figures and tables were re-worked to provide better quality. Fixed various typographical errors.

3. Chapter 8. Extensive revisions related to erosion and scour protection.

4. Chapter 11. Revisions to address public review comments.

5. Finalize manual for publication.

Approvals

APPROVAL BY CHIEF ENGINEER AND GENERAL MANAGER

The Drainage Design Manual for Maricopa County – Hydraulics is hereby approved and accepted for use within Maricopa County, AZ as best available technical information. This manual has been submitted to various Flood Control District of Maricopa County (FCDMC) staff, other agencies, consultants and the Public for technical review. Review comments have been addressed and the document is hereby incorporated into FCDMC and County Policy. The Hydraulics manual is only available in digital format and can be found on the FCDMC public web site at:

http://www.fcd.maricopa.gov/Pub/manuals/hydraulics.aspx

Refer to the Revisions section of the manual for a history of the changes made.

The objective of the Drainage Design Manual, Hydraulics, is to provide criteria and design guidance for storm drainage facilities in Maricopa County. This manual provides a convenient source of technical information that is specifically tailored to the unique hydrologic, environmental and social character of Maricopa County; and a consistent set of criteria that, when used by the local governing agencies and the land development community, will result in uniform drainage practices throughout the County.

This document is only advisory and, in conformance with A.R.S. 48-3641.6, is intended to inform the general public of the Flood Control District of Maricopa County's current approach or opinion to the requirements of the various federal, state and county floodplain and drainage related ordinances or regulations, including, where appropriate, the Flood Control District of Maricopa County's current recommended minimum practice, procedure or method of action based on that approach or opinion. This document is not intended to impose additional requirements or penalties on regulated parties or confidential information. Submissions made using other methodology shall be acceptable to the Flood Control District of Maricopa upon submission of scientific documentation and evidence showing that such methodology yields results that are consistent and in accordance with the requirements of the various ordinances and regulations. However, the burden of proof is on the applicant and may affect submittal review times.

Approved for use by:

9\9\13

Timothy S. Phillips, P.E. Date
Chief Engineer and General Manager
Flood Control District of Maricopa County

Page intentionally left blank

Table of Contents

4 STORM DRAINS

5 CULVERTS & BRIDGES

6 OPEN CHANNELS

7 FRICTION LOSSES IN OPEN CHANNELS

8 HYDRAULIC STRUCTURES

9 STORMWATER STORAGE

10 PUMP STATIONS

11 SEDIMENTATION

Page intentionally left blank

List of Tables

8 HYDRAULIC STRUCTURES

9 STORMWATER STORAGE

10 PUMP STATIONS

11 SEDIMENTATION

List of Figures

1 INTRODUCTION

2 HYDROLOGY

3 STREET DRAINAGE

4 STORM DRAINS

5 CULVERTS & BRIDGES

6 OPEN CHANNELS

7 FRICTION LOSSES IN OPEN CHANNELS

8 HYDRAULIC STRUCTURES

9 STORMWATER STORAGE

10 PUMP STATIONS

11 SEDIMENTATION

1 INTRODUCTION

TABLE OF CONTENTS

1.1 PURPOSE

The objective of the *Drainage Design Manual for Maricopa County, Hydraulics* (Hydraulics Manual) is to provide criteria and design guidance for storm drainage facilities in Maricopa County. There are two reasons to develop such a manual: 1) it provides a convenient source of technical information that is specifically tailored to the unique hydrologic, environmental, and social character of Maricopa County; and 2) it provides a consistent set of criteria that, when used by the local governing agencies and the land development community, will result in uniform drainage practices throughout the county. Use of the Hydraulics Manual will result in improved hydraulic performance of drainage facilities, uniformity in design practices across jurisdictional boundaries, and reduction of conflict between the regulatory agencies and the land development community. Recommended policy and standard requirements are provided in a separate volume and are jurisdictional specific. That is, each jurisdictional entity (municipal or county) will have its own policies and standards. In many cases, these may be the same or only slightly modified for each jurisdiction. For this reason, the user is encouraged to review the policies and standards for the jurisdiction in which the project is located.

1.2 BACKGROUND

The first edition of this manual was produced by a team of consultants and the Flood Control District of Maricopa County. Beginning in 1987, the manual was developed through a highly interactive process involving work groups for each major topic. The work groups were composed of the engineering consultant, the Flood Control District, representatives of the various communities in Maricopa County, and representatives of home builders and land developers. The work groups were charged with advising the consultant about applicability of technical criteria, special problem areas to be addressed, and resolving conflict over potential differences in drainage standards between communities.

The first edition was made available to the public in 1991. By that time, several communities had policies, standards, criteria, and/or guidelines already in place. As a result, many communities elected to utilize this manual in conjunction with their own policies, criteria, etc.

In 1998, the City of Phoenix, which was in the process of updating its drainage manual, started a collaborative effort with the Flood Control District of Maricopa County to meld their drainage manuals. The purpose was threefold. First, various technical aspects of both the City and County's manuals required updating due to advances in the engineering science and further experience with applications unique to Maricopa County. Second, advances in computer technology provided the opportunity to develop a living document that would be posted on the internet that encompassed unique engineering software for the design/evaluation of drainage facilities. Thirdly, Volumes I (Hydrology) and II (Hydraulics) of the *Drainage Design Manual for Maricopa County*, included recommended uniform policy requirements. As identified above, several communities had policies that varied, however slightly, from the recommended uniform policies. This third edition has afforded the opportunity for individual jurisdictional entities to have their own policies and standards to suit their particular needs within the confines of federal and state laws/regulatory requirements. Thus, the Hydrology and Hydraulics Manuals serve as technical manuals, thereby affording each community flexibility in setting policies.

1.3 SCOPE

The *Drainage Design Manual for Maricopa County, Hydraulics,* is divided into ten chapters that address the major subject areas of hydraulic design. The intent of this manual is to provide general design guidance for designs that are common to the Maricopa County environment. Complex designs requiring specific expertise are not included in this manual; however, where design exceeds the scope of this manual, the user is referred to documentation appropriate for that design. The following sections briefly summarize each of the chapters in the manual.

1.3.1 Introduction

Chapter 1 defines the purpose, background, and scope of the manual along with a brief summary of each chapter. It also includes a discussion of public safety associated with drainage structures.

1.3.2 Hydrology

Chapter 2 provides an overview of the hydrology criteria for drainage structures; the flood hydrology that is recommended for use in Maricopa County is contained in the Hydrology Manual. That manual provides for the use of the Rational Method for small, uniform watersheds, and for use of the Unit Hydrograph Method for larger watersheds with diverse surface conditions. The Hydrology Manual provides design rainfall criteria that have been developed specifically for Maricopa County, rainfall loss methods that are based on the best practical technology that is available for estimating surface retention losses and infiltration rates, and unit hydrograph procedures that have been selected and developed for the various land-uses in Maricopa County.

1.3.3 Street Drainage

Chapter 3 provides design guidelines for the drainage of streets using curbs and storm drain inlets. An overall approach to stormwater management includes using the street system to transport runoff to storm drain inlets, and for transporting runoff from storms that exceed the capacity of the storm drain system. Design criteria, design procedures, and design aids are provided for streets and gutters, intersections, and roadside ditches. Catch basins are discussed in regard to alternative types and suggested applications, capacities, and design procedures. The procedures used in this chapter were primarily adapted from the Federal Highway Administration Hydraulic Engineering Circular No.12 (HEC-12), *Drainage of Highway Pavements* (FHWA, 1984).

1.3.4 Storm Drains

Chapter 4 provides coverage of storm sewers. A comprehensive treatment of storm sewers is provided including use of design aids for catch basins, manholes, and various types of storm sewer junctions.

1.3.5 Culverts and Bridges

Chapter 5 provides coverage of the design information required for the design of culverts. This includes the necessary design aids, guidance for treatment of culvert inlets and outlets, and scour protection at the culvert outlet. Use of example problems helps to illustrate the procedures to be used for most practical applications. The charts and procedures for culvert design used in this chapter were taken from the Federal Highway Administration Hydraulic Design Series No. 5

(HDS-5), *Hydraulic Design of Highway Culverts* (FHWA, 1985). Some brief guidelines are presented to follow when designing inverted siphons. The design of bridges requires special expertise and experience in regard to hydraulic analyses, design of flow training works, and estimates of pier and abutment scour. Therefore, only an overview of the hydraulic analyses for bridge openings is presented.

1.3.6 Open Channels

Chapter 6 is devoted to the analysis and treatment of both natural and artificial channels. The scope of this chapter covers the more commonly encountered open channel design applications by designers who do not possess special design skills in open channel hydraulics. Applications involving rivers and large washes or channels, which are considered as non-rigid, require special design skills, and the design of these channels should not be attempted with the design techniques contained in this chapter. The design procedure presented provides an appropriate level of analysis for most design problems that will be encountered for artificial channels. The design procedure assumes a rigid channel, and is valid for both subcritical and supercritical flows. Channel linings of concrete, soil cement, riprap, wire-enclosed rock (gabion), and grass are discussed in the manual. The analysis of natural channels is discussed in broader terms than is the treatment of artificial channels. Although the basic theory is the same for both channel types, more complex flow conditions (nonuniform and unsteady flow) and concepts of sediment transport often need to be incorporated in the analysis of natural channels.

A guide for the estimation of friction losses in both natural and artificial channels is provided in Chapter 7. This guide was derived from the U.S. Geological Survey Scientific Investigations Report 2006-018 (Phillips and Tadayon, 2006).

1.3.7 Hydraulic Structures

The hydraulic structures that are described in Chapter 8 are used to control or alter the flow characteristics, such as velocity, depth, energy, and other hydraulic characteristics, and to affect a change in the configuration of an open channel, such as channel slope. The purpose of such structures is to achieve safer and more stable conveyance systems with improved maintainability. Channel drop structures are a major topic of this chapter and guidance is provided for the design of baffle chute drops, vertical hard basin drops, vertical riprap basin drops, sloping concrete drops, and grade control structures. Information is provided for the dissipation of energy at conduit outlet structures with emphasis on riprap protection for outlets with moderate flow conditions and concrete structures for more severe conditions. Guidance is provided for the design or evaluation of channel transitions, bifurcation structures, channel junctions, spillways, trash racks, access ramps, supercritical flow chutes, and bends in channels designed for supercritical flow. A brief discussion is provided on groins and guide dikes. The manual provides instruction in the theory and use of the hydraulic jump as a means of energy dissipation. The design of various, appropriate hydraulic jump energy dissipaters are included.

1.3.8 Stormwater Storage Facilities

Chapter 9 presents the engineering methodologies and details associated with the planning, analysis, and design of stormwater storage facilities. Detention and retention basins are man-made storage facilities that are intended to mitigate the effects of urbanization on storm drainage. They serve to reduce peak discharges and can also reduce the volume of storm runoff downstream of the basin under certain conditions. Since detention and retention basins often require a considerable commitment of land resources by the community or land developer, particular emphasis is placed on planning basins that are amenities, and, where possible, incorporate multiple-use concepts. National stormwater quality standards are being promulgated and criteria for use of detention and retention basins that will not jeopardize the quality of surface water and groundwater resources are presented. The theory and procedure for performing routing of an inflow flood through such facilities is provided.

1.3.9 Pump Stations

The criteria for use of pump stations in Maricopa County are provided in Chapter 10; however, the intent is to provide only an overview of the conditions that should be considered in the design of stormwater pumping facilities. Stormwater pump stations are used where gravity discharge is infeasible, such as depressed highway intersections, or for the controlled release of outflow, such as from a detention or retention facility. Reference to another readily available document for the rigorous design of stormwater pump stations is also provided.

1.3.10 Sedimentation

Chapter 11 provides an overview of sediment transport theory. There is a general discussion of scour and sedimentation. It provides basic concepts of sedimentation engineering and analytical methods and design procedures for sediment yield and scour estimation in support of the goal of minimizing maintenance. It identifies considerations to be taken in the design of culverts, bridges, channel, and stormwater storage facilities to minimize maintenance from scour and sedimentation. It is not intended to be all-inclusive, but instead, its purpose is to identify the issues and provide references for further consideration by the design engineer.

1.4 SAFETY

During storm events, people are known to intentionally or inadvertently enter water that is dangerous during flood conditions. Or, worse, purposely boat or float in drainage facilities during high runoff levels. It is not possible to develop drainage facilities that are without hazard, that will preclude people from doing unintelligent acts, and that will also be hydraulically efficient. These objectives are, for the most part, mutually exclusive. However, reasonable levels of protection can be provided to people exercising reasonable judgement even when the structure is performing its primary function, i.e., efficiently passing storm water.

An overriding goal of any public improvement project is to protect, maintain, and enhance the public health, safety and general welfare by establishing requirements and procedures to control the adverse affects of stormwater runoff and pollution.

The issue of safety includes the following principles:

- Stormwater naturally accumulates, frequently in amounts that present hazards to property, traffic, and life and health.
- Because of the accumulation of stormwater, certain levels of hazards cannot be eliminated.
- There are three levels of safety to consider, in order of priority:
    - Life and Health
    - Traffic
    - Property

Public access and safety are inherent elements in the design of all drainage facilities. These elements are of primary importance, particularly in the case of multiple-use facilities where public use is encouraged in areas subject to potential flooding. The primary factors associated with safety at stormwater storage or conveyance facilities are user education, advance warning, potential water depth/velocity, slopes, escape routes from flooded areas, and time to drain.

These factors can be addressed in two ways. The first relates to the need to identify and communicate potential hazards to the public. For example, with proper signage, users can be made aware of the existence of potential hazards, such as flooding, high velocity flows, etc. User education is a fundamental element in safety design for a stormwater facility. Clear, concise signage with illustrative graphics can inform the public of the primary flood control purpose of the facility and describe the various features and their potential danger during a flood.

The second relates to the design of the facility to include safety devices that can be readily maintained. Appropriate steps should be taken to mitigate potentially dangerous conditions. Where the dangerous condition cannot be prevented, appropriate measures should be implemented to keep users away from hazardous locations. Advance warning (alarms or lights triggered by upstream water levels) should be considered for multiple-use facilities, particularly where flash flooding and rapid basin inflow is possible.

Safety devices can be divided into two types:

Devices that Limit or Deter Access

- Fencing
- Guard rails

- Warning signs
- Safety barriers

Devices that Permit Escape

- Safety nets & cables
- Safety racks (to prevent persons already in a flood hazard from passing to an area of more severe hazard)
- Egress facilities (mild slopes, stepped walls, ladders, etc.)

An important distinction between these two categories is that devices that permit escape, may also impede the flow of stormwater into or through drainage facilities.

Safety devices for drainage facilities should be considered for both dry weather and runoff conditions. Dry weather hazards include traffic and personal safety. Examples of traffic hazards include improper placement of guardrails on structures, unprotected drops at structures located near roads, and grading, all of which promote vehicle rollovers. Dry weather hazards include vertical drops or walls that may present hazards to the public that would be attractive to them for unsafe recreation.

The basic concept of this proposed approach to safety is to apply more restrictive measures as hazards increase. The primary purpose for constructing drainage facilities is the efficient conveyance of stormwater to minimize property damage and to permit traffic flow across and parallel to drainageways; therefore, safety in this context refers to protection from life and health hazards.

Safety considerations by hydraulic topic are enumerated below:

1.4.1 Street Drainage

Streets are used for the conveyance of stormwater. Excessive stormwater depths threaten safe vehicular passage, including passage of emergency vehicles. Gutter flow depth should not exceed 8 inches for the design storm used as the basis for stormwater storage. Refer to governing agencies drainage policy and standards manual for guidance. When grated catch basins are used, the engineer should design them to optimize hydraulic efficiency, bicycle and pedestrian safety, and structural adequacy.

1.4.2 Storm Drains & Culverts

During design, conduit entrances may require additional consideration for safety and for debris transported by stormwater. Frequently, trash collection devices are also used as safety devices. The need for trash collection or safety devices should be determined during planning and before the design of drainage facilities.

Access barriers at conduit outlets prevent access and potential entrapment during dry periods.

Access barriers serve a similar safety function as trashracks. It is rare that cost-effective access barriers and trash collectors can be retroactively added without a reduction of intended system design capacity.

When any of the following conditions are met, trashracks should be required on the entrances and access barriers on outlets to all conduits or other hydraulic structures:

- When a conduit or hydraulic structure outfalls into a channel with side slopes steeper than 4(H): 1(V) for hydraulically smooth (concrete and soil cement) banks, 3(H): 1(V) for riprap linings, 2(H): 1(V) gabion embankments, and 1(H): 1(V) stepped side slopes.
- Conduits and hydraulic structures with a cross sectional area of 20 square feet or less.
- Conduits and hydraulic structures with a cross sectional area greater than 20 square feet and longer than 200 feet in length.
- Conduits and hydraulic structures with energy dissipaters at the end.
- Conduits and hydraulic structures being used as outlets from multiple-use detention facilities.
- Conduits and hydraulic structures with sufficient bend that the opposite ends cannot be clearly seen.

Flap gates can be considered for substitution for access barriers on conduit outlets when it can be shown that sedimentation will not prevent the flap gate from opening or that the design of the outlet structure will reduce downstream sedimentation that would prevent the flap gate from opening.

1.4.3 Open Channels

Deep channels with steep side-slopes and high flow velocities can be a hazard to the health, safety, and welfare of the general public. Therefore, the design engineer should always consider the safety aspects of any design. Fencing should be provided for all supercritical channels regardless of depth. Depending upon velocity, shallower subcritical channels may require fencing. Concrete, shotcrete, or smooth sided soil cement channels meeting certain criteria should have emergency escape ladders or equivalent. Refer to the governing agencies drainage policies and standards manual for guidance. In instances where open channels connect conduits that meet the geometric and hazard requirements previously listed, safety devices are recommended to restrict access by the general public along the entire reach of that channel. An example would be a concrete lined channel with 1(H):1(V) side slopes.

1.4.4 Hydraulic Structures

Hydraulic structures constructed in Maricopa County will usually be subject to public access. Designs for hydraulic structures should address the issue of safety. First, signage should be provided to identify the potential hazard of flooding or dangerous flow measures to the public. Second, appropriate measures should be designed to keep the public away from hazardous locations. For example, vertical drop structures should not exceed 2.5 feet in height, and adequate fencing or railings should be provided along all other walls, such as wing walls or training walls.

1.4.5 Stormwater Storage

Often higher flood flow is directed into a multiple-use stormwater storage facility by an overflow side channel spillway or by a drop structure. A large volume of water entering the facility at high velocity can literally wash away an individual who is on or near the inlet structure. The design of an inlet that minimizes the velocity of incoming water will greatly enhance safety and should be included in the criteria for inlet structure design. Railing or fencing should be provided at the top of structural walls.

Within a stormwater storage facility, safety concerns increase with an increase in potential water depth. A facility with a potential water depth of 2 to 3 feet (less than the head height of most users) is typically less dangerous than a facility with a potential water depth of 5 to 6 feet, or more. For reasons of safety, potential water depth in detention/retention facilities should be kept to a minimum. When possible, potential water depth of 3 feet or less is recommended for small stormwater storage basins immediately next to residential areas.

In all facilities, regardless of depth, slopes in flood-prone areas should be kept as shallow as possible. This will allow users who find themselves caught in flooded areas (or users who deliberately enter flooded areas) to walk out and up to non-flooded zones. It is recommended that slopes in flood-prone areas be 4(H): 1(V) or flatter.

For facilities that feature permanent pools, public safety should be a primary criterion in the design. The pond edge should be designed to minimize safety hazards. Water depth should be limited to 1.5 to 2 feet within 8 feet of the shoreline. Where the permanent pool design depth exceeds these recommendations at the pond edge, other safety measures should be considered.

In addition to slopes, consideration should be given to bottom conditions in flood-prone areas. Soils that provide firm footing when saturated are safer than soils that do not. In severe cases of unsuitable soils, partial or total removal may be necessary.

In addition to gentle slopes, routes out of flood-prone areas should be provided. Barriers that could trap a user in a flood-prone area should be avoided. Safe, well-signed exit routes that are negotiable under wet conditions should be developed.

User safety should be of primary concern with the design of outlets or drains. They should be designed so that it is not possible for a user to be trapped during wet or dry conditions (see discussion above regarding trashracks and access barriers). This is particularly important when considering children using the outlet structures as a playground.

A properly designed trashrack can prevent clogging by debris as well as prevent a person from being swept into the outlet structure and pipe. In addition, where hydraulic conditions at the outlet structure can lead to the formation of a vortex, the design should include anti-vortex protection. It is important to note, however, that an outlet structure is not a safe structure during flood conditions, whether it is a horizontal pipe outlet or a riser type structure mounted to a horizontal pipeline. Powerful inlet velocities can draw a person underwater at the outlet structure regardless of the existence of a trashrack or grate. Signage is important to alert the public of this danger. In addition, trashracks should be designed to prohibit, to the extent practical, a small child being forced through the openings.

All site furnishings, such as benches, trash receptacles, and picnic tables should be secured to prevent them from becoming waterborne-debris that could clog the outlet structure.

Safety should also be considered downstream of outlet structures. Release flows, even though they may be controlled, can present a hazard. Specific conditions downstream of an outlet should be evaluated in terms of safety. To protect the public, structural walls should have fencing or railing along the top of an outlet structure.

1.5 SUPPLEMENTAL DISCIPLINES

1.5.1 Geotechnical Engineering

Geotechnical investigations may be required for designs of embankments, infiltration wells (for draining retention basins), storm sewers, berms, levees, culverts, and rigid lined channels. Determination of foundation characteristics and evaluation of soil materials proposed for construction is routinely required for many drainage projects. Samples obtained from borings and exploratory pits should be tested under laboratory conditions to evaluate more precisely the soil and rock classification properties, strength, permeability, compatibility and other specialized tests pertinent to the specific project conditions. The results of these analyses are used to develop guidelines for economic and safe designs. This Drainage Design Manual does not go into the requirements and procedures for geotechnical studies. Nonetheless, the designer must always recognize the importance of this information and secure this expertise as appropriate for the project at hand.

1.5.2 Structural Engineering

Structural engineering expertise is required in applications where standard details (i.e. MAG, ADOT, etc.) do not meet the project's needs. Here, the structural engineer assesses the anticipated loads or forces that the drainage structure must endure and specifies the materials and geometry for the structure. This Drainage Design Manual does not provide guidance for structural analysis. When the design engineer faces situations where the available standard details can not be applied or there is reason to doubt the applicability of a standard, a structural engineer must design the drainage improvement for structural integrity.

1.5.3 Environmental Expertise

Stormwater drainage improvements often co-exist, interact, or interfere with other man-made or natural resources. The designer of drainage improvements must consider these during the design process. Depending upon the project at hand, specialty studies related to archeology, waters of the U.S., historic properties, wildlife, hazardous waste, etc. might be required. With recognition of these various issues, the designer must realize the need for a specialist to assist with the design of the stormwater drainage improvement/project. Often times, alternative alignments or configurations are required to avoid or mitigate these other resources. This manual does not delve into these resource issues nor provide guidance as to their mitigation.

1.6 REFERENCES

Phillips, J.V., and Tadayon, S., 2006, Selection of Manning's roughness coefficient for natural and constructed vegetated and non-vegetated channels, and vegetation maintenance plan guidelines for vegetated channels in Central Arizona: U.S. Geological Survey Scientific Investigations Report 2006-5108, 41 p.

United States Department of Transportation, Federal Highway Administration, 1984, Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12.

——, 1985, Hydraulic Design of Highway Culverts, Hydraulic Design Series No.5.

Page intentionally left blank

2 HYDROLOGY

TABLE OF CONTENTS

2.1 METHODOLOGY

The determination of flood hydrology for designing stormwater facilities in Maricopa County is to be made using one of the following:

- Existing studies of record that have been approved by the jurisdiction. Such studies include flood insurance studies, Area Drainage Master Studies and Plans, and design reports for adjacent facilities.

- If appropriate existing information is not available, then the procedures set forth in the *Drainage Design Manual for Maricopa County, Hydrology Manual* (FCDMC, 2008), hereinafter referred to as the Hydrology Manual are to be used.

Peak flow rates and volumes from studies or reports of record should be checked for reasonableness using the procedures set forth in Chapter 8 Indirect Methods of the Hydrology Manual. Use of historical data of record and deviations from the procedures in the Hydrology Manual require prior approval from the jurisdictional agency and/or the Flood Control District of Maricopa County (FCDMC) before proceeding with the determination of design hydrology.

It is not the intent of the Hydrology Manual to inhibit sound, innovative analysis, utilization of superior technology, or the development of improved techniques. Therefore, the investigation, development, and use of the best practical technology for flood hydrology is strongly encouraged in all situations.

The selection of the procedure used to determine the design flood hydrology is dependent upon the intended application. For small urban watersheds (defined as less than 160 acres and having fairly uniform land use), the use of the Rational Method is acceptable. Use of this method will only produce peak discharges and it should not be used if a complete runoff hydrograph is needed, such as for the routing of flow through a detention facility. For larger, more complex watersheds or drainage networks, a rainfall-runoff model should be developed. The Hydrology Manual provides guidance in the development of such a model and the estimation of the necessary input parameters to the model.

Although not necessarily required, the use of the U.S. Army Corps of Engineers' HEC-1 Flood Hydrology Program (USACE, 1998) facilitates the use of the procedures that are contained in the Hydrology Manual, which was written to supplement the HEC-1 User's Manual.

All of the hydrology that is required for the design of stormwater storage facilities that are normally encountered can be performed by using the HEC-1 program. The design and performance of pump stations cannot normally be satisfactorily performed using the simplified procedures that are incorporated in the HEC-1 program. Although the inflow hydrograph to a pump station can be adequately developed with HEC-1, the performance and design of pump stations will often require the use of specialized programs. Furthermore, HEC-1 does not efficiently model street drainage/storm drain systems.

2.2 CRITERIA

The Hydrology Manual is to be used to develop the design discharge for storms of frequencies up to and including the 100-year event. Section 2.1.2 of the Hydrology Manual lists the different durations to be analyzed depending upon the size of the drainage area.

All development should make provisions to retain the peak flow and volume of runoff from rainfall events up to and including the 100-year, 2-hour duration storm falling within the boundaries of the proposed development. The criteria to be applied is provided in the *Policies and Standards Manual* for the local jurisdiction. Refer to the *Uniform Drainage Policies and Standards for Maricopa County, Arizona* (FCDMC, 2007) for criteria for unincorporated Maricopa County.

2.3 DRAINAGE PLANNING

Drainage planning shall be done in the earliest stages of the planning process. A drainage plan shall incorporate the hydrologic analysis for on-site and off-site runoff and outline the recommended plan for handling stormwater runoff.

Drainage planning can be encountered on both basin-wide and local scales. When undertaking a basin-wide plan, the designer must comprehensively evaluate practical alternatives to find the most cost-effective solution for the general public. When preparing drainage plans for local development, the designer shall illustrate conformance with basin-wide drainage plans where they exist, or shall demonstrate that the plan will not increase extraordinarily the cost of providing basin-wide drainage for the local agency or the FCDMC.

The planning process begins with the conceptual layout of the drainage system, which includes both large and small drainage facilities. All drainageway entrance and exit points in the proposed development must remain in the original location and, as near as possible, in the original condition.

In many areas about to be urbanized, the runoff has been so minimal that natural channels do not exist. However, surface depressions normally exist and will provide an excellent basis for the initial siting of open channels. This condition is also true for open channels that are to be used primarily for road or highway drainage.

Drainage plans illustrate selected alternatives, including the footprint of facilities or land uses, approximate sizes, and physical impact on the land. General requirements for structures and their overall size and impacts are also determined during the master planning phase; however, detailed selection of structure types, sizing of riprap, structural design, and selection and detailing of peripheral elements (inlets, trashracks, fencing, etc.) are completed in later phases using the criteria outlined in this manual.

2.4 REFERENCES

Flood Control District of Maricopa County (FCDMC), 2007, *Uniform Drainage Policies and Standards for Maricopa County, Arizona*.

——, 2008, *Drainage Design Manual for Maricopa County - Hydrology*, 4th Edition.

U.S. Army Corps of Engineers (USACE), 1998, *HEC-1 Flood Hydrograph Package, User's Manual*.

Page intentionally left blank

\

3 STREET DRAINAGE

TABLE OF CONTENTS

3.1 SYMBOLS

The following symbols will be used throughout Chapter 3.

a = Gutter depression, inches

a' = Inlet depression, inches

A = Clear opening area, or flow area, sq ft

A_g = Clear area of grate, sq ft

C_o = Orifice coefficient

C_w = Weir coefficient

d = Depth of flow at curb measured from the normal cross slope, ft (i.e., $d = TS_x$)

d_i = Depth of flow at lip of curb opening, ft

d_o = Effective depth of flow at the center of the curb-opening orifice, ft

E = Hydraulic efficiency of an inlet shorter than the length required for total interception (Q_i/Q)

E_o = Ratio of flow in the depressed section to total gutter flow

g = Gravity, 32.2 ft/sec^2

H = Height of curb opening catch basin, curb-opening orifice, or orifice throat width, ft

L = Length of curb opening, grate or slot, ft

L_T = Curb-opening length required to intercept 100% of the gutter flow, ft

n = Manning's roughness coefficient

P	=	Perimeter of the grate, disregarding bars and side against the curb, ft
Q	=	Total gutter flow rate, cfs
Q_{cap}	=	Allowable flow rate per gutter, cfs
Q_i	=	Amount of street flow intercepted by inlet, cfs
Q_s	=	Flow rate in paved area, cfs
Q_t	=	Theoretical gutter carrying capacity, cfs
Q_w	=	Flow rate in width W, cfs
r_H	=	Hydraulic radius, ft
R_f	=	Ratio of frontal flow intercepted to frontal flow
R_s	=	Ratio of side flow intercepted to total side flow
S	=	Longitudinal street slope, ft/ft
S_e	=	Equivalent cross slope, ft/ft
S_x	=	Pavement cross slope, ft/ft
S_w	=	Cross slope of a depressed gutter, ft/ft
S'_w	=	Cross slope of a depressed gutter section measured from the normal cross slope of the pavement (a/W), ft/ft
T	=	Width of flow, spread, ft
T_s	=	Spread of flow on the pavement for a composite section, ft
V	=	Velocity of flow in the gutter, ft/sec
V_o	=	Gutter velocity where splash-over first occurs, ft/sec
W	=	Width of grate, width of slotted drain slot or width of gutter, ft
Y	=	Depth of flow, ft
Z	=	Reciprocal of pavement cross-slope, $1/S_x$, ft/ft

3.2 INTRODUCTION

3.2.1 General Discussion

The intent of this chapter is to provide guidelines and procedures for the removal of stormwater flow from urban roadways. Removal of stormwater from roadways during frequent events minimizes the nuisance of flow on the roadway to traffic thus allowing traffic to move safely and efficiently. The removal of stormwater from roads is also essential to reducing maintenance cost.

3.2.2 Source of Data

This chapter describes methodology that should be used for the estimation of street flow capacity, allowable spread, and catch basin design. The procedures, equations, and nomographs in this section are adapted from the Federal Highway Administration, Hydraulic Engineering Circular No. 22 (HEC-22), *Urban Drainage Design Manual* (USDOT, FHWA, 1996) and U.S. Department of Transportation, Federal Highway Administration (FHWA), March 1984, *Hydraulic Engineering Circular No. 12, Drainage of Highway Pavements*. Policies and Standards relative to Street Drainage are listed in the *Policy and Standards Manual*.

3.3 PROCEDURES

3.3.1 General Considerations

The procedures are established for the collection of storm drainage on urban streets. Storm drainage may outfall to a designed storm drain or channel, a natural channel or a retention facility. Typical urban street sections can be obtained from the appropriate governmental agency.

Catch Basin Selection

Catch basins used for drainage can be divided into four main categories, curb-opening catch basins, grated catch basins, combination catch basins and slotted drain catch basins. Typical catch basin inlets are shown in Figure 3.1. Catch basins may be further classified as being on a continuous grade or in a sump. The continuous grade condition exists when the street grade is continuous past the catch basin and the water can flow past. The sump condition exists whenever water is restricted to the catch basin area because the catch basin is located at a low point. This may be due to a change in grade of the street from positive to negative or due to the crown slope of a cross street when the catch basin is located at an intersection.

Curb-opening catch basins are effective in the drainage of roadways. Curb-openings are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians, individuals using mechanical aids for commuting, and bicyclists. A depressed-curb opening is hydraulically more efficient than an undepressed curb-opening.

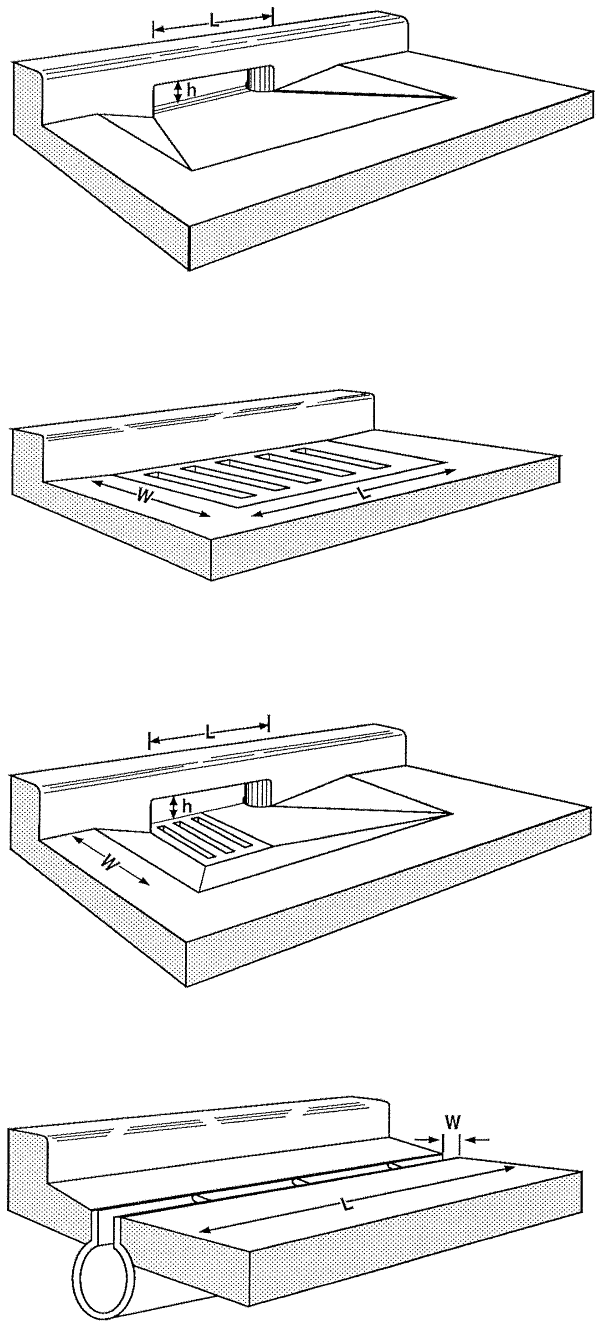
Grated or gutter catch basins refers to an opening in the gutter covered by one or more grates through which water falls. As with other catch basins, grated catch basins may be depressed or undepressed and are more efficient than curb-opening catch basins when located on a continuous grade. When grated catch basins are used, the engineer should design them to optimize hydraulic efficiency, bicycle and pedestrian safety, and structural adequacy. Grated catch basins shall not extend into traffic lanes.

The interception capacity of a combination catch basin on a continuous grade consisting of a curb-opening and grate placed side by side is not appreciably greater than that of the grate alone. The interception capacity is computed using only the grate for this situation. A combination catch basin with the curb-opening longer than the grate has additional capacity. The curb-opening in such an installation intercepts debris which might otherwise clog the grate and has been termed a “sweeper” by some. A combination inlet with a curb-opening upstream of the grate has an interception capacity equal to the sum of the two inlets, except that the frontal flow and thus the interception capacity of the grate is reduced by interception of the curb.

In a sump, combination inlets are very desirable. The curb-opening provides a relief if the grate should become clogged.

A slotted drain is a slot opening in the pavement which intercepts sheet flow and conveys it through a pipe (normally corrugated steel). Slotted drains are most effective when street slopes are shallow. Slotted drains can be used on curbed or uncurbed sections and offer little interference to traffic operations.

FIGURE 3.1
CATCH BASIN INLETS



(a) Curb Opening Catch Basin Inlet

(b) Grated Catch Basin Inlet

(c) Combination Catch Basin Inlet

(d) Slotted Drain Catch Basin Inlet

Site Specific Design Considerations

Figure 3.2 is a typical illustration of the variations in grade when local streets intersect. When local streets intersect arterial or collector streets, the grades of the arterial or collector street should be continued uninterrupted.

When collector and arterial streets intersect, the grade of the more major street should be maintained as much as possible. For drainage purposes, no form of valley gutter should be constructed across an arterial street. Occasionally, with agency approval, valley gutters may be considered on collector streets.

Conventional valley gutters may be used to transport runoff across local streets when a storm drain system is not required *and* when approved by the governmental agency. The valley gutter should be sufficient to transport the runoff across the intersection with encroachment equivalent to that allowed on the street.

The theoretical carrying capacity of each gutter approaching an intersection shall be calculated based upon the effective slope, as outlined herein.

When the gutter slope will be continued across an intersection – as when valley gutters are in place – use the slope of the gutter flow line crossing the street to calculate capacity.

When the gutter flow must undergo a direction change at the intersection greater then 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, and 50 feet upstream from the point of direction change.

When the gutter flow is 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.

In highly concentrated business areas where large volumes of pedestrian traffic are likely, consider using walk-over curbs (where pavement grade is raised to match the curb elevation at the crosswalk) at intersections. If used, however, two catch basins would be required at nearly every corner as flow may not be allowed to continue around the corner.

Where concentration of pedestrians occurs, depth and flow area limitations may need modification. Designing for pedestrian traffic is as important as designing for vehicular traffic. Ponding water and gutter flow wider than 2 feet is difficult for pedestrians to negotiate.

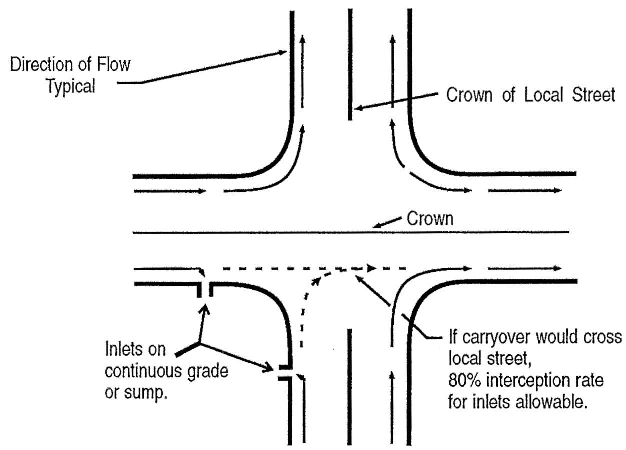
Storage Facilities

In some areas it may be favorable to retain street drainage within retention facilities. This is acceptable with approval from the appropriate governmental agency. Please refer to Chapter 8 for storage facility design.

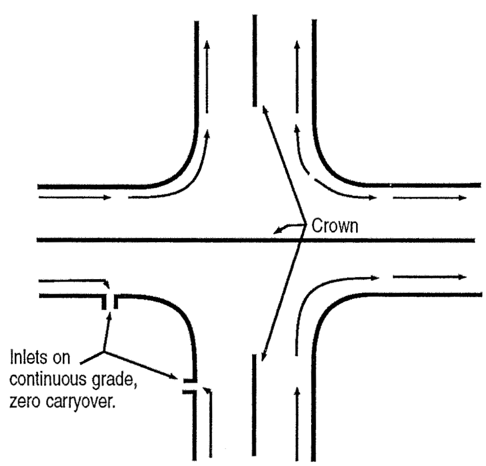
Roadside ditches are commonly used in rural areas to convey runoff from the highway pavement, and from areas which drain toward the highway. Where practicable, the flow from major areas draining toward curbed highway pavements should be intercepted by ditches.

These examples show the minimum required inlets. Additional inlets may be necessary based upon allowable carrying capacity of gutters.

FIGURE 3.2
TYPICAL STREET INTERSECTION DRAINAGE TO STORM DRAIN SYSTEM



Local or Collector to Arterial



Arterial to Arterial
(One Continuous Crown)

The following criteria pertain to the design of open channels along roadsides. For additional criteria for open channels, see Chapter 6.

Roadside ditches adjacent to public streets are discouraged in urban areas and require approval from the governing agency. When they are allowed, adhere to the criteria outlined in this section.

Depth of flow in roadside ditches for the design storm shall be limited to preclude saturation of the adjacent roadway subgrade. Where curbs exist and roadside ditches are used in lieu of storm drains, catch basins or scuppers should be provided as needed to drain the pavement into the drainage ditch.

Geometric considerations in the design of channel cross sections should incorporate hydraulic requirements for the design discharge, safety, minimization of right-of-way acquisition, economy in construction and maintenance, and good appearance.

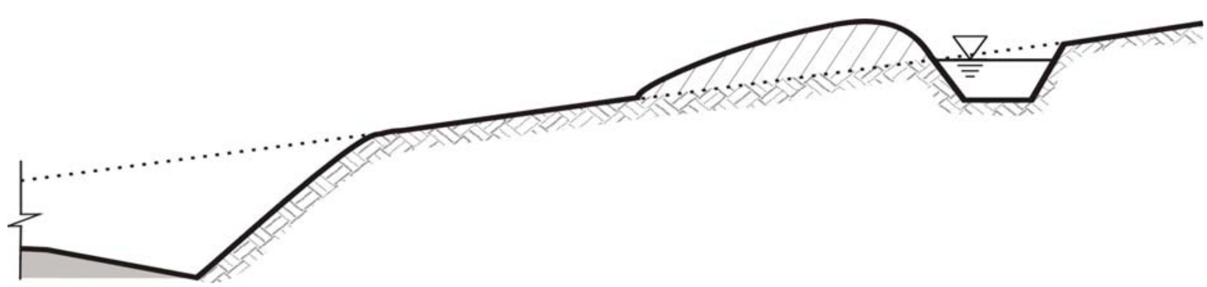
Channel side slopes should be as mild as practical and should be no steeper than 4:1 where terrain and right-of-way permit. The advantages of mild slopes are that the potential for erosion and slides is lessened, the cost of maintenance is reduced, and the safety of errant vehicles is enhanced. Safety considerations are subject to the requirements of the local jurisdiction.

Trapezoidal channel bottoms should be a minimum of 4 feet wide for maintenance purposes. V-shaped channels may also be used when approved by the governing agency.

Local soil conditions, flow depths, and velocities within the channel are usually the primary hydraulic considerations in channel geometric design; however, terrain and safety considerations have considerable influence. Steeper side slopes of rigid, lined channels may be more economical and will improve the hydraulic flow characteristics. The use of steeper slopes is normally limited to areas with limited right-of-way where the hazard to traffic can be minimized through the use of guardrails or parapets.

Rural Crown Ditch: In mountainous terrain where large cuts are required, crown ditches constructed on top of the cut embankment will intercept runoff preventing it from eroding the face of the cut slope. A typical crown ditch is shown in Figure 3.3.

FIGURE 3.3
CROWN DITCH



3.3.2 Applications and Limitations

Street Capacity

When estimating the total capacity of a roadway (curb to curb or sidewalk to sidewalk) Manning's equation as expressed in Equation (3.1) shall be used.

$$Q = A\left(\frac{1.49}{n}\right)r_H^{0.67}S^{0.5} \tag{3.1}$$

where:

Q	=	Total flow, cfs
n	=	Manning's roughness coefficient. A n-value of 0.015 or 0.016 is typically used for paved streets unless special conditions exist.
A	=	Flow area, sq ft
r_H	=	Hydraulic radius, ft
S	=	Slope of energy grade line, assumed equal to longitudinal street slope, ft/ft

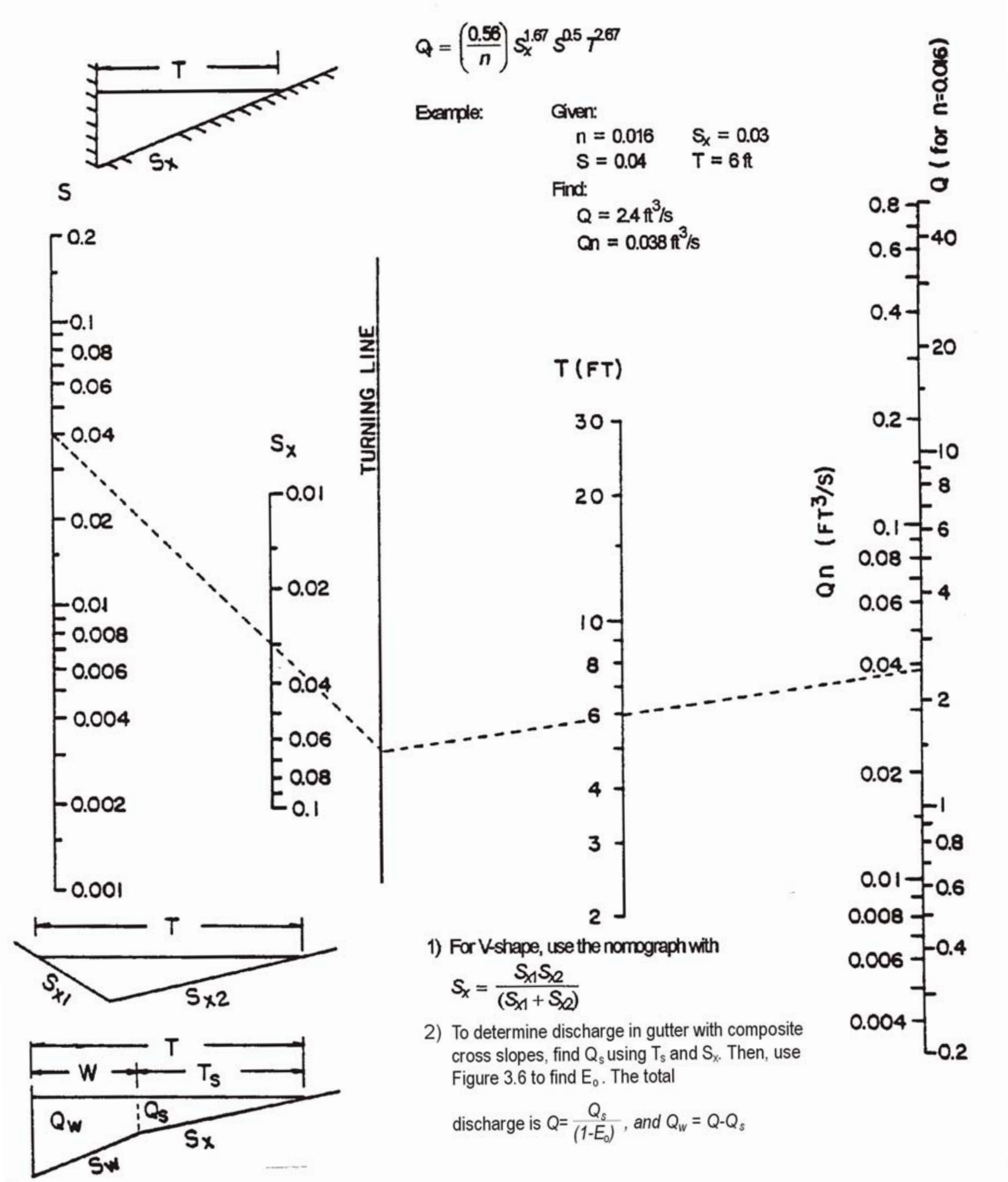
When the allowable pavement spread has been determined, the theoretical gutter carrying capacity shall be computed using the modified Manning's formula as expressed in Equation (3.2) or shown on Figure 3.4.

$$Q_t = \left(\frac{0.56}{n}\right)S_x^{1.67}S^{0.5}T^{2.67} \tag{3.2}$$

where:

Q_t	=	Theoretical gutter carrying capacity, cfs
T	=	Spread of flow on pavement, ft
S_x	=	Pavement cross slope, ft/ft
S	=	Longitudinal slopes, ft/ft

FIGURE 3.4
NOMOGRAPH FOR TRIANGULAR GUTTERS
(USDOT, FHWA, 1984, HEC-12, CHART 3)



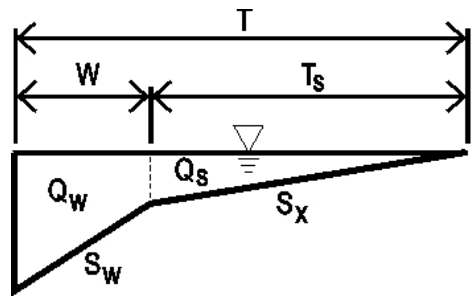
1) For V-shape, use the nomograph with

$$S_x = \frac{S_{x1} S_{x2}}{(S_{x1} + S_{x2})}$$

2) To determine discharge in gutter with composite cross slopes, find Q_s using T_s and S_x. Then, use Figure 3.6 to find E_o. The total discharge is $Q = \frac{Q_s}{(1-E_o)}$, and $Q_w = Q - Q_s$

For gutters with composite cross-slopes, pavement spread is determined using the relationships presented in Figure 3.5.

FIGURE 3.5
COMPOSITE CROSS-SLOPE GUTTER SECTION



To determine discharge in a gutter with a composite cross-slope, a multi-step analysis is required. First, find Q_s, using Equation (3.3). Next, find the total gutter flow (Q) using Equation (3.5) or Figure 3.6. Then determine the ratio of flow in the depressed section to total gutter flow using Equation (3.4). Gutter flow (Q_w) can then be determined using Equation (3.6).

$$Q_s = \left(\frac{0.56}{n}\right) S_x^{1.67} S^{0.5} T_s^{2.67} \tag{3.3}$$

where:

Q_s = Flow rate in paved area, cfs
T_s = Spread of flow on pavement for a composite section, ft
S = Longitudinal slope, ft/ft
S_x = Pavement cross-slope, ft/ft

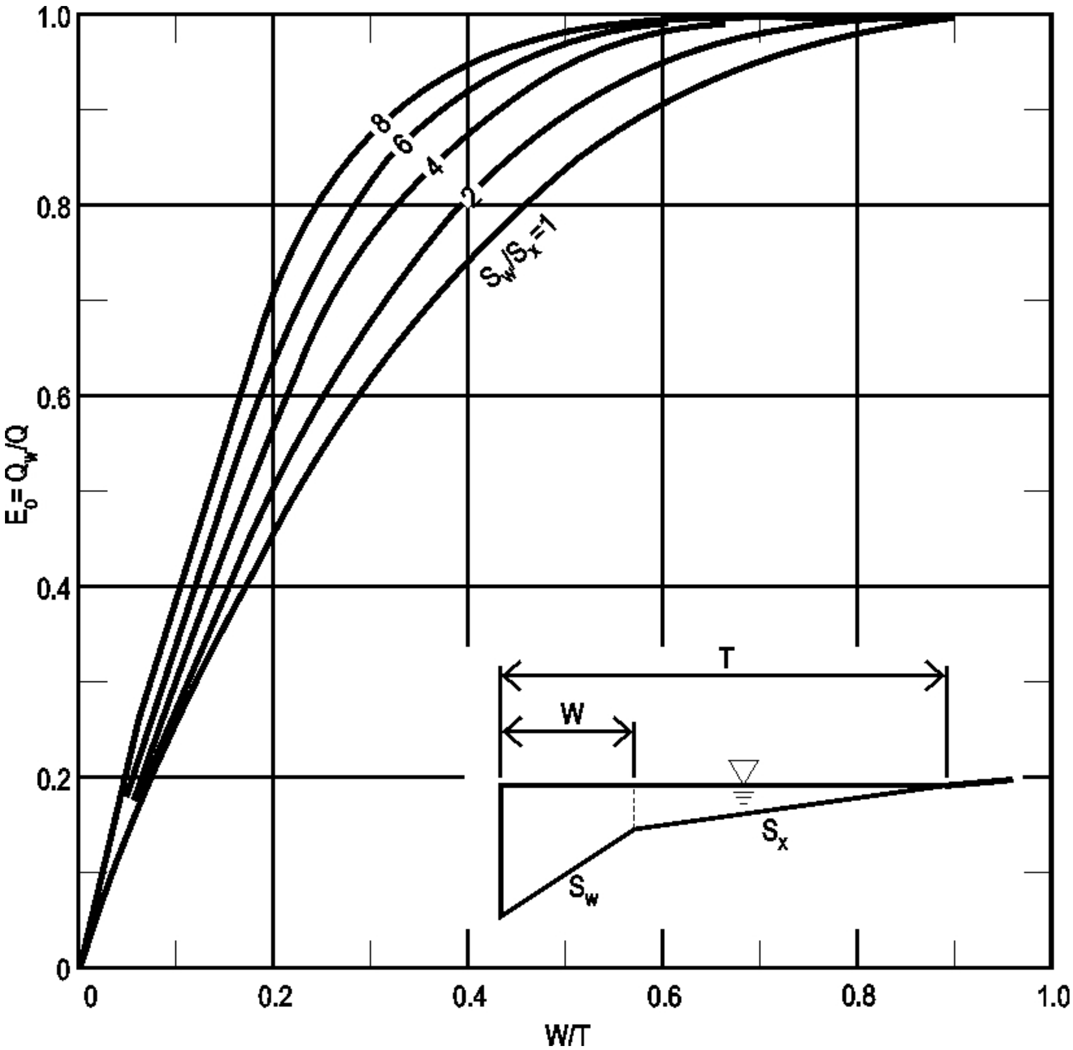
$$E_o = 1 / \left(1 + \frac{S_w / S_x}{\left[1 + \frac{S_w / S_x}{\frac{T}{W} - 1} \right]^{2.67} - 1} \right) \tag{3.4}$$

where:

E_o = Ratio of flow in the depressed section to total gutter flow
S_x = Pavement cross-slope, ft/ft
W = Width of gutter, ft
T = Width of flow, spread, ft
S_w = Cross-slope of a depressed gutter ($S_x + \frac{gutter\ depression}{W}$), ft/ft

(Equation (3.4), Reference: USDOT, FHWA, 1996, HEC-22, Equation 4-4)

FIGURE 3.6
RATIO OF FRONTAL FLOW TO TOTAL GUTTER FLOW
(USDOT, FHWA, 1984, HEC-12, CHART 4)



$$Q = \frac{Q_s}{(1-E_o)} \tag{3.5}$$

$$Q_w = Q - Q_s \tag{3.6}$$

where:

Q_w	=	Flow rate in depressed section of gutter, cfs
Q_s	=	Flow rate in paved area, cfs
Q	=	Total gutter flow rate, cfs

Curb-Opening Catch Basins

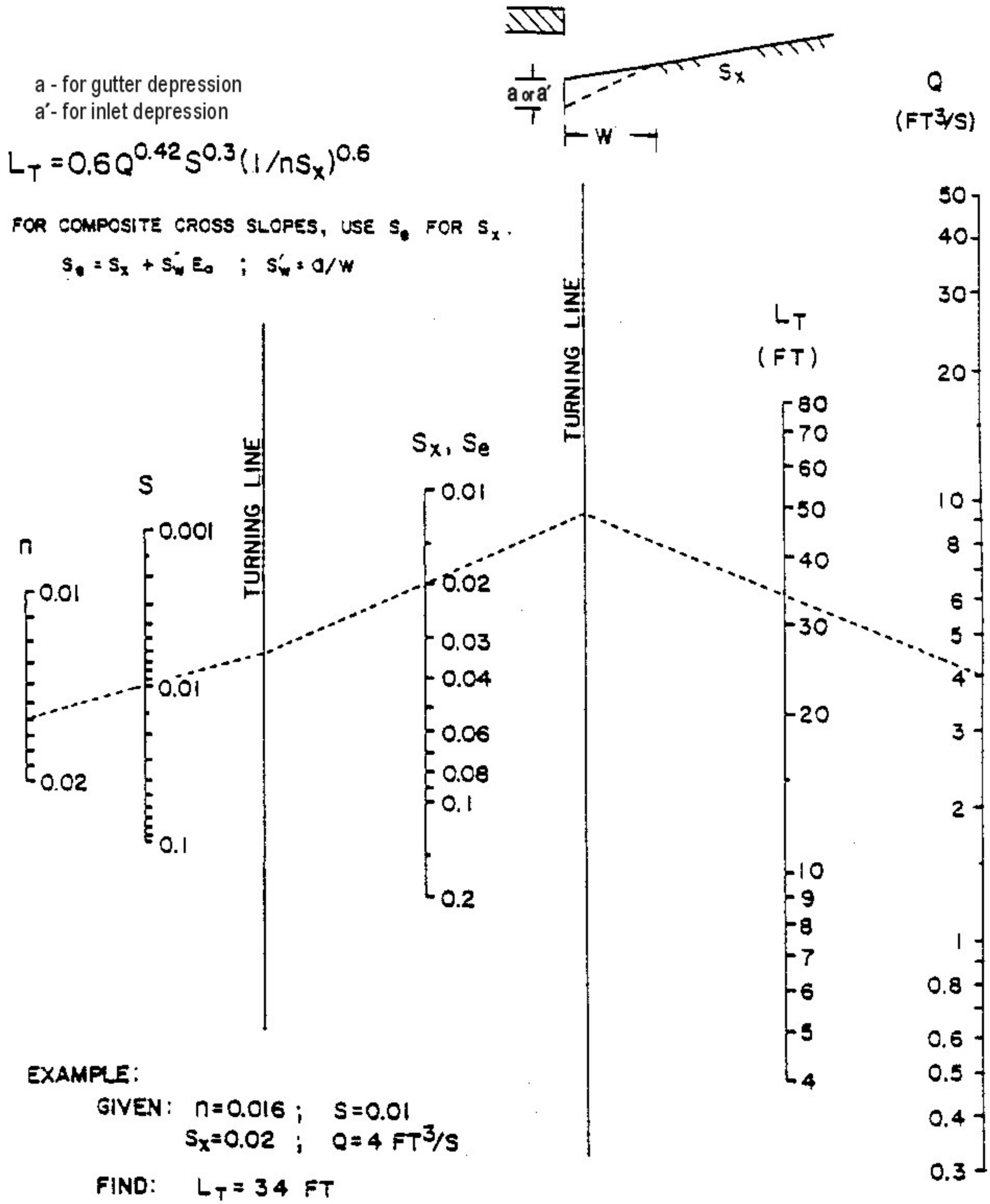
On-Grade - The length (L_t) of curb opening catch basin required for total interception of gutter flow on a pavement section with a straight cross slope is expressed as:

$$L_t = 0.6Q^{0.42}S^{0.3}\left(\frac{1}{nS_x}\right)^{0.6} \qquad (3.7)$$

where:

where:	Q	=	Total gutter flow rate, cfs
	S	=	Longitudinal slope, ft/ft
	S_x	=	Pavement cross-slope, ft/ft
	n	=	Manning’s roughness coefficient

FIGURE 3.7
CURB OPENING AND SLOTTED DRAIN INLET LENGTH FOR TOTAL INTERCEPTION
(USDOT, FHWA, 1984, HEC-12, CHART 9)



The efficiency (E) of curb-opening catch basins shorter than the length required for total interception is:

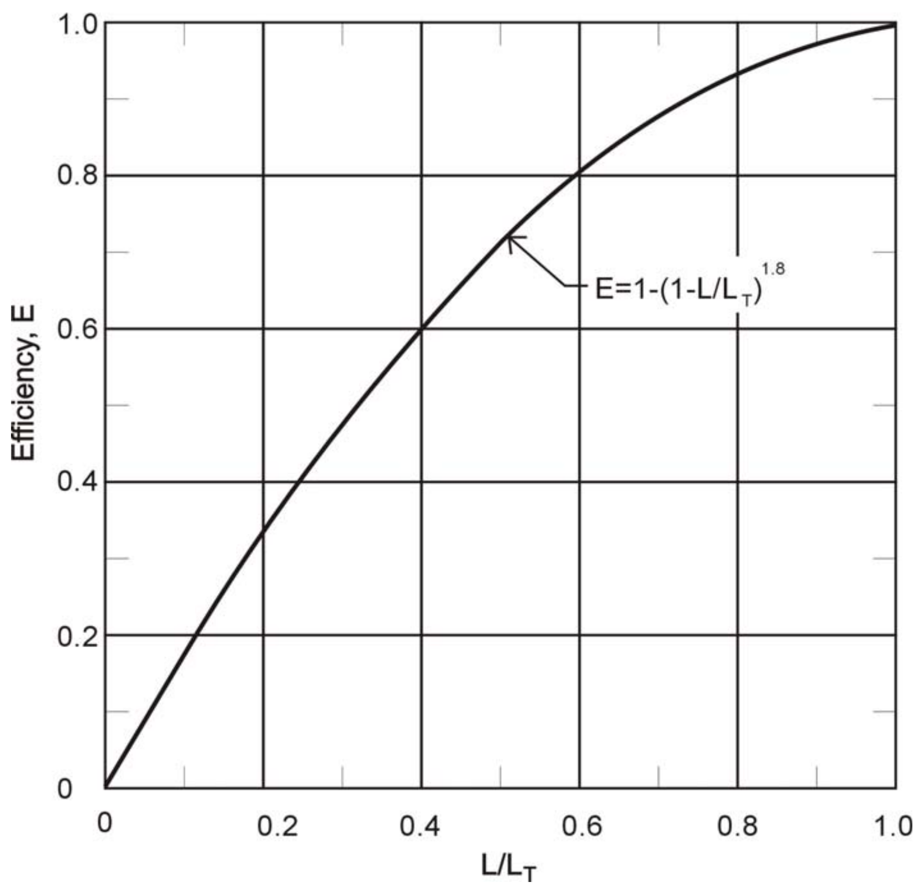
$$E = 1 - \left(1 - \frac{L}{L_t}\right)^{1.8} \tag{3.8}$$

where: L = Length of curb opening, grate or slot, ft

L_t = Curb opening length required to intercept 100% of the gutter flow, ft

Figure 3.8 provides a solution of Equation (3.8) and the equation is applicable with either straight cross slopes or compound cross slopes.

FIGURE 3.8
CURB OPENING AND SLOTTED DRAIN INLET INTERCEPTION EFFICIENCY
(USDOT, FHWA, 1984, HEC-12, CHART 10)



The length of catch basin required for total interception by depressed curb-opening catch basins or curb openings in depressed gutter sections can be found by using an equivalent cross slope, S_e. S_e can be calculated using Equation (3.9).

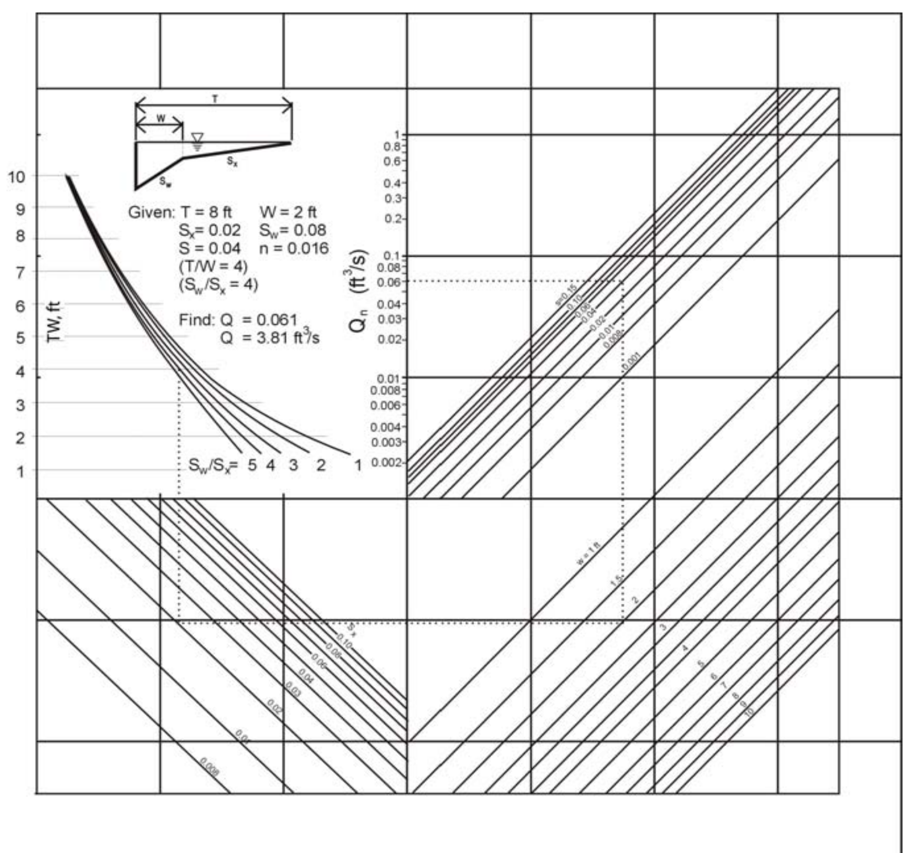
$$S_e = S_x + S'_w E_o \tag{3.9}$$

where:

S'_w	=	Cross slope of the gutter (at the inlet) measured from the cross slope of the pavement, ft/ft ($S'_w = a/12W$ see Figure 3.7)
E_o	=	Ratio of flow in the depressed section to total gutter flow
S_x	=	Pavement cross-slope, ft/ft

E_o is the ratio of flow in the depressed section to the total gutter flow, and S'_w is the cross slope of the gutter measured from the cross slope of the pavement, S_x. Figure 3.9 can be used to determine the spread, and then Figure 3.6 can be used to determine E_o.

FIGURE 3.9
FLOW IN COMPOSITE GUTTER SECTIONS
(USDOT, FHWA, 1984, HEC-12, CHART 5)



The length of curb-opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

Using the equivalent cross slope, S_e Equation (3.7) becomes:

$$L_t = 0.6Q^{0.42}S^{0.3}\left(\frac{1}{nS_e}\right)^{0.6} \tag{3.10}$$

Figure 3.7 and Figure 3.8 are applicable to depressed curb-opening catch basins using S_e rather than S_x.

Sumps - The capacity of a curb-opening catch basin in a sump depends on water depth at the curb, the curb opening length, and the height of the curb opening. The catch basin operates as a weir for depths of water up to the curb-opening height and as an orifice at depths greater than 1.4 times the opening height. At water depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

The weir location for a depressed curb-opening catch basin is at the edge of the gutter, and the effective weir length is dependent on the width of the depressed gutter and the length of the curb-opening. The weir location for a curb opening catch basin that is not depressed is at the lip of the curb-opening, and its length is equal to that of the curb-opening catch basin.

The equation for the interception capacity of a depressed curb opening-catch basin operating as a weir is:

$$Q_i = C_w(L + 1.8W)d^{1.5} \tag{3.11}$$

where:

where:	Q_i	=	Amount of street flow intercepted by inlet, cfs
	C_w	=	Weir coefficient = 2.3
	W	=	Width of grate or depressed gutter, ft
	d	=	Depth of flow, ft (measured from water surface to projected cross slope)
	L	=	Length of curb opening, or slot, ft

The weir equation is applicable to depths at the curb approximately equal to the height of the opening plus the depth of the depression. Thus, the limitation on the use of Equation (3.11) for a depressed curb opening catch basin is:

$$d < h + \frac{a'}{12} \tag{3.12}$$

where: h = Height of curb opening catch basin, curb opening orifice, or orifice throat width, ft

a = Gutter depression, inches

Experiments have not been conducted for curb opening catch basins with a continuously depressed gutter, but it is reasonable to expect that the effective weir length would be as great as that for a catch basin in a local depression. Use of Equation (3.11) will yield conservative estimates of the interception capacity.

The weir equation for curb opening catch basins without depression ($W = 0$) becomes:

$$Q_i = C_w L d^{15} \tag{3.13}$$

where: C_w = 3.0

d = Depth of flow, ft

L = Length of curb opening or slot, ft

The depth limitation for operation as a weir becomes: $d \leq h$

Curb opening catch basins operate as orifices at depths greater than approximately 1.4h. The interception capacity can be computed by Equation (3.14):

$$Q_i = C_o h L (2 g d_o)^{0.5} \tag{3.14}$$

where: C_o = Orifice coefficient = 0.67

g = Gravity, 32.2 ft/sec^2

d_o = Effective depth at the center of the curb opening orifice, ft

h = Height of curb opening catch basin, curb-opening orifice, or orifice throat, ft

L = Length of curb opening, ft

Equation (3.14) is applicable to depressed and undepressed curb opening catch basins and the depth at the catch basin includes any gutter depression.

Height of the orifice in Equation (3.14) assumes a vertical orifice opening. As illustrated in Figure 3.10, other orifice throat locations can change the effective depth on the orifice and the dimension $(d_i - h/2)$. A limited throat width could reduce the capacity of the curb-opening catch basin by causing the catch basin to go into orifice flow at depths less than the height of the opening.

FIGURE 3.10
CURB OPENING CATCH BASIN INLETS
(Modified from: USDOT, FHWA, 1984, HEC-12, Figure 21)

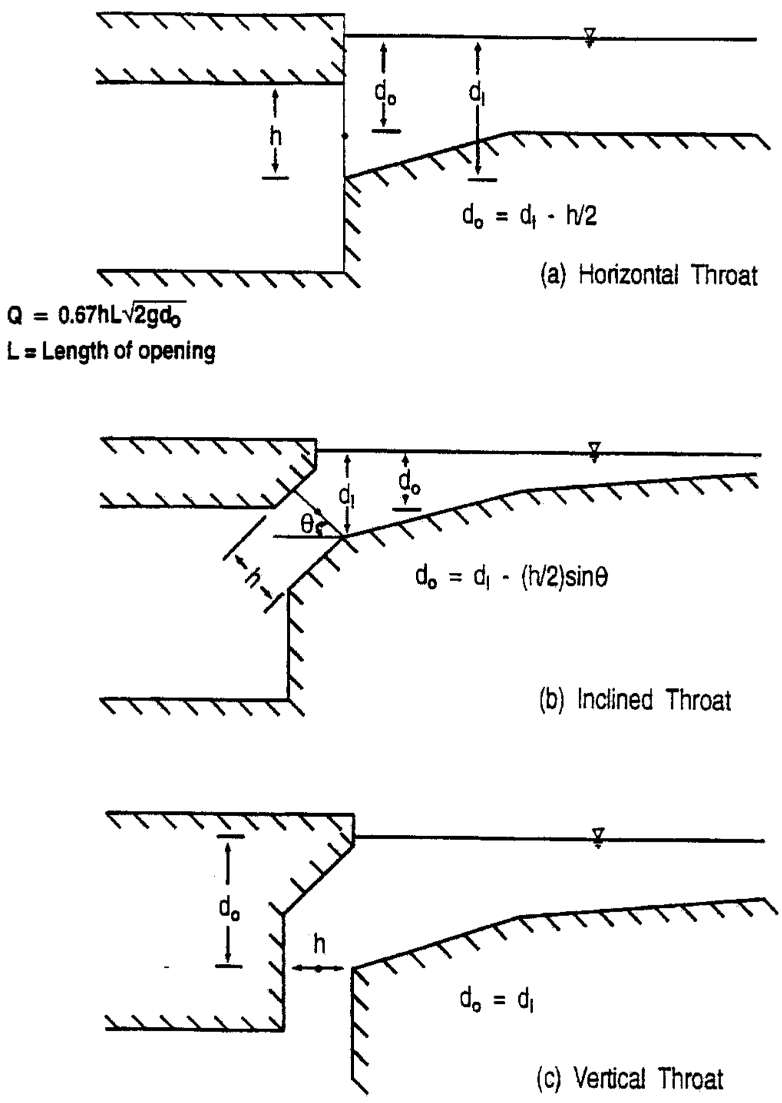


Figure 3.11 provides solutions for Equations 3.11 and 3.14 for depressed curb-opening catch basins, and Figure 3.12 provides solutions for Equations 3.13 and 3.14 for curb-opening catch

basins without depression. Figure 3.13 is provided for use for curb openings with inclined or vertical orifice throats.

FIGURE 3.11
DEPRESSED CURB OPENING INLET CAPACITY IN SUMP LOCATIONS
(USDOT, FHWA, 1984, HEC-12, Chart 12)

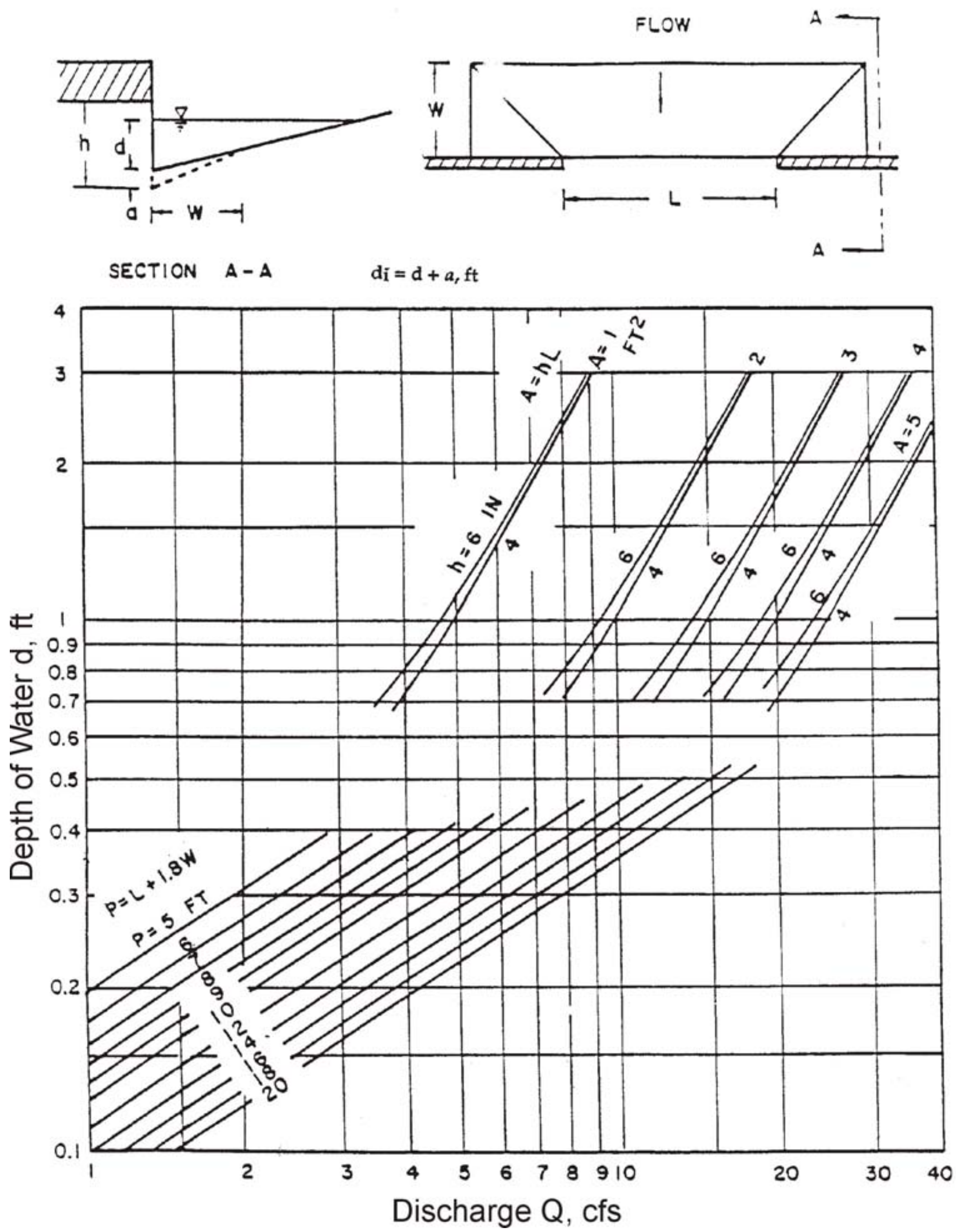


FIGURE 3.12
CURB OPENING INLET CAPACITY IN SUMP LOCATIONS
(USDOT, FHWA, 1984, HEC-12, Chart 13)

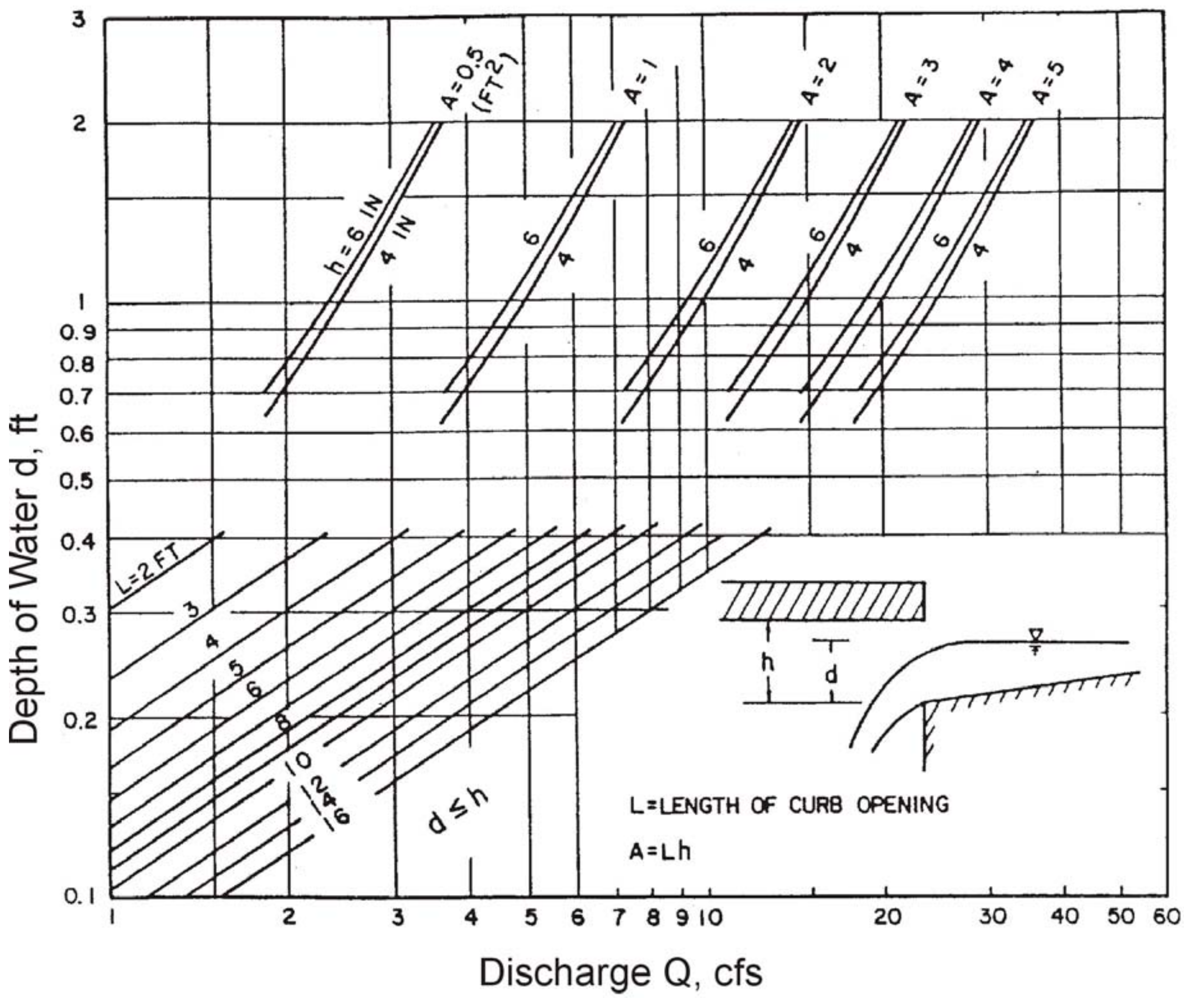
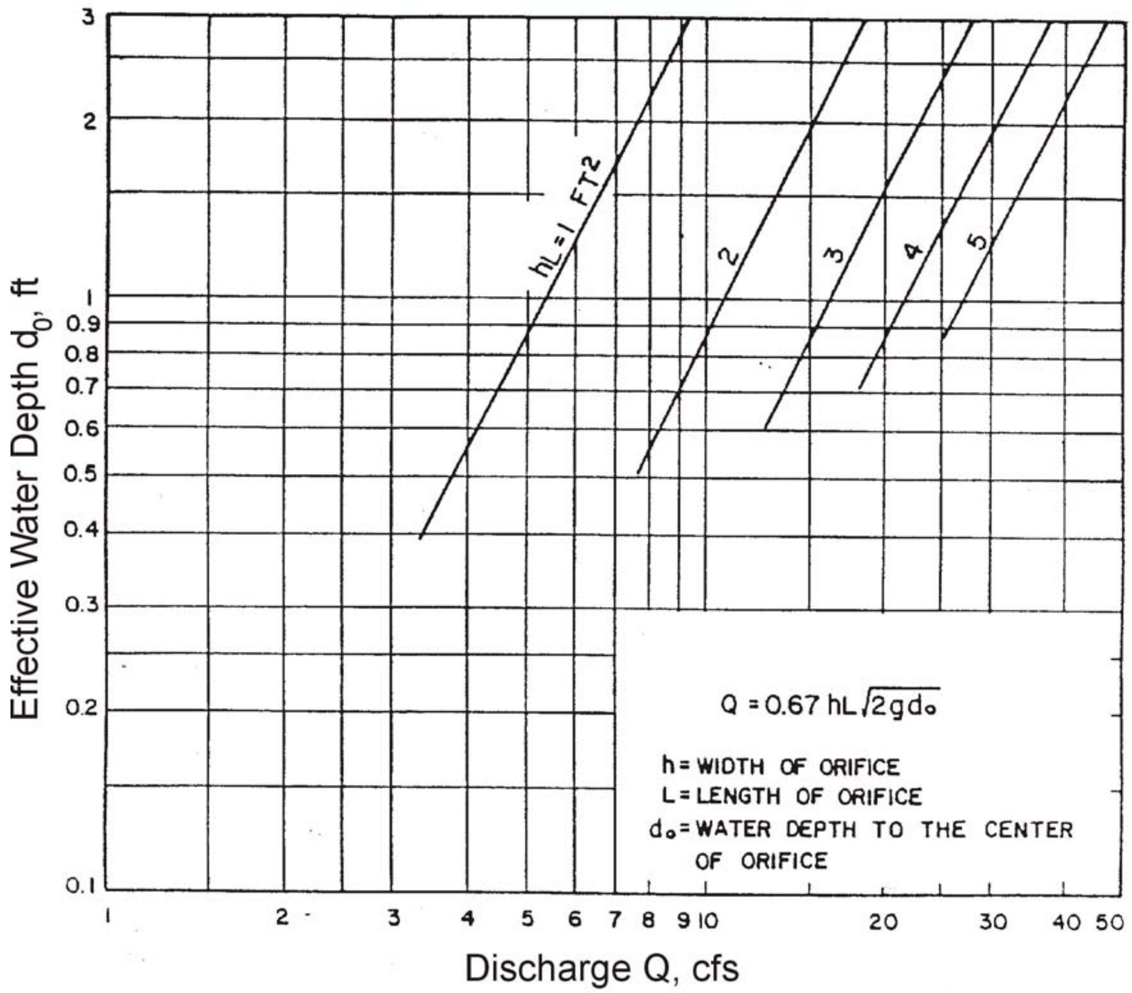


FIGURE 3.13
CURB OPENING INLET CAPACITY FOR INCLINED AND VERTICAL ORIFICE THROATS
(USDOT, FHWA, 1984, HEC-12, Chart 14)



Grated Catch Basins

On-Grade - Grated catch basins intercept all of the frontal flow until splash over (the velocity at which water begins to splash over the grate) is reached. At velocities greater than splash over, grate efficiency in intercepting frontal flow is diminished. Grates also intercept a portion of the flow along the length of the grate, or the side flow, dependent on the cross slope of the pavement, the length of the grate, and flow velocity.

The ratio of frontal flow to total gutter flow, E_o for a straight cross slope is:

$$E_o = \frac{Q_w}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67} \tag{3.15}$$

where:
- Q_w = Flow rate in width (W), cfs
- Q = Total flow, cfs
- W = Width of grate or gutter, ft
- T = Spread of flow on the pavement, ft

Figure 3.6 provides a graphical solution of E_o for either straight cross slopes or depressed gutter sections.

The ratio of side flow, (Q_s) to total gutter flow (Q) is:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o \tag{3.16}$$

where:
- Q_s = Flow rate outside of width (W), cfs
- Q_w = Flow rate in width of grate or gutter (W), cfs

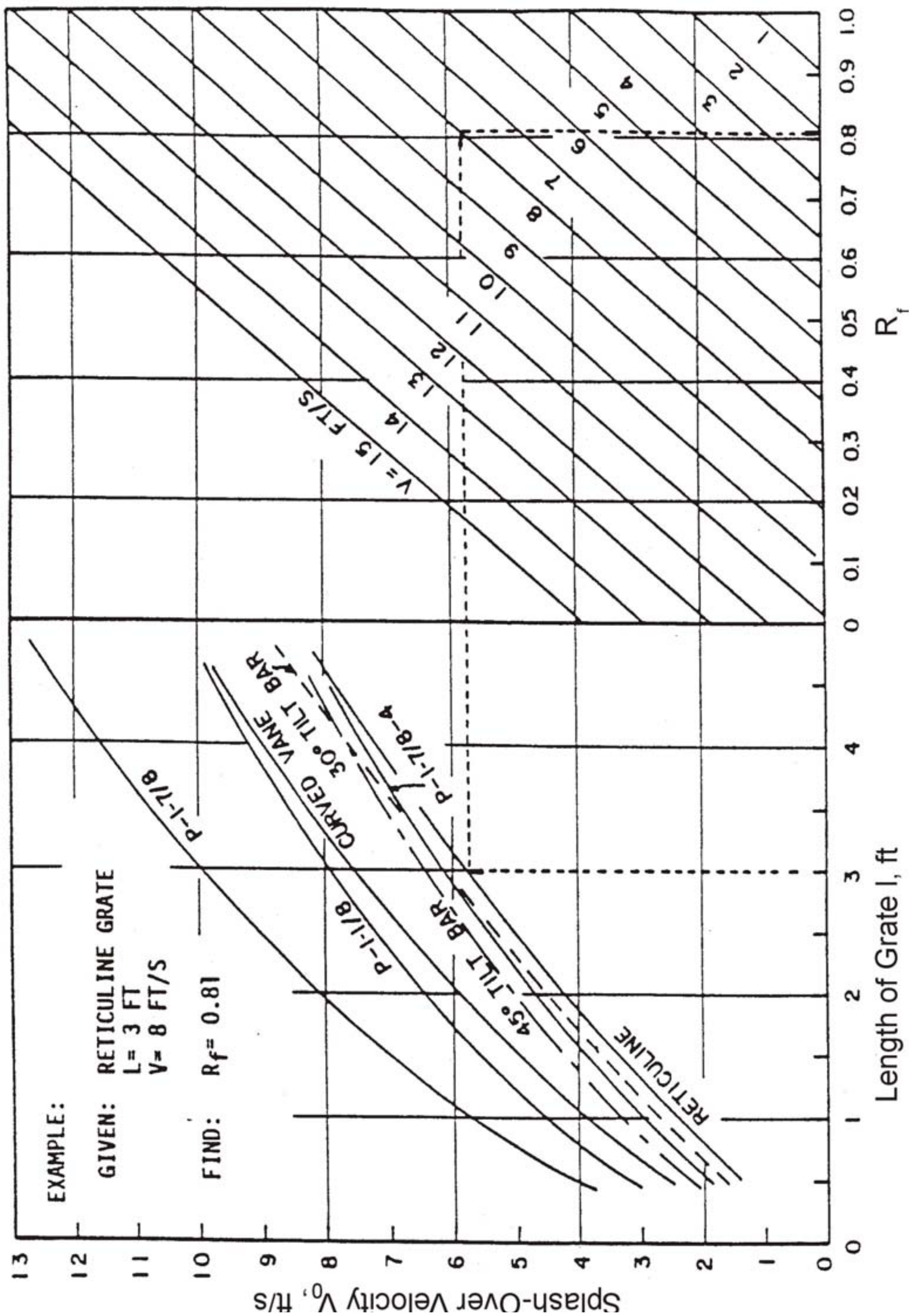
The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed:

$$R_f = 1 - 0.09(V - V_o) \tag{3.17}$$

where:
- R_f = Ratio of frontal flow intercepted to total frontal flow
- V = Velocity of flow in the gutter, ft/sec
- V_o = Gutter velocity where splash over first occurs, ft/sec

This ratio is equivalent to frontal flow interception efficiency. Figure 3.14 provides a solution of Equation (3.17) which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 3.14 is total gutter flow divided by the area of flow.

FIGURE 3.14
GRATE INLET FRONTAL FLOW INTERCEPTION EFFICIENCY
(USDOT, FHWA, 1984, HEC-12, Chart 7)



The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed:

$$R_s = \frac{1}{1 + \frac{0.15V^{1.8}}{S_x L^{2.3}}} \tag{3.18}$$

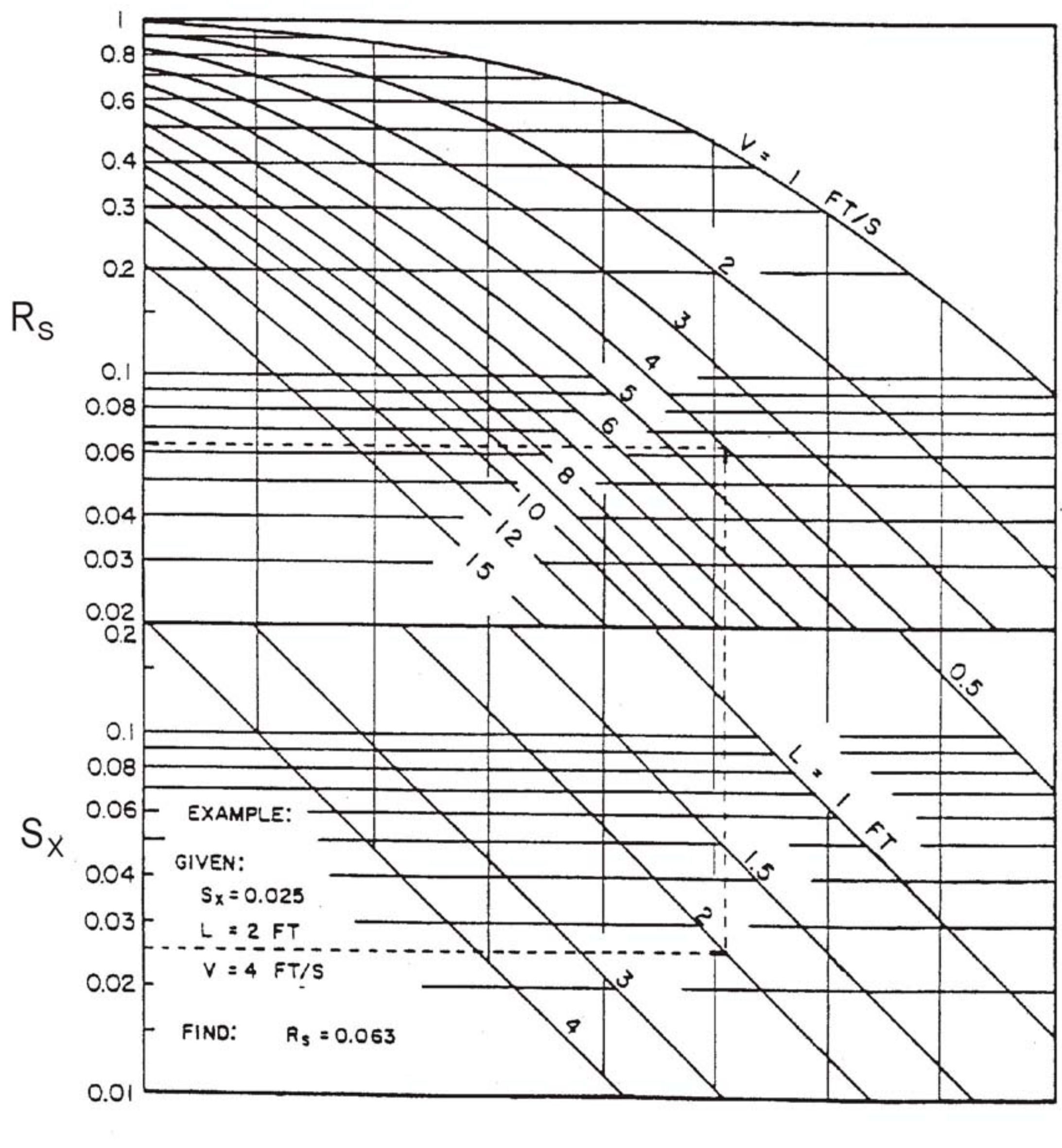
where:

	S_x	=	Pavement cross slope, ft/ft
	L	=	Length of grate, ft
	V	=	Velocity of flow in the gutter, ft/sec

Figure 3.15 provides a solution of Equation (3.18).

A deficiency in developing empirical equations and charts from experimental data is evident in Figure 3.15. The fact that a grate will intercept all or almost all of the side flow where the velocity is low and the spread only slightly exceeds the grate width is not reflected in the figure. Error due to this deficiency is very small. In fact, where velocities are high, side flow interception can be neglected entirely without significant error.

FIGURE 3.15
GRATE INLET SIDE FLOW INTERCEPTION EFFICIENCY
(USDOT, FHWA, 1984, HEC-12, Chart 8)



The efficiency, E, of a grate is:

$$E = R_f E_o + R_s(1 - E_o) \tag{3.19}$$

The first term on the right side of Equation (3.19) is the ratio of intercepted frontal flow to total gutter flow, and the second term is the ratio of intercepted side flow to total side flow. The second term is insignificant with high velocities and short grates.

The interception capacity (Q_i) of a grate catch basin on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q[R_f E_0 + R_s(1 - E_0)] \tag{3.20}$$

Sump - The efficiency of catch basins in passing debris is critical in sump locations because all runoff which enters the sump must be passed through the catch basin. Total or partial clogging of catch basins in these locations can result in hazardous ponding conditions. Grate catch basins alone are not recommended for use in sump locations because of the tendencies of grates to become clogged. Combination catch basins or curb-opening catch basins are recommended for use in these locations.

A grate catch basin in a sump location operates as a weir to depths dependent on the bar configuration and size of the grate and as an orifice at greater depths. Grates of larger dimension and grates with more open area, that is, with less space occupied by lateral and longitudinal bars, will operate as weirs to greater depths than smaller grates or grates with less open area.

The capacity of grate catch basins operating as weirs is:

$$Q_i = C_w P d^{1.5} \tag{3.21}$$

where:

C_w = Weir coefficient = 3.0

P = Perimeter of the grate, disregarding bars and side against curb, ft

d = Depth of flow at curb, ft

The capacity of a grate catch basin operating as an orifice is:

$$Q_i = C_o A_g (2gd)^{0.5} \tag{3.22}$$

where:

C_o = Orifice coefficient = 0.67

A_g = Clear opening area of the grate, sq ft

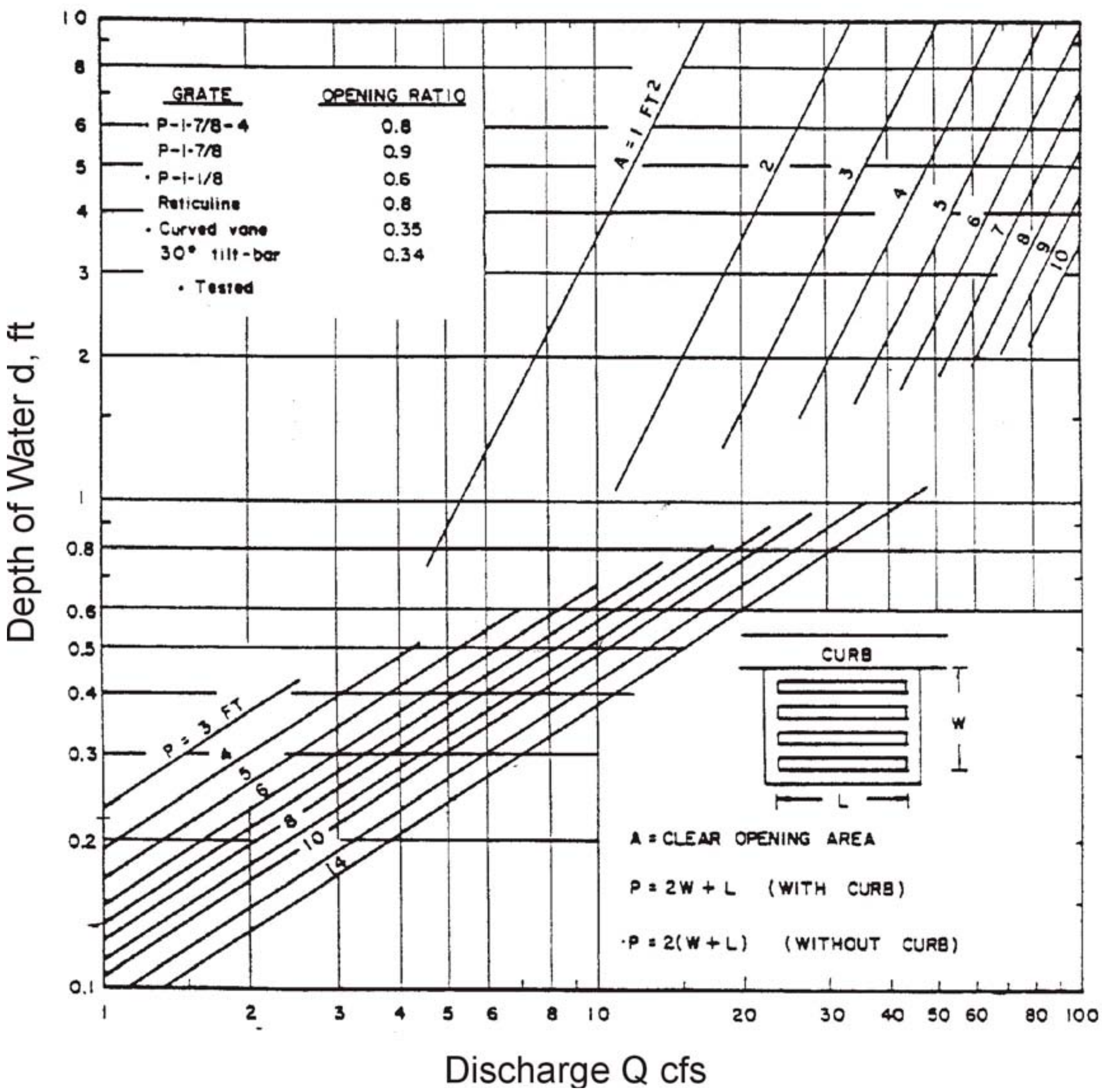
d = Depth of flow at curb, ft

g = Gravity, 32.2 ft/sec^2

Use of Equation (3.22) requires the clear opening area of the grate. Tests of three grates for the Federal Highway Administration showed that for flat bar grates, such as *P-1-7/8-4* and *P-1-1/8* grates, the clear opening is equal to the total area of the grate less the area occupied by longitudinal and lateral bars.

Figure 3.16 is a plot of Equation (3.21) and Equation (3.22) for various grate sizes. The effects of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either the weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used.

FIGURE 3.16
GRATE INLET CAPACITY IN SUMP CONDITIONS
(USDOT, FHWA, 1984, HEC-12, Chart 11)



Combination Catch Basins

On-Grade - The interception capacity of a combination catch basin consisting of a curb opening and grate placed side-by is not appreciably greater than that of the grate opening alone. Capacity is computed by neglecting the curb opening. A combination catch basin is sometimes used with the curb opening or part of the curb opening placed upstream of the grate. A combination catch basin with a curb opening extending upstream of the grate has an interception capacity equal to the sum of the grated catch basin and of the portion of the curb opening inlet upstream of the grate. The frontal flow and thus the interception capacity of the grate is reduced by the flow intercepted by the curb opening.

Sump - Combination catch basins consisting of a grate and a curb opening are considered advisable for use in sumps where hazardous ponding can occur. The interception capacity of the combination catch basin is essentially equal to that of a grate alone in weir flow unless the grate opening becomes clogged. In orifice flow, the capacity is equal to the capacity of the grate plus the capacity of the curb opening.

Equation (3.21) or Figure 3.16 can be used for weir flow in combination catch basins in sump locations. Assuming complete clogging of the grate, Equation (3.11), Equation (3.13), and Equation (3.14), or Figure 3.11, Figure 3.12 and Figure 3.13 for curb-opening catch basins are applicable.

Where depth at the curb is such that orifice flow occurs, the interception capacity of the catch basin is computed by adding Equation (3.22) and Equation (3.14):

$$Q_i = 0.67A_g(2gd)^{0.5} + 0.67hL(2gd_o)^{0.5} \tag{3.23}$$

where:

Q_i	=	Amount of street flow intercepted by inlet, cfs
A_g	=	Clear opening area of the grate, sq ft
g	=	Gravity, 32.2 ft/sec^2
d	=	Depth of flow at curb, ft
h	=	Height of curb opening portion of catch basin, curb-opening orifice or orifice throat, ft
L	=	Length of curb opening, ft
d_o	=	Effective depth at the center of the curb opening orifice, ft

Trial and error solutions are necessary for depth at the curb for a given flow rate using Figure 3.11, Figure 3.12, Figure 3.13, or Figure 3.16 for orifice flow.

Slotted Drain Catch Basins

On-Grade - Wide experience with the debris handling capabilities of slotted drain catch basins is not available. Deposition in the pipe is the problem most commonly encountered; however, the catch basin is accessible for cleaning with a high pressure water jet.

Flow interception by slotted drain catch basins and curb-opening catch basins is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement. Analysis of data from the HEC-12 tests of slotted drain catch basins with slot widths greater than or equal to 1.75 inches indicates that the length of the slotted drain catch basin required for total interception can be computed using Equation (3.7). Figure 3.7 is therefore applicable for both curb-opening catch basins and slotted drain catch basins. Similarly, Equation (3.8) is also applicable to slotted drain catch basins and Figure 3.8 can be used to obtain the catch basin efficiency for the selected length of the catch basin.

Using Figure 3.7 and Figure 3.8 for slotted drain catch basins is the same as using them for curb-opening catch basins. It should be noted, however, that it is much less expensive to add length to a slotted drain catch basin to increase interception capacity than it is to add length to a curb-opening catch basin.

Sump - Slotted drain catch basins in sump locations perform as weirs to depths of about 0.2 ft, dependent on slot width and length. At depths greater than about 0.4 ft, they perform as orifices. Between these depths, flow is in a transition stage. The interception capacity of a slotted drain catch basin operating as an orifice can be computed by:

$$Q_i = 0.8LW(2gd)^{0.5} \tag{3.24}$$

where:

where:	Q_i	=	Amount of street flow intercepted by slotted inlet, cfs
	L	=	Length of slotted inlet, ft
	W	=	Width of slot, ft
	d	=	Depth of water at slot, $d \geq 0.4$ ft
	g	=	Gravity, 32.2 ft/sec^2

Equation (3.24) becomes:

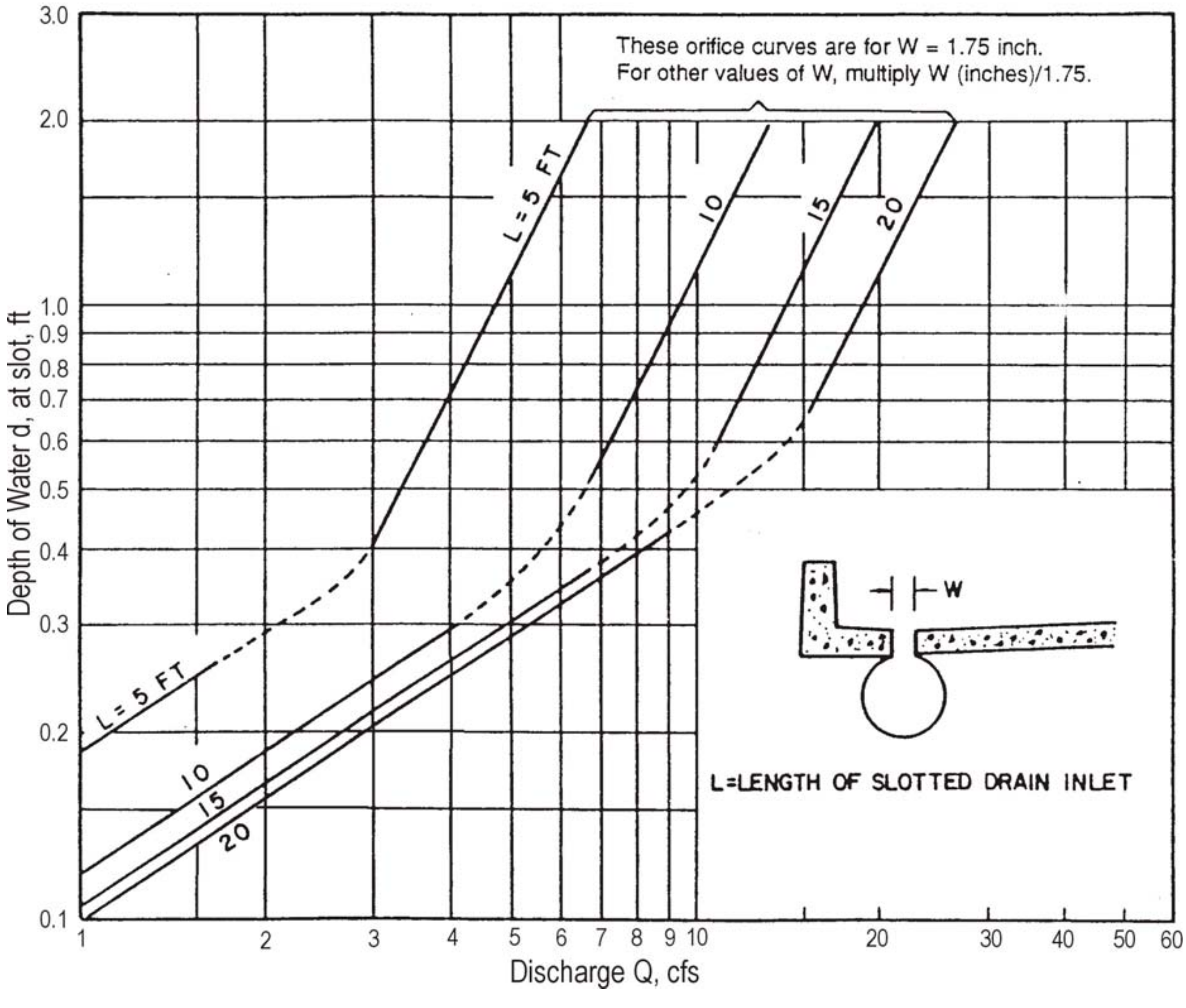
$$Q_i = 0.94Ld^{0.5} \tag{3.25}$$

when: W = 0.15 ft (1.75 inches)

The interception capacity of slotted drain catch basins at depths between 0.2 and 0.4 feet can be computed by using the orifice equation. The orifice coefficient varies with depth, slot width, and the length of the slotted drain catch basin.

Figure 3.17 provides the solutions for weir flow, transition flow and orifice flow.

FIGURE 3.17
SLOTTED DRAIN INLET CAPACITY IN SUMP CONDITION
(USDOT, FHWA, 1984, HEC-12, CHART 15)



Guidelines

Inlets in sumps are generally much more efficient and economically justifiable than inlets on a continuous grade, so the street designer should strive to adjust grades, when practical, to provide sumps for inlets. A sump is created at each intersection of a side street with a major street where the crown of the side street is extended at least to the quarter point of the major street. This provides an efficient pick up point. However, on the downstream side of the side street, incoming storm drainage will tend to flow on down the major street and bypass a catch basin. Therefore, where conditions permit, the side street may be depressed for a short distance upstream from the curb return to provide a second efficient pick up point, if the side street is bringing a large volume of runoff. Another alternative is multiple catch basins to intercept the excessive runoff. The most economical alternative shall be used.

To account for a potential reduction of inflow capacity due to clogging, the design of the inlet should include a factor of safety. Here the area or length required is adjusted by clogging or reduction factors as set forth by the standards used by the jurisdictional entity. For Maricopa County, clogging or reduction factors are set forth in the *Policies and Standards Manual*.

3.4 APPLICATION

This section offers design procedures for street drainage and presents design examples. Equations presented in this chapter shall be used for design purposes. Nomographs presented in this chapter can be used for design concept evaluations or initial evaluations.

3.4.1 Design Procedures

Design procedures for street drainage on a continuous grade are as follows:

1. For a given longitudinal street slope and cross slope at a location determine the flow rate that would provide a flow spread that is equal to the allowable spread.

2. Determine if the drainage area draining to the location used in Step 1 will generate the discharge determined in Step 1. If not choose a different location for Step 1. Continue the iterative process until the drainage area flow rate is consistent with the allowable spread flow rate.

3. Determine if there are conflicts with the placement of a catch basin at this location. Conflicts could be but are not limited to, side streets, driveways, utilities that would be costly to relocate, etc. Should there be conflicts, move the catch basin location upstream.

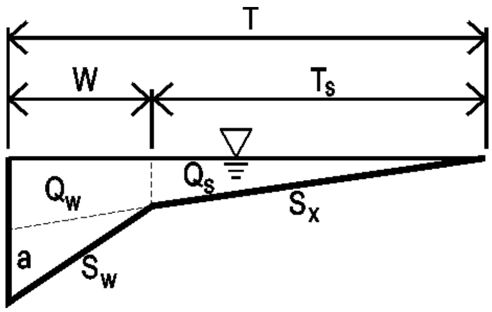
4. Size a catch basin to intercept the calculated flow. Determine the efficiency of the catch basin and determine the flow rate, if any, that will by pass the catch basin.

5. Choose a location downstream in which the drainage area contributing to the location will generate a flow rate that when added to the by pass flow rate determined in Step 4 is equal to the flow rate that would generate a spread that is equal to the allowable spread.

6. Continue steps 3 through 5 to termination of the project. Design examples for these procedures are shown in Section 3.4.2.

3.4.2 Design Examples

Example 1



Determine the total discharge (Q) for the composite gutter section.

Given:	Allowable spread	T	=	12 ft
	Cross-slope	S_x	=	0.02 ft/ft
	Gutter depression	a	=	1 inch or 0.0833 ft
	Longitudinal Slope	S	=	0.008 ft/ft
	Gutter Width	W	=	1.42 ft
	Manning's roughness value	n	=	0.015

NOTE: For MAG, (1979) Details, *a* is typically 0.37 inches.

Step 1:

Determine the flow spread (T_s) for the pavement section.

$$T = W + T_s$$

$$T_s = T - W = 12\ ft - 1.42 = 10.58\ ft$$

Step 2:

Determine the discharge (Q_s) in the paved section using Equation (3.3) and/or Figure 3.4.

$$Q_s = \left(\frac{0.56}{n}\right) S_x^{1.67} S^{0.5} T_s^{2.67} \qquad (3.3)$$

$$Q_s = \left(\frac{0.56}{0.015}\right) \times 0.02^{1.67} \times 0.008^{0.5} \times 10.58^{2.67} = 2.64\ cfs$$

Use Figure 3.4 to determine the discharge (Q_s). To do this, connect the values for S and S_x with a straight line that intersects the turning line. Now draw a straight line from the turning line through the value T_s to the discharge line. Read the value $Q_s n$.

$$Q_s n = 0.04$$

$$Q_s = \frac{0.04}{n} = \frac{0.04}{0.015} = 2.67\ cfs$$

Step 3:

Determine the total discharge (Q) using Equations 3.4 and 3.5.

$$E_o = 1/\left\{1 + \frac{S_w/S_x}{\left[1 + \frac{S_w/S_x}{\frac{T}{W} - 1}\right]^{2.67} - 1}\right\} \qquad (3.4)$$

To solve Equation (3.4), determine S_w, S_w/S_x and T/W.

$$S_w = \frac{a}{W} + S_x = \frac{0.0833\,ft}{1.42\,ft} + 0.02\frac{ft}{ft} = 0.0787\frac{ft}{ft}$$

$$\frac{S_w}{S_x} = \frac{0.0787}{0.02} = 3.93$$

$$\frac{T}{W} = \frac{12}{1.42} = 8.45$$

By substitution:

$$E_o = 1/\left\{1 + \frac{3.93}{\left[1 + \frac{3.93}{8.45 - 1}\right]^{2.67} - 1}\right\} = 0.348$$

Determine the total discharge using Figure 3.5 and Equation (3.5). To determine the value E_o using Figure 3.6, begin with the W/T value and go vertically up until you intersect the S_w/S_x value. Project horizontally to the E_o axis and read value.

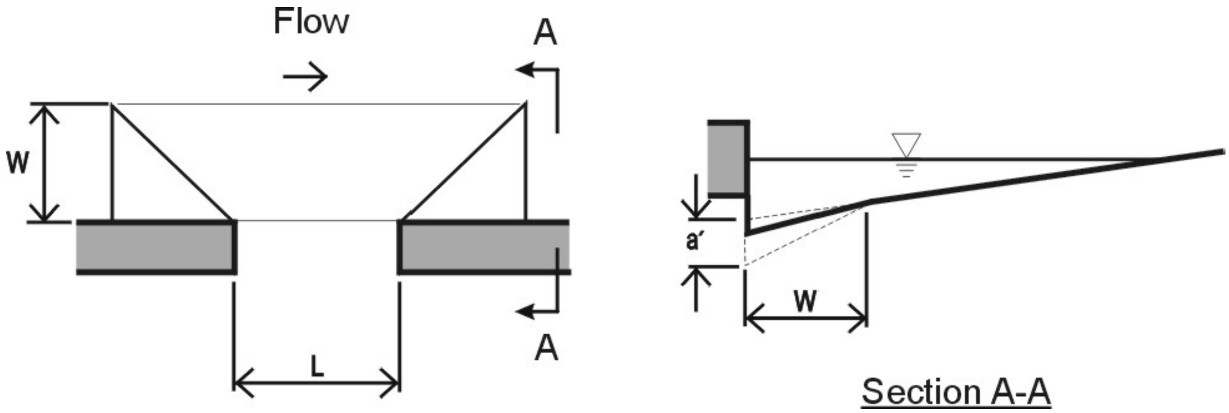
$$E_o = 0.33$$

Use Equation (3.5) to solve for the total discharge using Q_s and E_o from Figure 3.4 and Figure 3.6.

$$Q = \frac{Q_s}{(1 - E_o)} = \frac{2.67}{1.0 - 0.33} = 3.99\ cfs \ \text{ say } 4\ cfs \qquad (3.5)$$

Example 2

Determine the length of a curb-opening inlet on grade for the interception of the gutter flow determined in Example 1.



Given:	Total gutter flow	Q	=	4 cfs
	Cross-slope	S_x	=	0.02 ft/ft
	Gutter depression	a	=	1 inch or 0.0833 ft
	Longitudinal Slope	S	=	0.008 ft/ft
	Gutter Width	W	=	1.42 ft
	Manning's roughness value	n	=	0.015
	Clogging factor		=	1.25 x required length
	Gutter depression (at inlet)	a'	=	2 inches or 0.167 ft

NOTE: For MAG, (1979) Details, *a'* is a minimum of 2.37 inches.

Step 1:

Determine the equivalent cross-slope using Equation (3.9). Note, use gutter depression at inlet.

$$S_e = S_x + S'_w E_o \qquad (3.9)$$

$$S'_w = \frac{a'}{W} = \frac{0.167ft}{1.42ft} = 0.12\frac{ft}{ft}$$

$E_o = 0.35$ from Example 1

$$S_e = 0.02\frac{ft}{ft} + 0.12\frac{ft}{ft} \times 0.35 = 0.062\frac{ft}{ft}$$

Step 2:

Using Equation (3.10) solve for length.

$$L_t = 0.6Q^{0.42}S^{0.3}\left(\frac{1}{nS_e}\right)^{0.6} \qquad (3.10)$$

$$L_t = 0.6 \times 4^{0.42} \times 0.008^{0.3} \times \left(\frac{1}{0.015 \times 0.062}\right)^{0.6} = 16.6\ ft$$

To use Figure 3.7 to determine the length of curb opening, first draw a straight line through the n and S values to intersect the turning lane. Then draw a straight line from the turning line through the S_e value intersecting the second turning line. From the second turning line draw a straight line to the Q value. Read the value L_t.

$$L_t = 17\ feet$$

Step 3:

Determine length with a clogging factor of 1.25.

L_t (with clogging factor) $= L_t \times 1.25$

$$= 16.6 \times 1.25 = 20.75$$

Try a curb opening inlet catch basin with a 10-foot wing. Total Length, $L = 13\ ft$.

Step 4:

The curb opening provided is 13 feet; therefore, determine the catch basin efficiency (E), the flow intercepted (Q_i) and the bypass flow.

Use Equation (3.8) to determine the efficiency of the catch basin provided.

$$E = 1 - \left(1 - \frac{L}{L_t}\right)^{1.8} \qquad (3.8)$$

$$E = 1 - \left[1 - \frac{13}{20.75}\right]^{1.8} = 0.83$$

$$Q_i = Q \times E$$

$$Q_i = 4\ cfs \times 0.83 = 3.3\ cfs$$

$$Q_{bypass} = Q - Q_i$$

$$= 4\ cfs - 3.3\ cfs$$

$$= 0.7\ cfs$$

To use Figure 3.8 to determine the efficiency, begin with the L/L_t value and go vertically up to the efficiency curve (E) and then project horizontally to the efficiency value.

$$E = 0.83$$

Example 3

Determine the interception capacity of a single grated inlet on grade for the flow rate determined in Example 1.

Given:	Total gutter flow	Q	=	4 cfs
	Flow in pavement section	Q_s	=	2.64 cfs (from Example 1)
	Cross slope	S_x	=	0.02 ft/ft
	Gutter depression	a	=	1 in or 0.0833 ft
	Cross slope of a depressed gutter ft/ft	S_w	=	0.0788 ft/ft (from Example 1)
	Longitudinal slope	S	=	0.008 ft/ft
	Gutter width	W	=	1.42 ft
	Manning's Roughness Coefficient	n	=	0.015
	Ratio of flow in the depressed section to total gutter flow	E_o	=	0.35 (from Example 1)
	Allowable spread	T	=	12 ft

Note: Assume grate is equivalent to the *P-1-7/8-4* grate presented in HEC-22

Step 1:

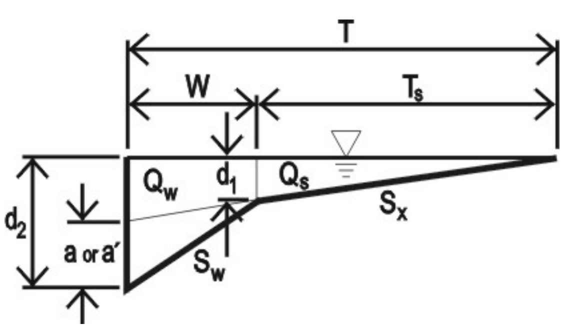
Determine flow rate Q_w in gutter width:

$$Q_w = Q - Q_s$$

$$= 4 - 2.64 = 1.36\ cfs$$

Step 2:

Determine velocity of flow in gutter width; W.



Determine d_1 and d_2 in figure above

$$d_1 = (T - W)S_x = (12\,ft - 1.42\,ft) \times 0.02\frac{ft}{ft} = 0.21\,ft$$

$$d_2 = TS_x + a = 12\,ft \times 0.02\frac{ft}{ft} + 0.0833\,ft = 0.32\,ft$$

Determine flow area of Q_w.

$$\text{Flow area} = d_1 \times W + \left(\frac{(d_2 - d_1) \times W}{2}\right)$$

$$= (0.21\,ft \times 1.42\,ft) + \left(\frac{(0.32\,ft - 0.21\,ft) \times 1.42\,ft}{2}\right)$$

$$= 0.376\,sq\,ft$$

Use $Q_w = VA_w$ to determine velocity

$$\frac{Q_w}{A_w} = V = \frac{1.36\,cfs}{0.376\,sq\,ft} = 3.6\,fps$$

Step 3:

Determine splash over velocity (V_o) from Figure 3.14.

Length of grate = 3 feet, extend vertically from the length of grate value a line to the *P-1-7/8-4* curve, then extend a line horizontally to the splash-over velocity axis, read value.

$$V_o = 6.1\,fps$$

Step 4:

Using Equation (3.17) or Figure 3.14 determine the ratio of frontal flow intercepted to total frontal flow.

$$R_f = 1 - 0.09(V - V_o) = 1 - 0.09(3.6\,fps - 6.1\,fps) \qquad (3.17)$$

$$= 1.22\ say\ 1.0\ or\ 100\%$$

With a clogging factor the width of opening perpendicular to flow is 0.5 times the actual width of the grate. Therefore R_f actual is equal to R_f with clogging $R_f \times 0.5 = 0.5$.

To use Figure 3.14 to determine R_f, extend a line vertically from the length of grate value to the *P-1-7/8-4* curve, then extend a line horizontally to the diagonal V line to the value determined in Step 2 and then vertically down to the R_f axis, read value. Maximum R_f value is equal to 1.

Step 5:

Using Equation (3.18) or Figure 3.15 determine the ratio of side flow intercepted to total side flow, applying a 1.25 clogging factor to length of grate, L.

$$R_s = \frac{1}{1 + \frac{0.15V^{1.8}}{S_x\left(\frac{L}{1.25}\right)^{2.3}}} = \frac{1}{1 + \frac{0.15(3.6)^{1.8}}{(0.02)\left(\frac{3.0}{1.25}\right)^{2.3}}} = 0.09 \qquad (3.18)$$

To use Figure 3.15 to determine R_s, extend a line horizontally from the S_x value to the diagonal L line with the L value adjusted for clogging, extend the line vertically to the diagonal V line with the V value determined in Step 2, then horizontally extend a line to the R_s axis and read the value.

$$R_s = 0.09$$

Step 6:

Using Equation (3.19) determine the efficiency of the grate.

$$E = R_f E_o + R_s(1 - E_o) \qquad (3.19)$$

$$= 0.5 \times 0.35 + 0.09 \times (1 - 0.35)$$

$$= 0.234$$

Step 7:

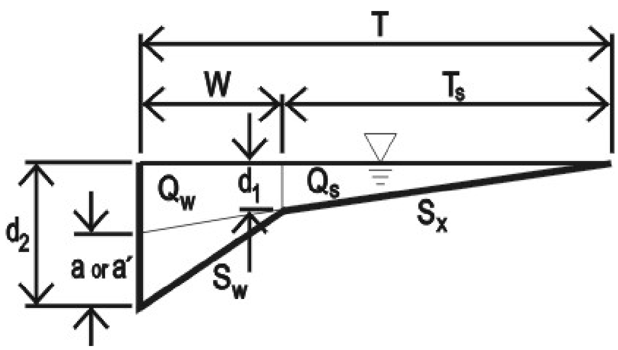
Determine flow rate Q_i intercepted

$$Q_i = (Q)(E) = 4 \times 0.234 = 0.94\ cfs$$

Example 4

Determine the capacity of a combination curb opening inlet for the flow rate determined in Example 1.

Given:	Total gutter flow	Q	=	4 cfs
	Cross slope	S_x	=	0.02 ft/ft
	Gutter depression	a	=	1 in or 0.0833 ft
	Longitudinal slope	S	=	0.008 ft/ft
	Gutter width	W	=	1.42 ft
	Manning's Roughness Coefficient	n	=	0.015
	Total curb opening length		=	14 ft
	Curb opening length upstream of grate		=	11 ft
	Grate length		=	3 ft
	Gutter depression at inlet	a'	=	2 inches or 0.167 ft



Step 1:

Compute the interception capacity (Q_{ic}) of the curb opening upstream of the grate.

From Example 2: $L_t = 16.6$ with clogging factor $L_t = 16.6 \times 1.25 = 20.75$

Use Equation (3.8) to determine efficiency of curb opening.

$$E = 1 - \left[1 - \frac{L}{L_t}\right]^{1.8} \quad (3.8)$$

$$E = 1 - \left[1 - \frac{11}{20.75}\right]^{1.8} = 0.74$$

$$Q_{ic} = Q \times E$$

$$Q_{ic} = 4\ cfs \times 0.74 = 2.97\ cfs\ say\ 3\ cfs$$

Step 2:

Determine interception capacity (Q_{ig}) of the grate.

Flow to grate $Q_g = Q - Q_{ic} = 4\ cfs - 3.0\ cfs - 1.0\ cfs$

Step 2.1:

By assuming the flow spread T_s calculate the discharge Q_s in the paved section adjacent to grate using the procedure listed in Example 1 Step 2. This is an iterative process.

Assume: $Ts = 4.2\ ft$

$$Q_s = 0.22\ cfs$$

Step 2.2:

Determine the total discharge following procedures listed in Example 1 Step 3.

Note: Use gutter depression-value at inlet.

$$S_w = \frac{0.167}{1.42} + 0.02 = 0.138\frac{ft}{ft}$$

$$\frac{S_w}{S_x} = \frac{0.138}{0.02} = 6.88$$

$$\frac{T}{W} = \frac{4.2 + 1.42}{1.42} = 3.96$$

$$E_o = 1 / \left\{ 1 + \frac{6.88}{\left[1 + \frac{6.88}{3.96 - 1}\right]^{2.67} - 1} \right\} = 0.78$$

$$Q_t = \frac{Q_s}{(1 - E_o)} = \frac{0.22}{(1 - 0.78)} = 1\ cfs$$

Q_t from Step 2.2 equals Q_g from Step 2 therefore the assumption of T_s = 4.2 feet in Step 2.1 is correct. Should Q_t not equal Q_g, a different value for T_s would need to be assumed.

Step 3:

Determine flow rate (Q_w) in gutter width.

$$Q_w = Q - Q_s$$

$$= 1.0\ cfs - 0.22\ cfs = 0.78\ cfs$$

Step 4:

Determine velocity of flow in gutter width using procedures listed in Example 3, Step 2.

Note: $T = T_s + W = 4.2\ ft + 1.42\ ft = 5.62\ ft$

$$d_1 = 0.084\ ft$$

$$d_2 = 0.28\ ft$$

Flow area = 0.26 sq ft

$$V = 3\ fps$$

Step 5:

Determine splash over velocity (V_o) from Figure 3.14.

$$V_o = 6.1\ fps$$

Step 6:

Determine the ratio (R_f) of frontal flow intercepted to total frontal flow for the grate. Use procedures listed in Example 3, Step 4.

$R_f = 1.28$ *if greater than 1, say 1*

With a combination curb opening and grate no clogging factor is applied to the grate.

Step 7:

Determine the ratio (R_s) of side flow intercepted to total side flow for the grate. Use procedures listed in Example 3, Step 5. No clogging factor applied.

$$R_s = 0.19$$

Step 8:

Using procedures listed in Example 3, Step 6 determine efficiency of the grate.

$$E = R_f E_o + R_s(1 - E_o)$$

$$E = (1)(0.78) + (0.19)(1 - 0.78) = 0.82$$

Step 9:

Determine the flow rate (Q_{ig}) intercepted by the grate.

$$Q_{ig} = Q \times E = 1\ cfs \times 0.82 = 0.82\ cfs$$

Step 10:

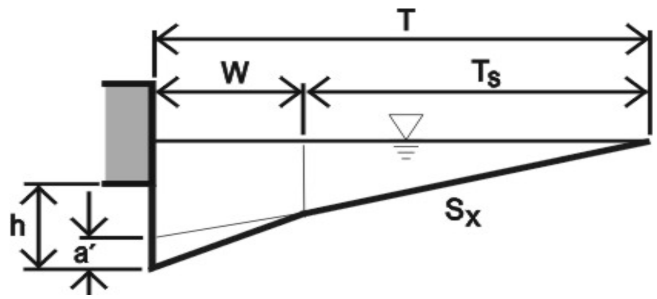
Determine the total flow (Q_i) intercepted by the combination catch basin.

$$Q_i = Q_{ic} + Q_{ig} = 3.0\ cfs + 0.82\ cfs = 3.82\ cfs$$

$$Q_{bypass} = Q - Q_i = 4\ cfs - 3.82\ cfs = 0.18\ cfs$$

Example 5

Determine length of curb opening inlet in a sump location.



Given:	Total flow rate	Q	=	4 cfs
	Allowable spread	T	=	12 ft
	Cross-slope	S_x	=	0.02 ft/ft
	Gutter depression at inlet	a	=	2 inches or 0.167 ft
	Width of gutter	W	=	1.42 ft
	Height of curb opening	H	=	5 inches or 0.417 ft
	Weir coefficient	C_w	=	2.3
	Clogging factor		=	1.25 applied to inlet length

Step 1:

Determine depth at inlet (d_i)

$$d_i = (S_x) \times (T)$$

$$d_i = (S_x) \times (T) = 0.02 \text{ ft/ft} \times 12\ feet = 0.24 feet$$

Step 2:

Check that

$$d_i < h + \frac{a}{12}$$

$$0.24\ ft < 0.417 + \frac{2}{12}$$

$$0.24\ ft < 0.58\ ft$$

Step 3:

Using Equation (3.11) determine length

$$Q_i = C_w(L + 1.8W)d_i^{1.5} \qquad (3.11)$$

or

$$L = \frac{Q_i}{C_w d_i^{1.5}} - 1.8W = \frac{4\ cfs}{(2.3)(0.24^{1.5})} - (1.8)(1.42\ feet) = 12.24\ feet$$

Using Figure 3.11 extend a vertical line up from the discharge rate value to the water depth value determined in Step 1, read P value.

$$P = 15$$

$$P = L + 1.8W$$

$$P - 1.8W = L$$

$$15 - (1.8)(1.42) = 12.44\ feet$$

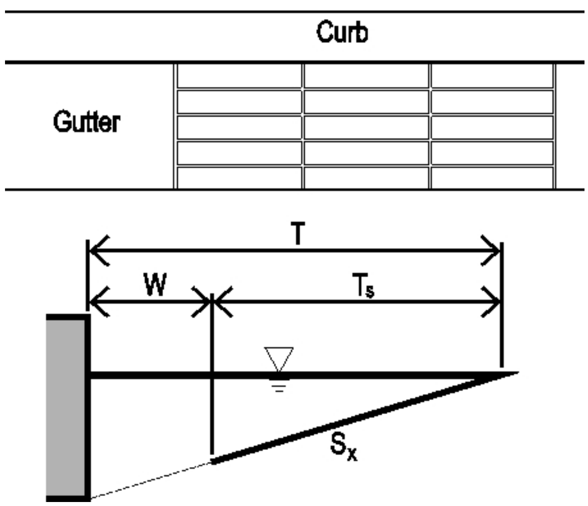
Step 4:

Apply clogging factor to inlet length

$$L \times clogging\ factor = 12.44 \times 1.25 = 15.55\ feet$$

Example 6

Determine size of a grate inlet in a sump condition.



Given:	Total flow rate	Q	=	2 cfs
	Allowable spread	T	=	12 ft
	Cross-slope	S_x	=	0.02 ft/ft
	Gutter depression	a	=	0
	Width of gutter	W	=	2

Weir coefficient	C_w	=	3.0
Grate dimensions		=	3 ft by 2 ft
Clogging factor		=	2 applied to grate perimeter

Step 1:

Determine depth at inlet (d)

$$d = (S_x) \times (T) = 0.02\,\text{ft/ft} \times 12.0\,feet = 0.24$$

Step 2:

Use Equation (3.21) to solve for P where P is equal to the perimeter of the grate in feet disregarding bars and the length of the side against the curb.

$$Q_i = C_w P d^{1.5} \tag{3.21}$$

$$\frac{Q_i}{C_w d^{1.5}} = P = \frac{2\,cfs}{3.0 \times 0.24^{1.5}} = 5.67\,ft$$

Using Figure 3.16 extend a vertical line up from the discharge rate value to the water depth value determined in Step 1, read P value.

$$P = 5.5\,feet$$

Step 3:

Apply clogging factor to perimeter of grate.

$$P \times clogging\,factor = 5.67\,feet \times 2 = 11.34\,feet$$

for 2 grates end to end $P = 10\,ft$

for 3 grates end to end $P = 13\,ft$

Use 3 grate inlets or try a different type of catch basin.

3.5 REFERENCES

Maricopa Association of Governments (MAG), 1979, *Uniform Standard Details for Public Works Construction*.

U.S. Department of Transportation (USDOT), Federal Highway Administration, 1984, *Hydraulic Engineering Circular No. 12, Drainage of Highway Pavements*. [USDOT Hydraulics WEB Site]

——, 1996, *Hydraulic Engineering Circular No. 22, Urban Drainage Design Manual*.

Page intentionally left blank

4 STORM DRAINS

TABLE OF CONTENTS

4.1 SYMBOLS

The following symbols will be used throughout Chapter 4.

a = The central angle of the bend, degrees

q = The horizontal angle of divergence or convergence between two sections, degrees

A = Area of water normal to flow, sq ft

C = Rational equation runoff coefficient

D = Diameter of storm drain, ft

E = Specific energy, ft

EGL = Energy grade line

g = Acceleration due to gravity, 32.2 ft/sec^2

HGL = Hydraulic grade line

h_b = Headloss due to a bend, ft

h_f = Headloss due to friction, ft

h_i = Headloss at inlet, ft

h_j	=	Headloss due to a junction, ft
h_{mn}	=	Headloss due to a manhole, ft
h_{minor}	=	Minor headlosses, ft
h_o	=	Headloss at outlet, ft
h_i	=	Headloss due to transition (contraction or expansion) in pipe size, ft
k	=	Pipe friction loss coefficient, dependant on Manning's *n*
k_b	=	Bend loss coefficient
k_c	=	Coefficient for transition loss due to contraction of flow
k_e	=	Coefficient for transition loss due to expansion of flow
k_j	=	Junction loss coefficient
k_{en}	=	Entrance loss coefficient
L	=	Horizontal length of a storm drain, ft
n	=	Manning's roughness coefficient
Q	=	Rate of flow, cfs
r	=	Radius of curvature, ft
R	=	Hydraulic radius, ft
S_f	=	Friction slope, ft/ft
S_o	=	Invert slope, ft/ft
T_c	=	Time of concentration, min.
V	=	Velocity, ft/sec
Y	=	Vertical distance from invert to hydraulic grade line, ft
Z	=	Elevation in reference to a known vertical datum, ft

4.2 INTRODUCTION

This chapter describes methodology that should be used for the hydraulic design of a storm drain system. In this manual, a storm drain system refers to a coordinated group of inlets, underground conduits, manholes, and various other appurtenances which are designed to collect stormwater runoff from the design storm and convey to a point of discharge into a major or regional drain outfall. The size of a storm drain system is based on a designated design storm. The design storm is a storm with a specific storm duration and return period. The design storm will vary from community to community. The designer shall determine the appropriate design storm from the governing agency.

Storm drains should generally only be considered for minor watercourses. Storm drains typically are not economical for the flows conveyed within larger watercourses. Therefore, the storm drain system will collect runoff to a point where storm drains become too large to be economical and will then discharge into a major or regional watercourse outfall consisting of a man-made channel, or natural watercourse.

The designer of the storm drain system will have to use professional judgement when dealing with the conflicts that can occur with existing utilities. When the designer has to deviate from the requirements of this chapter, he or she should contact the governing agency as soon as possible to explain the situation and agree upon an acceptable solution. This will expedite the design process.

There are many computer programs available to help in the design of storm drain systems. These programs, however, may determine the various headlosses by methods different than those presented in this chapter. It is therefore recommended that the designer of any storm drain system check with the governing agency before using a particular program.

4.3 PROCEDURES

4.3.1 General Considerations

The following considerations are intended to aid the designer in the design process for a storm drain system. The considerations discussed may not be applicable to all storm drain systems that are being designed. Also, the design approaches discussed may not be all of the alternatives a designer may have to take into consideration.

Manhole Design Considerations

A manhole is generally placed in a storm drain system at locations of pipe size/slope change, pipe horizontal alignment change, pipe intersections, and at other periodic locations to provide access to the system for maintenance. The following discussion applies to manholes and manholes/junctions.

Often a closed conduit designed for open channel flow operates as a pressure conduit. This may result when storm runoff exceeds that used for design purposes or simply because junction losses or manhole/junction losses were underestimated or neglected in the design. In storm drain systems, junctions in closed conduits can cause major headlosses across the junction. If these losses are not included in the hydraulic design, the capacity of the conduit may not be adequate for the desired design flow.

For a straight flow through condition at a manhole, pipes should be positioned vertically so that the crowns are aligned. An offset in the plan is allowable provided the projected area of the smaller pipe falls within that of the larger. Aligning the crowns of the pipes is the most hydraulically efficient.

When two inflowing laterals intersect in a manhole, the horizontal alignments of those laterals is important. For example, if two lateral pipes are aligned opposite each other such that the outflows impinge directly upon each other, the magnitude of the losses can be extremely high.

If the installation of directly opposed inflow laterals is necessary, the installation of a deflector, as shown in Figure 4.1 will result in significantly reduced losses. The research conducted on this type deflector is limited to the ratios of D_o/D_i = 1.25. The tests indicate that it would be conservative to assume the coefficient of pressure change at 1.6 for all flow ratios and pipe diameter ratios when no catch basin is considered, and 1.8 when the catch basin flow is more than 10 percent of Q_o.

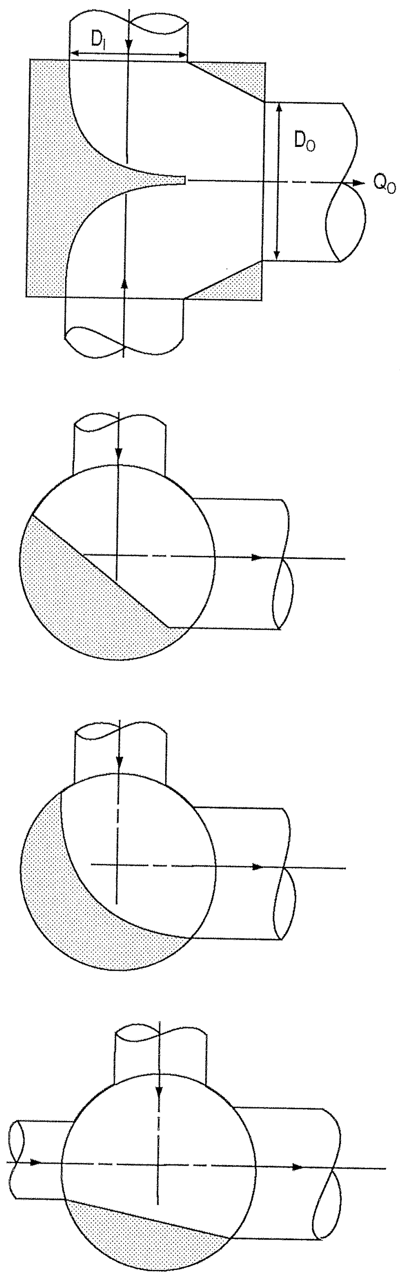
Lateral connector pipes should not be located directly opposite; rather, their centerlines should be separated laterally by at least the sum of the two lateral pipe diameters. Some jurisdictions require greater separation, and therefore, the design engineer should check jurisdiction specific standards. Studies have shown that this reduces headlosses as compared to directly opposed laterals, even with deflectors. Sufficient data has not been collected to determine the effect of offsetting laterals vertically.

Jets issuing from the upstream and lateral pipes must be considered when attempting to shape the inside of manholes. Tests for full flow revealed that very little, if anything, is gained by shaping the bottom of a manhole to conform to the pipe invert. Shaping of the invert may even be detrimental when lateral flows are involved, as the shaping tends to deflect the jet upwards, causing unnecessary headloss. From a practical point of view, limited shaping of the invert is necessary in order to handle low flows and to reduce sedimentation.

Figure 4.1 details several types of deflector devices that have been found efficient in reducing losses at junctions and bends. In all cases, the bottoms are flat, or only slightly rounded, to handle low flows. Numerous other types of deflectors or shaping of the manhole interiors were tested by the University of Missouri. Some of these devices which were found inefficient are shown in Figure 4.2. The fact that several of these inefficient devices would appear to be improvements indicates that special shapings deviating from those in Figure 4.1 should be used with caution, possibly only after model tests.

Tests indicate that rounding entrances or the use of pipe socket entrances do not have the effect on reducing losses that might be expected. Once again, the effect of the jet from the upstream pipe must be considered. Specific reductions to the pressure change factors are indicated with each design figure.

FIGURE 4.1
EFFICIENT MANHOLE SHAPING
(University of Missouri, 1958)



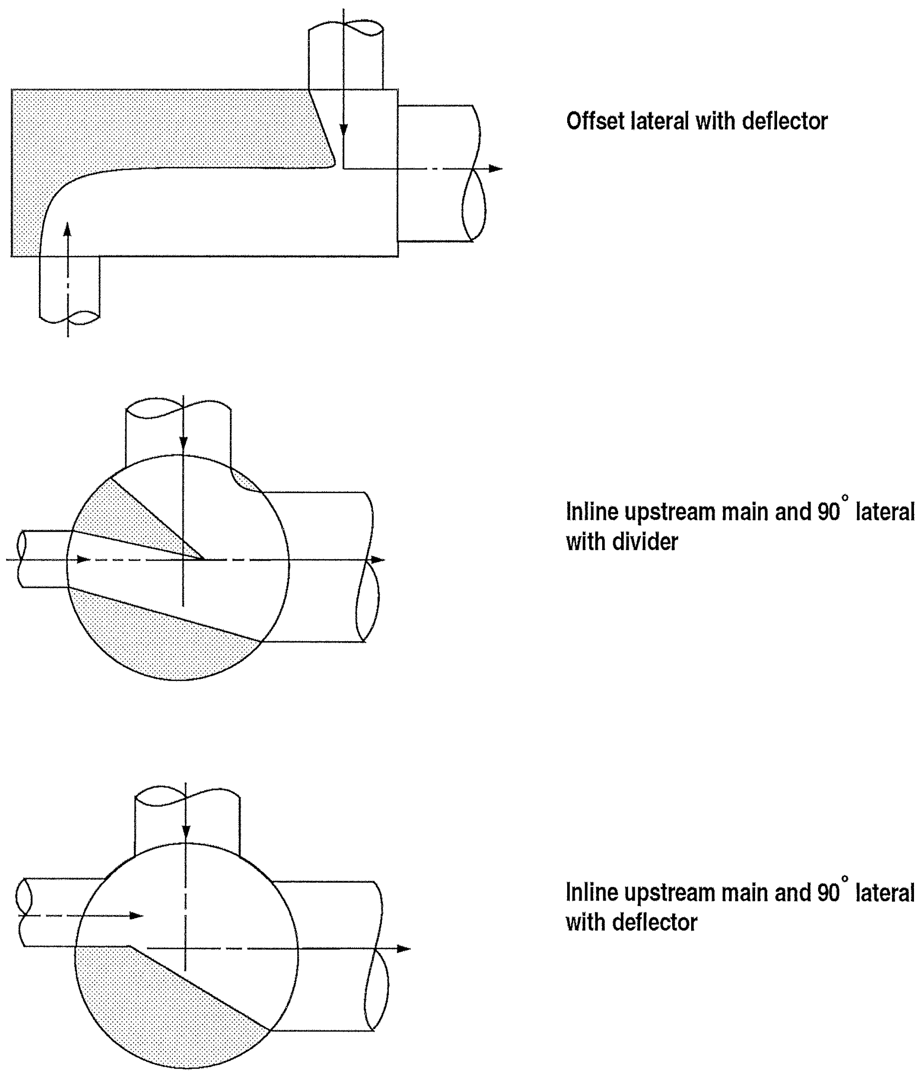
Directly opposed lateral with deflector (head losses are still excessive with this method, but are significantly less than when no deflector exists.)

Bend with straight deflector

Bend with curved deflector

Inline upstream main and 90° lateral with deflector

FIGURE 4.2
INEFFICIENT MANHOLE SHAPING



Although these modifications look like improvements, studies have proven these designs to be less efficient than the designs in Figure 4.1.

Use caution when deviating from recommended designs.

Other Junction Considerations

Lateral pipe entering a main line pipe storm drain generally should be connected radially (spring line to spring line). Lateral pipe entering a main line box structure should conform to the following:

a. Lateral pipe 24 inches or less in diameter should be no more than 5 feet above the invert.

b. Lateral pipe 27 inches or larger in diameter should be no more than 18 inches above the invert, with the exception that catch basin connector pipe less than 50 feet in length may be no more than 5 feet above the invert.

Exceptions to the above requirements may be permitted where it can be shown that the cost of bringing laterals into a main line box conduit in conformance with the above requirements would be excessive.

Debris/Access Barrier Considerations

An access barrier is a device for preventing people and animals from entering storm drain pipes. Protection barriers may consist of large, heavy breakaway gates, single horizontal bars across catch basin openings, or fencing around an exposed inlet or outlet. See Chapter 8 for more information on the hydraulic analysis of trashracks. Chapter 1 overviews safety related considerations for drainage structures including storm drains.

In some areas, there may be a high potential for debris to enter a storm drain which could block it. In these situations, a trashrack on an open inlet end of a storm drain pipe may be helpful. The governing agency should be contacted for determining how best to minimize the impact of the debris on the storm drain system.

Outlet Considerations

When a storm drain outlets into a natural channel, an outlet structure must be provided which prevents erosion and property damage. Velocity of flow at the outlet should agree as closely as possible with the existing channel velocity.

a. When the discharge velocity is low or subcritical, the outlet structure should consist of a concrete headwall, wingwalls, and an apron. See Chapter 6 for velocity tolerances for unlined and grass lined channels.

b. When the discharge velocity is high or supercritical, the designer should also consider adding bank protection in the vicinity of the outlet and an energy dissipator structure.

See Chapter 5 and Chapter 8 for additional information concerning conduit outlet structures.

The orientation of the outfall is another important design consideration. Where practical the outlet of the storm drain should be positioned in the outfall channel so that it is pointed in a downstream direction. This will reduce excessive flow disturbance and the potential for excessive erosion. If the outfall structures can not be oriented in a downstream direction, the potential for outlet scour must be considered. For example, where a storm drain outfall discharges perpendicular to the direction of flow of the receiving channel consideration should be given to the possibility of erosion on the opposite channel bank. If erosion potential exists, a channel bank lining of riprap or other suitable material should be installed in the bank. Alternatively an energy dissipator structure could be used at the storm drain outlet.

Inlet Design Considerations

In general, the interception of flow from a natural watercourse directly into a storm drain system should be avoided. If avoiding this situation is not possible, then an inlet structure should be provided. Strong consideration should be given to the use of a debris or sediment basin upstream of the inlet structure. The inlet structure should generally consist of a headwall, wingwalls to protect the adjacent banks from erosion, and a paved inlet apron. Wall heights should conform to the height of the water upstream of the inlet, and should be adequate to protect both the fill over the drain and the embankments. Headwall and wingwall fencing, an access barrier, and a trashrack to promote public safety should be considered. Chapter 8 provides more considerations on inlets/ outlets for storm conduits. See Chapter 1 for more information on safety and fencing.

Transition from Large to Small Conduit

As a general rule, storm drains are designed with sizes increasing in the downstream direction. However, when studies indicate it may be advisable to decrease the size of a downstream section, the conduit may be decreased in size in accordance with the following limitations:

a. For slopes of 0.0025 ft/ft (0.25 percent) or less, only conduits 78 inches and greater may be decreased in size a maximum of 6 inches.

b. For slopes of more than 0.0025 ft/ft, only conduits 36 inches and greater may be decreased in size. Each reduction should be limited to a maximum of 6 inches for pipe larger than 48 inches in diameter. Reductions exceeding the above criteria should be approved by the governing agency.

The pipe size reductions should include approved transitions; should result in a more economical system; and should not cause any adverse impacts.

4.3.2 Applications and Limitations

Presented in this section are the general procedures for hydraulic design and evaluation of storm drains. Calculations to determine a hydraulic grade line in a storm drain system begin with a known hydraulic grade elevation at some downstream point. To this point are added the various headlosses that occur in the subject segment to determine the upstream hydraulic grade line elevation. The following discussions and equations are to be used in the calculation of headlosses for a storm drain system. Criteria to be used in the estimation of a hydraulic grade line for a storm drain are discussed in the Criteria subsection of this chapter.

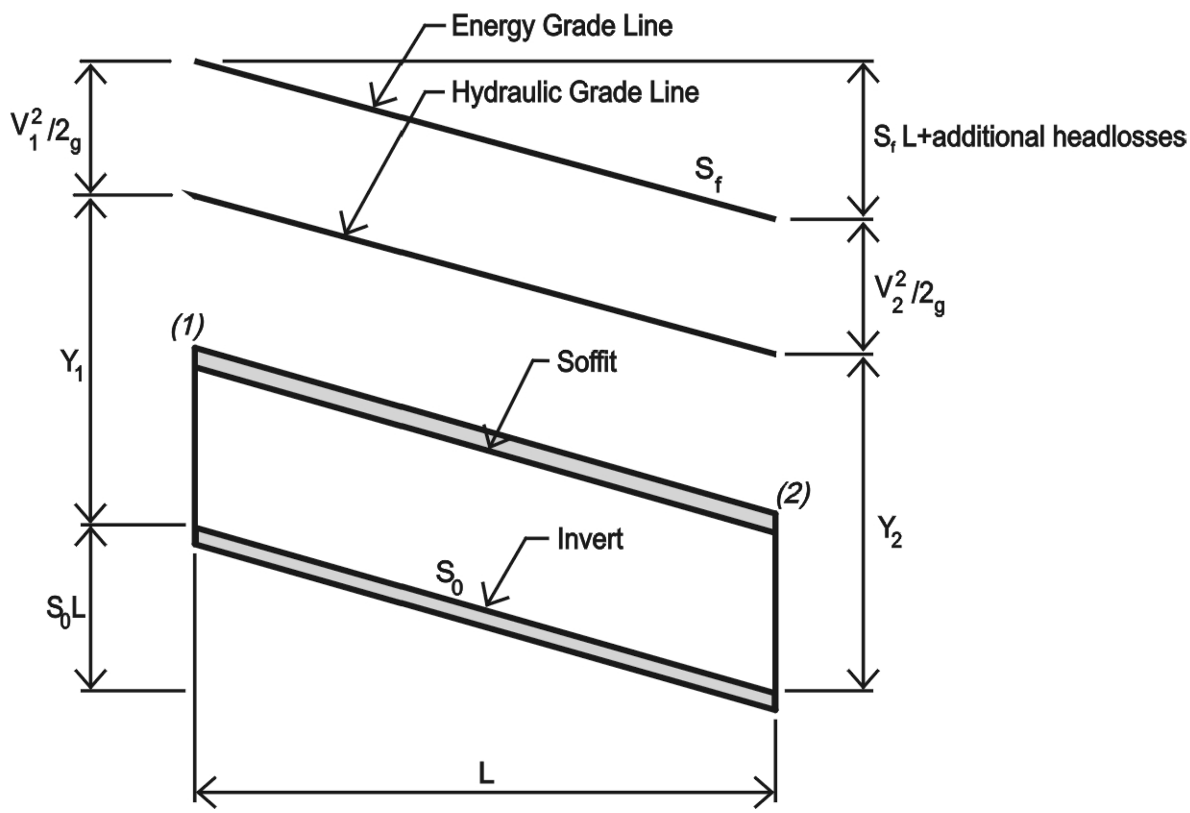
Energy Equation

Most procedures for calculating hydraulic grade line profiles are based on the energy equation and can be expressed as:

$$\frac{V_1^2}{2g} + Y_1 + S_o L = \frac{V_2^2}{2g} + Y_2 + S_f L + headlosses \qquad (4.1)$$

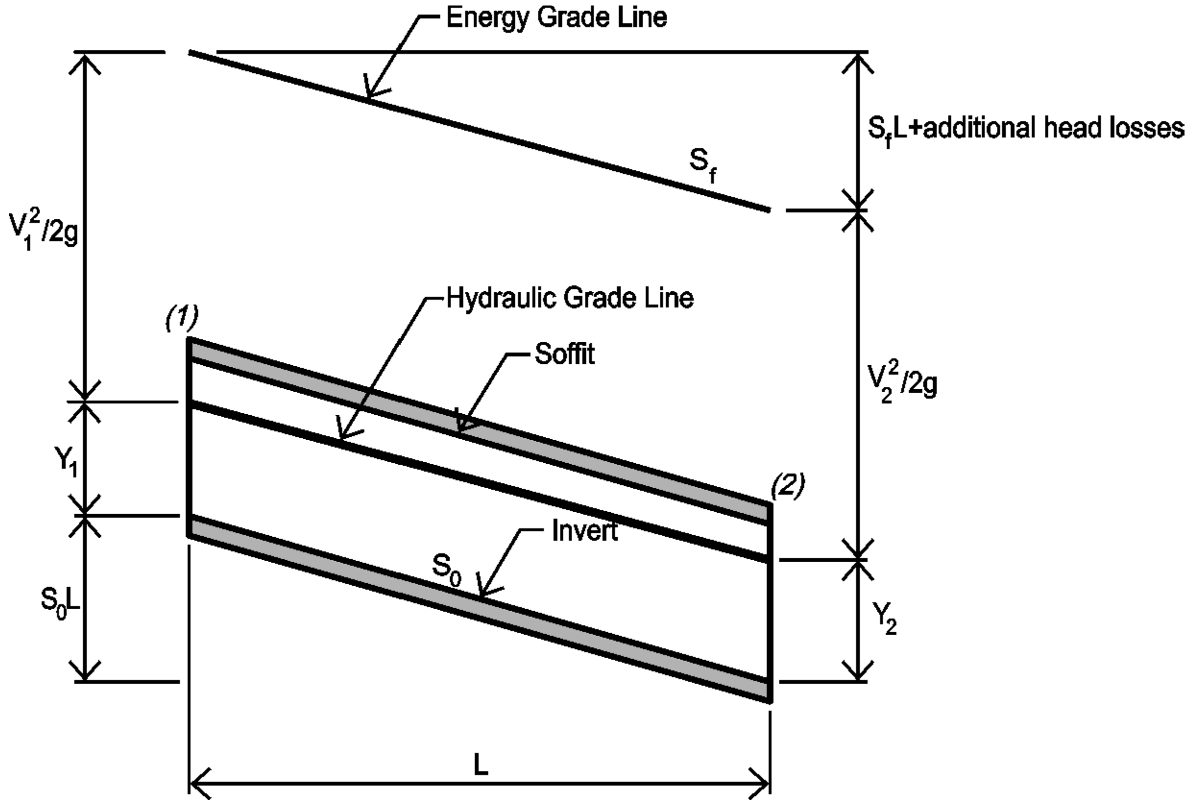
The various terms used in Equation (4.1) are identified in Figure 4.3 and Figure 4.4. Minor losses have been included in the energy equation because of their importance in calculating hydraulic grade line profiles.

FIGURE 4.3
STORM DRAIN PROFILE PRESSURE FLOW CONDITIONS
(MODIFIED FROM Los Angeles County Flood Control District, 1982)



As depicted, Y_1 and Y_2 include the pressure components since they are above the soffit of the pipe.

FIGURE 4.4
STORM DRAIN PROFILE OPEN FLOW CONDITIONS



In this presentation of design methods, provision is made to identify pipes by use of numbered subscripts. The number one (1) is used to identify the upstream main pipe, the number two (2) is used to identify the downstream main pipe, and the number three (3) is used for incoming or branching flow.

The general procedure for the hydraulic calculations is to establish the downstream control elevation. From there the hydraulic calculations proceed upstream from point of interest to point of interest. For example, from one junction to another junction or from a junction to the beginning of a bend. At the lower end of each point of interest the pipe friction losses from the downstream section are added to the downstream hydraulic grade line. The losses through the point of interest are added at the upstream end of the point of interest. The procedures for calculating the various headlosses encountered in a storm drain system are presented in the following Head Losses Section. Figure 4.5 may be used to assist in the accounting and computing of the losses.

FIGURE 4.5
HYDRAULIC GRADE LINE CALCULATION SHEET

Station	Invert Elev.	Size/Type	n	Area, sq ft	Q, cfs	V, ft/sec	$V^2/2g$, ft	K	S_f, ft/ft	Average S_f, ft/ft	Length, ft	h_1, ft	h_b, ft	h_j, ft	h_t, ft	Other Losses, ft	Total Losses, ft	Energy Grade Line Elev., ft	Hydraulic Grade Line Elev., ft	Comments

n = Manning's roughness coefficient

$V^2/2g$ = Velocity head

h_f = Headloss due to friction

Q = Rate of flow

K = $2gn^2/2.21$

h_b = Headloss due to a bend

V = Velocity

S_f = Friction slope

h_j = Headloss due to a junction

Equation (4.2) is a simplification of a more complex equation and is a convenient method for locating the approximate point where pressure flow may cease (may become open channel flow). It is derived by substituting specific energy (E) for the quantity $V^2/2g + Y$ in Equation (4.1) and rearranging the results. For S_f use the average friction slope between the two points of interest.

$$L = \frac{E_2 - E_1}{S_o - S_f} \tag{4.2}$$

Head Losses

The headlosses that need to be determined are: friction, transition, junction, manhole, bend, inlet and exit. These losses need to be determined individually and then added together to determine the overall headloss for each segment of the storm drain. The methods for determining the various headlosses presented in this section were selected for their wide acceptance and ease of use.

Friction Losses

Friction losses for closed conduits carrying stormwater, including pump station discharge lines, will be calculated from Manning's equation or a derivation thereof. The Manning's equation is commonly expressed as follows:

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \tag{4.3}$$

where:

Q = Rate of flow, cfs

n = Manning's roughness coefficient

A = Flow area, sq ft

R = Hydraulic radius, ft

S_f = Friction slope, ft/ft

The equation for determining pipe friction slope can be expressed as,

$$S_f = K \frac{V^2}{2gR^{4/3}} \tag{4.4}$$

where:

V = Velocity, ft/sec

g = Acceleration due to gravity, $32.2 ft/\sec^2$

The value of K is dependent only upon the roughness coefficient (n) for the pipe. The Manning's n-values for various pipe materials are given in Table 4.1. The value of K can be estimated using Equation (4.5).

$$K = \frac{2gn^2}{2.21} \tag{4.5}$$

where: g = Acceleration due to gravity, $32.2 ft/\sec^2$

TABLE 4.1
VALUES OF ROUGHNESS AND FRICTION FORMULA COEFFICIENTS FOR CLOSED CONDUITS

Conduit Material	Manning's n
Asbestos Cement Pipe	0.013
Brick	0.015
Cast Iron Pipe	
Cement lined & seal coated	0.013
Concrete (monolithic)	
Smooth forms	0.013
Rough forms	0.017
Concrete Pipe	0.013
Corrugated Metal Pipe (1/2 x 2 2/3 inch corrugations)	
Plain	0.024
Paved invert	0.020
Spun asphalt lined	0.013
Corrugated Polyethylene Pipe	
15” Diameter	0.018
18” to 36” Diameter	0.020
Plastic Pipe (smooth)	0.013
Vitrified Clay	
Pipes	0.013
Liner plates	0.013

The loss of head due to friction throughout the length of reach (L) is calculated by:

$$h_f = S_f L \tag{4.6}$$

where: h_f = Friction headloss, ft

L = Reach length, ft

Transition Losses

There are two types of pipe transitions that can occur in a storm drain system that would add headloss to the energy grade line. The transition types are expansion and contraction. Figure 4.6 shows the two types of transitions that can be encountered. The headloss due to the expansion of flow for a storm sewer ***flowing under open channel conditions*** is expressed as:

$$h_t = k_e\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \tag{4.7}$$

where:

h_t = Transition headloss, ft

k_e = Coefficient for transition loss due to expansion

V_1 = Upstream velocity, ft/sec

V_2 = Downstream velocity, ft/sec

g = Acceleration due to gravity, 32.2 ft/sec^2

Note: V_1 is greater than V_2

The values for the transition coefficient, k_e, for enlargements are given in Table 4.2.

The headloss due to the contraction of flow ***under open channel flow conditions*** is expressed as:

$$h_t = k_c\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) \tag{4.8}$$

where:

k_c = Coefficient for transition loss due to contraction

V_1 = Upstream velocity, ft/sec

V_2 = Downstream velocity, ft/sec

Note: V_2 is greater than V_1

Values for the transition loss coefficient, k_c, for contractions can also be found in Table 4.2.

FIGURE 4.6
TRANSITION LOSS

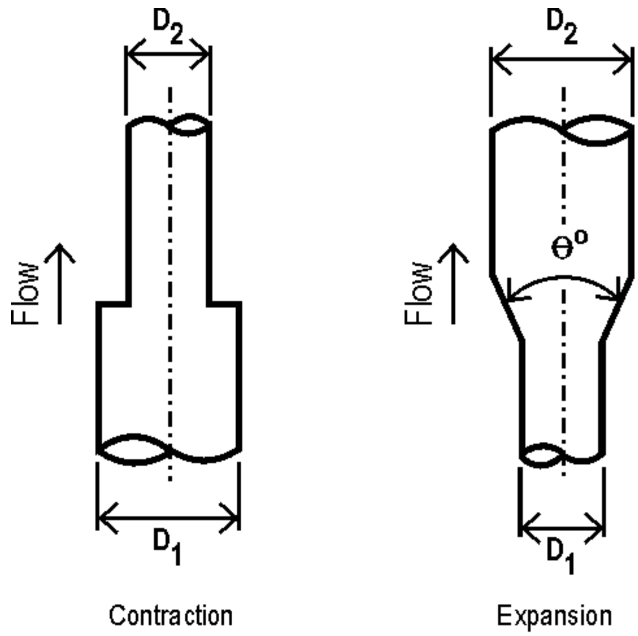


TABLE 4.2
STORM SEWER ENERGY LOSS COEFFICIENTS UNDER OPEN CHANNEL CONDITIONS
(ASCE, 1992)

(a) Contractions (K_c)		**(b) Expansion (K_e)**		
$\frac{D_2}{D_1}$	K_c	θ	$\frac{D_2}{D_1} = 3$	$\frac{D_2}{D_1} = 1.5$
0	0.5	10	0.17	0.17
0.4	0.4	20	0.40	0.40
0.6	0.3	45	0.86	1.06
0.8	0.1	60	1.02	1.21
1.0	0	90	1.06	1.14
		120	1.04	1.07
		180	1.00	1.00

Under pressure flow conditions, the headloss due to contraction and expansion of flow can be expressed as:

$$h = k\frac{V^2}{2g} \tag{4.9}$$

where:

h = Headloss due to a contraction or expansion, ft

k = Coefficient for contraction (k_c) or coefficient for expansion (k_e), see below.

V = Velocity of flow in the smallest diameter pipe, ft/sec

The values for the transition coefficient, k_e, for gradual enlargements are given in Table 4.3a. For sudden enlargements, values for the transition coefficients are listed in Table 4.3b. Values for the transition loss coefficient, k_c, for sudden contractions can be found in Table 4.4.

Table 4.3a

COEFFICIENT k_e FOR GRADUAL ENLARGEMENT UNDER PRESSURE FLOW CONDITIONS

(AISI, 1990)

D_2/D_1	Angle of Cone, degrees													
	2	**4**	**6**	**8**	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**60**
1.1	.01	.01	.01	.02	.03	.05	.10	.13	.16	.18	.19	.20	.21	.23
1.2	.02	.02	.02	.03	.04	.09	.16	.21	.25	.29	.31	.33	.35	.37
1.4	.02	.03	.03	.04	.06	.12	.23	.30	.36	.41	.44	.47	.50	.53
1.6	.03	.03	.04	.05	.07	.14	.26	.35	.42	.47	.51	.54	.57	.61
1.8	.03	.04	.04	.05	.07	.15	.28	.37	.44	.50	.54	.58	.61	.65
2.0	.03	.04	.04	.05	.07	.16	.29	.38	.46	.52	.56	.60	.63	.68
2.5	.03	.04	.04	.05	.08	.16	.30	.39	.48	.54	.58	.62	.65	.70
3.0	.03	.04	.04	.05	.08	.16	.31	.40	.48	.55	.59	.63	.66	.71
•	.03	.04	.05	.06	.08	.16	.31	.40	.49	.56	.60	.64	.67	.72

Table 4.3b

COEFFICIENT k_e FOR SUDDEN ENLARGEMENT UNDER PRESSURE FLOW CONDITIONS

(AISI, 1990)

D_2/D_1	Velocity, V_1 (ft/sec)											
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.2	.11	.10	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09	.08
1.4	.26	.26	.25	.24	.24	.24	.24	.23	.23	.22	.22	.21	.20
1.6	.40	.39	.38	.37	.37	.36	.36	.35	.35	.34	.33	.32	.32
1.8	.51	.49	.48	.47	.47	.46	.46	.45	.44	.43	.42	.41	.40
2.0	.60	.58	.56	.55	.55	.54	.53	.52	.52	.51	.50	.48	.47
2.5	.74	.72	.70	.69	.68	.67	.66	.65	.64	.63	.62	.60	.58
3.0	.83	.80	.78	.77	.76	.75	.74	.73	.72	.70	.69	.67	.65
4.0	.92	.89	.87	.85	.84	.83	.82	.80	.79	.78	.76	.74	.72
5.0	.96	.93	.91	.89	.88	.87	.86	.84	.83	.82	.80	.77	.75
10.0	1.00	.99	.96	.95	.93	.92	.91	.89	.88	.86	.84	.82	.80
•	1.00	1.00	.98	.96	.95	.94	.93	.91	.90	.88	.86	.83	.81

Table 4.4

COEFFICIENT k_c FOR SUDDEN CONTRACTION UNDER PRESSURE FLOW CONDITIONS

(AISI, 1990)

D_1/D_2	Velocity, V_2 (ft/sec)											
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.1	.03	.04	.04	.04	.04	.04	.04	.04	.04	.04	.05	.05	.06
1.2	.07	.07	.07	.07	.07	.07	.07	.08	.08	.08	.09	.10	.11
1.4	.17	.17	.17	.17	.17	.17	.17	.18	.18	.18	.18	.19	.20
1.6	.26	.26	.26	.26	.26	.26	.26	.26	.26	.25	.25	.25	.24
1.8	.34	.34	.34	.34	.34	.34	.33	.33	.32	.32	.31	.29	.27
2.0	.38	.38	.37	.37	.37	.37	.36	.36	.35	.34	.33	.31	.29
2.2	.40	.40	.40	.39	.39	.39	.39	.38	.37	.37	.35	.33	.30
2.5	.42	.42	.42	.41	.41	.41	.40	.40	.39	.38	.37	.34	.31
3.0	.44	.44	.44	.43	.43	.43	.42	.42	.41	.40	.39	.36	.33
4.0	.47	.46	.46	.46	.45	.45	.45	.44	.43	.42	.41	.37	.34
5.0	.48	.48	.47	.47	.47	.46	.46	.45	.45	.44	.42	.38	.35
10.0	.49	.48	.48	.48	.48	.47	.47	.46	.46	.45	.43	.40	.36
•	.49	.49	.48	.48	.48	.47	.47	.47	.46	.45	.44	.41	.38

Junction Losses

A junction occurs where one or more lateral pipes enter the main storm drain, at a formed junction, prefabricated fitting or at a manhole. Multiple pipes coming together at a junction should flow together smoothly to avoid high headlosses. Figure 4.7 through Figure 4.9 show typical junctions in plan and profile.

Junction headloss for a single lateral can be determined by applying the Energy Equation and the Thompson Equation (California Department of Transportation, 1985).

The Energy Equation (Equation (4.1)) at a junction (as displayed in Figure 4.7 through Figure 4.9) is expressed as:

$$V_1^2/2g + Y_1 + Z_1 = V_2^2/2g + Y_2 + Z_2 + headlosses \quad (4.1)$$

where: *headlosses* = h_j (junction loss) + h_T (transition loss) + h_F (friction loss)

$V_1^2/2g$ = Main line velocity head upstream of junction, ft

$V_2^2/2g$ = Mainline velocity head downstream of junction, ft

Y_1 = Upstream hydraulic gradient elevation measured from invert, ft

Y_2 = Downstream hydraulic gradient elevation measured from invert, ft

Z_1 = Elevation at location Z_1, ft

Z_2 = Elevation at location Z_2, ft

FIGURE 4.7
FORMED OR PREFAB STORM DRAIN JUNCTION

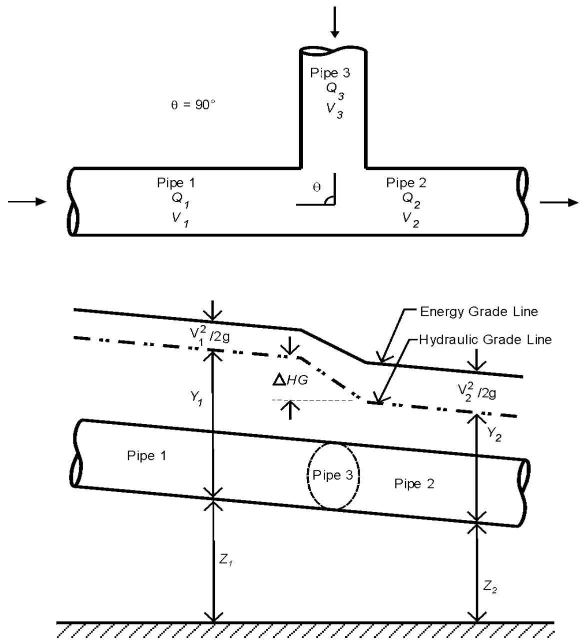


FIGURE 4.8
STORM DRAIN JUNCTION AT MANHOLE WITH ALIGNED CROWNS UNDER PRESSURE FLOW

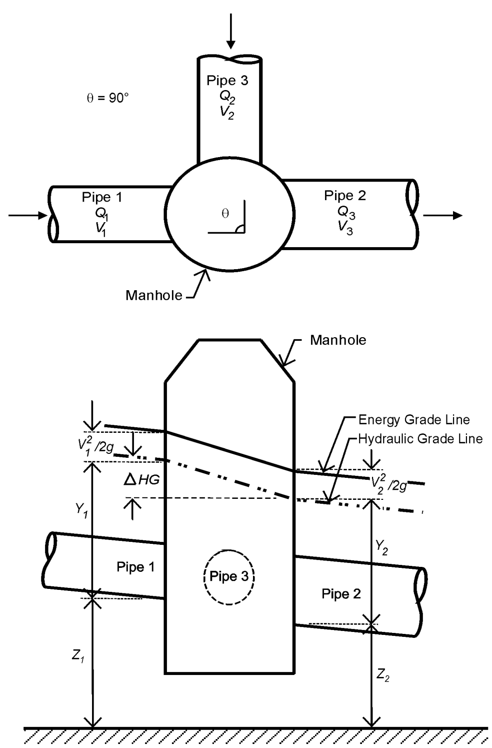
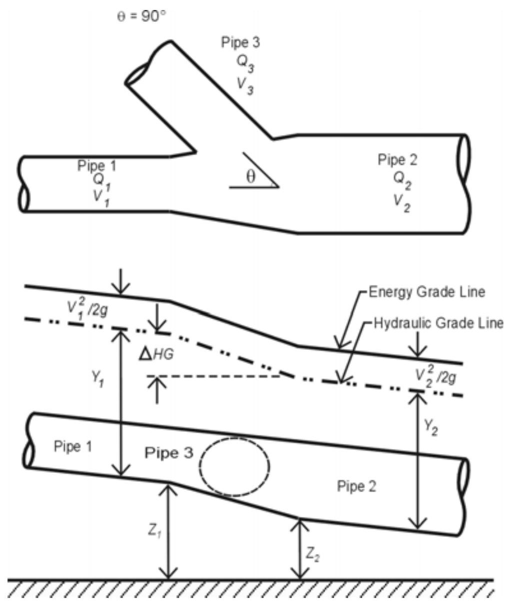


FIGURE 4.9
FORMED STORM DRAIN JUNCTION WITH ALIGNED CROWNS UNDER PRESSURE FLOW



Equation (4.1) can be rewritten to solve for headlosses

$$V_1^2/2g - V_2^2/2g + Y_1 - Y_2 + Z_1 - Z_2 = headlosses$$

Substitute HG_1 for $Y_1 + Z_1$ and HG_2 for $Y_2 + Z_2$

$$V_1^2/2g - V_2^2/2g + HG_1 - HG_2 = headlosses$$

$$V_1^2/2g - V_2^2/2g + \Delta HG = headlosses$$

The Thompson Equation (Equation (4.10a)), a form of the momentum equation, is used to determine the change in flow depth across a junction.

$$\Delta HG\frac{A_1 + A_2}{2} = \frac{Q_2V_2 - Q_1V_1 - Q_3V_3Cos\theta}{g} \qquad (4.10a)$$

or

$$\Delta HG = \frac{\left(\dfrac{Q_2V_2 - Q_1V_1 - Q_3V_3Cos\theta}{g}\right)}{\dfrac{A_1 + A_2}{2}}$$

where: ΔHG = Difference in upstream and downstream hydraulic grade line elevations, ft

A_1 = Upstream flow area, sq ft

A_2 = Downstream flow area, sq ft

Q_1 = Upstream flow rate, cfs

Q_2 = Downstream flow rate, cfs

Q_3 = Lateral flow rate, cfs

V_1 = Upstream flow velocity, ft/sec

V_2 = Downstream flow velocity, ft/sec

V_3 = Lateral flow velocity, ft/sec

q = Angle between lateral and main line storm drain (See Figure 4.9), degrees

To determine junction headloss h_j, substitute the Thompson Equation into the rewritten Equation (4.1), assuming transition and friction losses at the junction are negligible.

$$\frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3Cos\theta)}{(A_1 + A_2)g} + V_1^2/2g - V_2^2/2g = h_j \tag{4.10b}$$

Should friction losses be determined not to be negligible Equation (4.10c) should be used.

$$\frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3Cos\theta)}{(A_1 + A_2)g} + V_1^2/2g - V_2^2/2g + \left(\frac{S_{f1} + S_{f2}}{2}\right)L = h_j \tag{4.10c}$$

where: S_{f1} = Upstream friction slope, ft

S_{f2} = Downstream friction slope, ft/ft

L = Length of transition, ft

Should transition losses be determined not to be negligible but friction losses are negligible, then Equation (4.10d) should be used for computing junction loss h_j.

$$h_j = \frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3Cos\theta)}{(A_1 + A_2)g} + V_1^2/2g - V_2^2/2g + k_{je}\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \tag{4.10d}$$

where:

k_{je} = Coefficient for transition loss due to expansion at a junction.

k_{je} = 3.50 x (Tan θ /2) $^{1.22}$(California Department of Transportation, 1985). See Figure 4.7 through Figure 4.9 for location of θ angle.

V_1 = Upstream velocity, ft/sec

V_2 = Downstream velocity, ft/sec

g = Acceleration due to gravity, 32.2 ft/sec^2

In situations where crowns at a junction are not matching, a pressure momentum approach for solving headloss is suggested. A pressure momentum approach is described in Section 4.8.

Straight-Through Manhole Losses (no laterals) - In a straight-through manhole where there is no change in pipe size or rate of flow, the loss can be estimated by:

$$h_{mh} = 0.05\frac{V^2}{2g} \tag{4.11}$$

where: h_{mh} = Headloss due to a manhole, ft

V = Velocity, ft/sec

Bend Losses at Manholes (no laterals) - The bend loss at a manhole is determined using Equation (4.12). The bend loss coefficient, k_b, can be determined using Figure 4.11.

$$h_{mh} = k_b\frac{V^2}{2g} \tag{4.12}$$

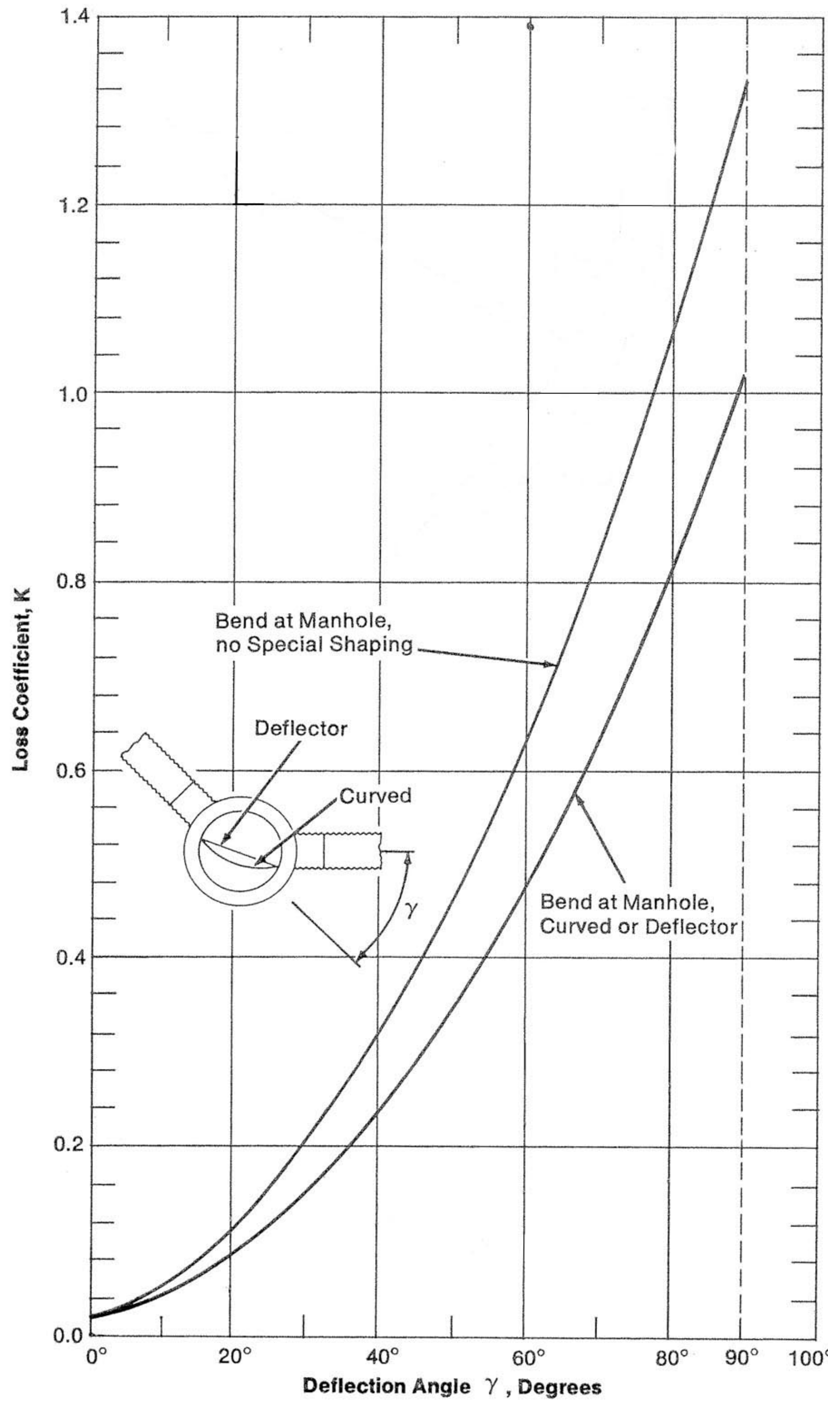
where: h_{mh} = Headloss due to a manhole, ft

k_b = Bend loss coefficient

V = Velocity, ft/sec

g = Acceleration due to gravity, 32.2 ft/sec^2

FIGURE 4.10
BEND LOSS COEFFICIENT
(MODIFIED FROM AISI, 1990)



Bend Losses at Curved Sewer - For bend loss at a curved sewer, the loss is calculated using Equation (4.13).

$$h_b = k_b \frac{V^2}{2g} \tag{4.13}$$

where: h_b = Headloss due to a bend, ft

k_b = Bend headloss coefficient

V = Velocity of flow in the bend, ft/sec

The value of the bend loss coefficient, k_b, depends upon the angle of the bend. It can be estimated from Equation (4.14) (USDOT, 2001).

$$k_b = 0.0033\Delta \tag{4.14}$$

where: k_b = Bend headloss coefficient

Δ = Angle of curvature or deflection, degrees

Bend losses should be included for all closed conduits, those flowing partially full as well as those flowing full.

Inlet Losses - At open inlets to a storm drain system, an inlet will function the same as a culvert inlet. Under inlet control, the hydraulic grade line at the entrance can be estimated by using the appropriate procedures and figures presented in the Culvert Chapter. Under outlet control, entrance losses can be calculated using Equation (4.15).

$$h_i = k_{en} \frac{V^2}{2g} \tag{4.15}$$

where: h_i = Headloss at inlet, ft

k_{en} = Entrance loss coefficient

The k_{en} in the equation is equivalent to k_e values listed in Table 5.1.

In addition to the entrance loss, losses associated with a protection barrier or trashrack over the inlet should be taken into consideration. Procedures to estimate headlosses due to barriers or trashracks can be found in Section 8.2.5.

Exit Losses - When a storm drain outfalls to a retention basin, lake, or open channel, additional headloss occurs due to the change in velocity at the outlet of the pipe, and due to the changes in flow direction. The exit headloss at storm drain outlets is expressed as (Clark County Regional Flood Control District, 1990):

$$h_o = 1.0\frac{V_o^2}{2g} \tag{4.16}$$

where: h_o = Headloss at outlet, ft

V_o = Average outlet velocity, ft/sec

4.4 CRITERIA

4.4.1 Main Line Hydraulic Grade Line

Presented in this section are the general criteria for hydraulic design and evaluation of storm drains. Calculations to check the pressure (hydraulic grade) of water surface elevations in the storm drain system begin with a known hydraulic grade elevation at some downstream point. To this are added the various losses that occur to determine the upstream hydraulic grade elevation. These losses are commonly referred to as headlosses. The procedures for calculating the various headlosses are presented in the Head Losses section of this chapter.

If the hydraulic grade line is above the pipe crown at the upstream junction, full flow calculations may proceed. If the hydraulic grade line is below the pipe crown at the upstream junction, then open channel flow calculations must be used.

To expedite computations, the storm drain hydraulic grade line elevation determined at a junction should first be compared to the elevation of the top of the downstream pipe and the gutter. Because of the usual losses that occur at a junction, the upstream hydraulic elevations and the water elevation in the catch basin can be much higher than the elevation of the downstream storm drain hydraulic grade line. Comparison to limiting conditions will indicate whether the design may be continued upstream or re-designed to accommodate limiting conditions.

The general procedures for establishing the quantity of flow and layout are the same for a closed conduit flowing either as an open channel or as a pressure conduit. Because of the nature of hydraulic elements in circular conduits, it may be reasonably assumed that open channel flow will occur only when the flow depth is less than 80 percent of the conduit diameter.

Even though a conduit may be designed to carry stormwater as open channel flow, losses at bends and junctions will frequently cause pressure flow to occur for some distance upstream of the “loss” area. Situations may occur in steep terrain where the flow often interchanges between open channel and pressure flows. Because it is not economical to size conduits to avoid pressure flow under all storm runoff and flow conditions, it follows that it is reasonable and even necessary to design the conduits as flowing full. Planned management of stormwater runoff is also easier to achieve if the hydraulic grade line is kept higher than the crown of the conduit. The discharge through a circular pipe flowing full is constant for a given pipe diameter and hydraulic gradient. Once the hydraulic gradient intercepts the elevation of the inflow at a catch basin, no further runoff can be admitted to the pipe network. This phenomenon in the field would be evidenced by runoff passing directly over the catch basin to flow down the street (or overland) until it enters the system elsewhere. Another indication is water standing in sumps (storage facility ponding) until there is sufficient capacity in the storm drain to admit the ponded water. The designer should size the pipes so that the hydraulic grade line is below the inlet elevation for the design storm frequency. The separation distance between the inlet elevation and the hydraulic grade line is set by the reviewing agency as a standard for storm sewer design.

Often a closed conduit designed for open channel flow operates as a pressure conduit. This may result when storm runoff exceeds that used for design purposes or simply because junction losses were underestimated or neglected in the design. In storm drain systems, junctions in closed conduits can cause major losses in the energy grade line across the junction. If these losses are not included in the hydraulic design, the capacity of the conduit may not be adequate for the design flow.

Although not always feasible, the recommended procedure is to design storm drains to flow under pressure because this maximizes conveyance while minimizing capital expenditure. Whether or not the final design assumes the pipe is flowing partially or completely full, a hydraulic grade line must be computed and displayed on a profile drawing of the conduit. The design shall establish the hydraulic grade line to be below an inlet, ground or manhole rim elevation. When the hydraulic grade line rises above ground level, stormwater can be found shooting out of catch basins or popping manhole covers, which can lead to damage and inconvenience to pedestrian and vehicular traffic.

4.4.2 Determination of Controlling Water Surface Elevation

A storm drain system may discharge into one of the following:

1. A body of water such as a storage facility, reservoir, or lake.
2. A natural watercourse or open channel (either improved or unimproved).
3. Another closed conduit.

The controlling water surface elevation at the point of discharge is commonly referred to as the tailwater elevation. The tailwater elevation at the storm drain outfall must be considered carefully. Evaluation of the hydraulic grade line for a storm drain system begins at the system outfall with the tailwater elevation.

Generally, it shall be assumed that the tailwater elevation at the storm drain outlet is equivalent to the water surface elevation within the receiving channel or facility which has the same return period as the storm drain design discharge, unless otherwise approved by the governing agency. In general the two types of tailwater conditions are:

1. Tailwater elevation above the crown elevation. In such situations the control shall conform to the following criteria:

    a. In the case of a conduit discharging into a storage basin, the control shall be the storage basin water surface elevation coinciding with the design peak flow to the storage basin.

b. In the case of a conduit discharging into an open channel, the tailwater elevation shall be the water surface elevation of the channel coinciding with same return period as the storm drain design peak discharge.

c. In the case of a conduit discharging into another conduit, the control shall be the highest hydraulic grade line elevation of the outlet conduit immediately upstream or downstream of the confluence.

2. Tailwater elevation at or below the crown elevation. The tailwater shall be the crown elevation at the point of discharge.

4.4.3 Connector Pipe Hydraulic Grade Line

Connector pipes connecting catch basins to storm drains can be sized and/or evaluated by estimating headlosses due to friction and inlet losses at catch basin. The designer should consider the catch basin connector pipes to be flowing full. The headloss due to friction can be estimated by using Equation (4.6). The headlosses at the inlet of the connector pipe can be estimated by using Equation (4.17). Equation (4.17) is modified from Equation (4.15):

$$h_i = (1 + k_{en})\frac{V^2}{2g} \tag{4.17}$$

where: h_i = Headloss at inlet, ft

k_{en} = Entrance loss coefficient

The k_{en} in the equation is equivalent to k_e values listed in Table 5.1.

4.5 DESIGN STANDARDS

Design standards may vary from community to community. The designer shall adhere to policies and standards of the governing agency. For a detailed description of design standards the designer is referred to the Policy/Standards Manual of the governing agency. When the designer has to deviate from the standards for storm drain design, they should contact the governing agency as soon as possible to explain the situation and come to an agreement on a solution.

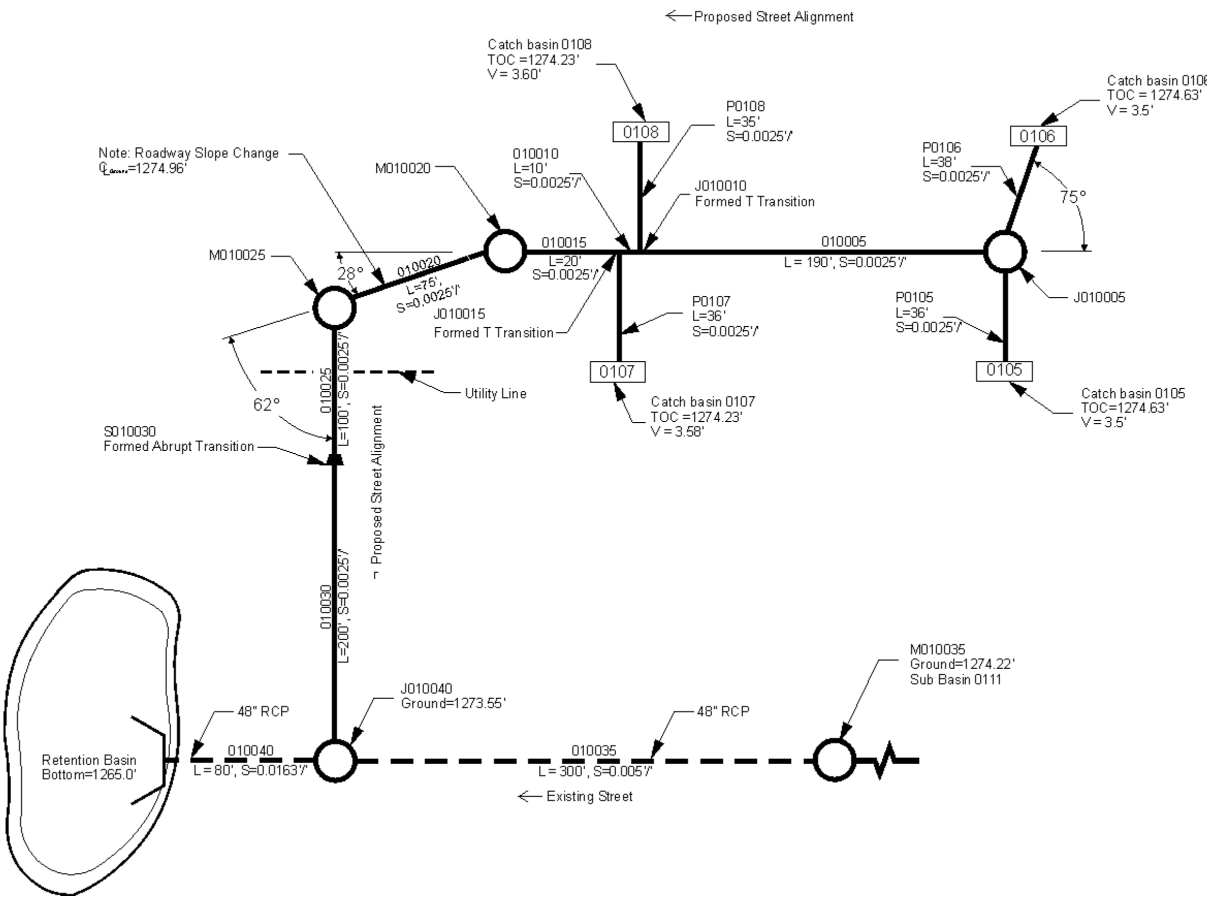
4.6 DESIGN EXAMPLE

OBJECTIVE:

Design and evaluation of an existing and proposed storm drain system.

PROBLEM STATEMENT:

FIGURE 4.11
INITIAL STORM DRAIN LAYOUT FOR EXAMPLE PROBLEM



Determine the initial and final system design of the initial storm drain layout presented in Figure 4.11 by calculating peak design discharge, size of storm drain pipes required and the associated hydraulic and energy grade lines (HGL and EGL respectively). Storm drain segments 010005 through 010030 represent the proposed storm drain system whereas storm drain segments 010035 and 010040 represent the existing storm drain.

GIVEN:

1. Minimum connector pipe size is equal to 15 inch diameter.
2. Minimum storm drain pipe size is equal to 18 inch diameter.
3. Design event is the 2-year storm.
4. Drainage areas, runoff coefficients, and rainfall intensity to be used for estimating peak discharges for Catch Basins 0105, 0106, 0107, 0108, and flow in storm drain segment 010035 are listed in Table E.1.
5. Regional retention basin bottom elevation = 1265.0 ft
6. Pipe 010040's inlet invert elevation and Junction J010040's outlet elevation = 1266.8 ft
7. 18 inch minimum cover required over pipe (18 inch + pipe wall thickness).
8. The design requirement for the catch basin HGL elevation is at least 1 ft below the catch basin inlet elevation (freeboard is 1 ft).
9. At M010025 and M010020, no special shaping for bends are proposed.
10. There is an existing buried utility between M010025 to S010030. The client has requested that the storm drain be sized to travel over the existing utility. The maximum pipe diameter available to use in these sections is a 18 inch pipe, which will safely lay over the utility without having to go under the utility line. There is a formed abrupt transition at S010030.
11. Use *City of Phoenix Standard Drawings (2005)* for Catch Basin Type.
12. Initial storm drain size and estimated travel time between concentration points assumed full conditions.
13. Storm drain outlet pipe invert elevation at retention basin is set 0.5 ft above basin bottom.

SOLUTION:

Step 1. Compute Peak Discharge at Catch Basins

Given the hydrologic parameters listed in Table E.1 calculate the peak discharge at catch basin locations. Utilizing the estimated peak discharges, determine inlet capacities and inlet dimensions per Chapter 3, Street Drainage. Parameters for estimating inlet capacities and dimensions are listed in Table E.2. Results for sizing inlets are also listed in Table E.2.

TABLE E.1
SUBBASIN PARAMETERS

Sub Basin							
Sub Basin ID	Area	Runoff Coefficient	Length	Slope	Time of Concentration	Rainfall Intensity (2YR)	Sub Basin Runoff
	(acres)		(ft)	(ft/ft)	(min)	(in/hr)	(cfs)
1	2	3	4	5	6	7	8
0105	0.47	0.91	390	0.0077	10.00	2.80	1.20
0106	0.86	0.91	1000	0.0170	10.00	2.80	2.20
0108	0.67	0.91	510	0.0059	10.00	2.80	1.70
0107	0.47	0.91	450	0.0111	10.00	2.80	1.20
0111	50.00	0.85	1350	0.0074	15.00	2.40	102.00

TABLE E.2
STREET AND INLET PARAMETERS AND DIMENSIONS

Street and Inlet																
Inlet ID	Station	Average Upstream Gutter Slope	Cross Slope at Inlet (Sx)	Flow by to Inlet (Column 25)	Total Flow to Inlet	Depth of Flow Upstream Of Inlet	Spread Of Flow In Street	Gutter Velocity	Depth Of Sump	Inlet Type [1]	Catchbasin Depth	Intercepted Flow	Flow By	Flow By To Inlet ID	Top of Curb Elev.	V, Sump Elev.
		(ft/ft)	(ft/ft)	(cfs)	(cfs)	(ft)	(ft)	(fps)	(ft)		(ft)	(cfs)	(cfs)		(ft)	(ft)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
0105	38+00 RT	0.0120	0.0200	0.00	1.20	0.20	6.79	2.38		P1569M1-3	3.50	0.80	0.40	0107	1274.73	1271.23
0106	34+10 LT	0.0091	0.0270	0.00	2.20	0.27	7.66	2.63		P1569M2-3,6	3.50	2.20			1274.74	1271.24
0108	33+50 RT	0.0095	0.0200	0.00	1.70	0.22	8.27	2.34		P1569M2-6,6	3.60	1.70			1274.23	1270.63
0107	28+50 RT	0.0095	0.0270	0.40	2.00	0.26	7.31	2.62	0.39	P1569M1-3	3.58	2.00			1274.23	1270.65

1. Inlet type identifications are from the City of Phoenix Standard Drawings (2005).

Step 2. Layout Initial Storm Drain System

Layout storm drain system and determine pipe lengths and slopes, the locations of manholes and junctions, preliminary pipe sizes and design peak discharge. The following steps relate the procedures utilized to layout the initial system.

2.1 Considering proposed catch basin, manholes and existing storm drain, a preliminary schematic of the storm drain system was drawn. Figure 4.11 displays the layout of the initial storm drain system.

2.2 Considering design constraints such as storm drain and connector pipe soffit elevations, and the invert elevation of catch basins (catch basin v depths) initial storm drain slopes are estimated.

- For this example an assumed hydraulic grade line (HGL) was estimated by taking into consideration, dimensions of the existing storm drain system, design criteria (listed under given) and ground elevations. Assuming that the system will be in full flow conditions, the hydraulic grade line of the proposed system at the junction with the existing system was set above the 48-inch pipe and approximately 2 feet below the surface. At the upstream end of the proposed system the assumed hydraulic grade line was set approximately 2 feet below the surface (setting a target elevation of 2 feet below the surface will help insure that there is at a minimum of 1 foot of freeboard below the catch basin inlet). A slope was then determined between the two points (slope = 0.0025 ft/ft). The assumed hydraulic grade line slope was then used as the initial pipe slope for the main line and for the connector pipes (an assumption was made that the connector pipe HGL slope will be the same as for the main line). Figure 4.13 displays a HGL profile for the initial storm drain layout.

- The initial profile (soffit profile) of the storm drain was laid out by matching the storm drain soffit of the proposed system to the soffit of the existing storm drain and utilizing the storm drain slope determined above and proceeded up stream to the beginning of the proposed system.

- The next step was to determine the initial soffit profile for the collector pipes and v depths of catch basins. To determine catch basin v depth and collector pipe soffit profiles, initial collector pipe size were estimated. Initial collector pipe size was estimated utilizing the following steps and assumptions:

- Assume full flow.

- Greatest design peak discharge for connector pipes = 2.20 cfs.

- Connector pipe slope = 0.0025 ft/ft (slope is the same for all connector pipes).

- Manning's Roughness (n) = 0.013.

- For pipes flowing full, pipe diameter can be estimated utilizing the following equation:

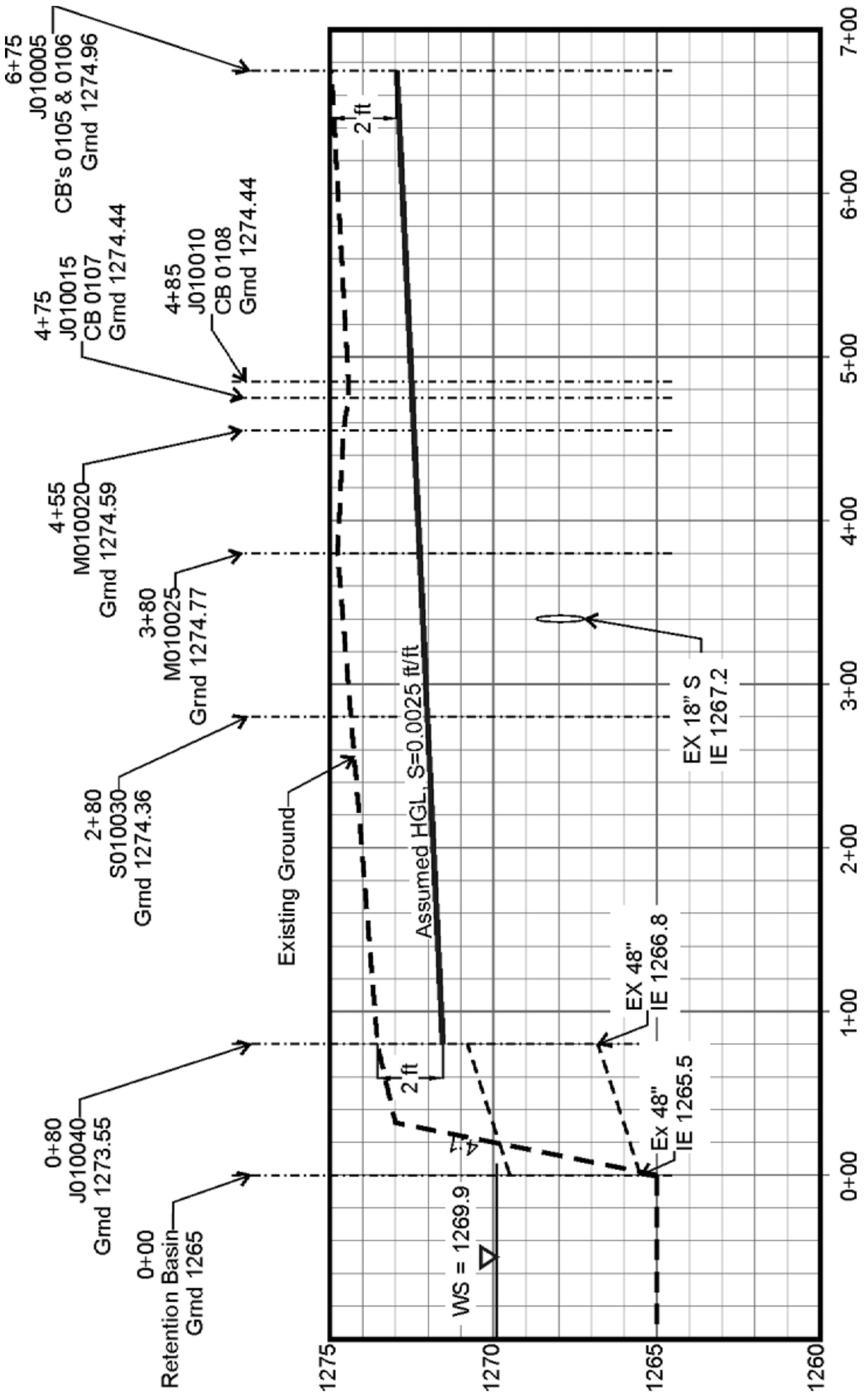
$$D = 1.33\left(\frac{nQ}{\sqrt{S}}\right)^{3/8}$$

$$D = 1.33\left(\frac{(0.013)(2.20)}{\sqrt{0.0025}}\right)^{3/8} = 1.07\ feet$$

$D = 12.80\ inches$ - use minimum pipe diameter of 15 inches for all connector pipes.

- Catch basin v depth is a catch basin dimension cited in the Uniform Standard Details for Public Works Construction (Maricopa Association of Governments, 1998) measured from the top of curb to the catch basin invert. Minimum v depth is typically 3.5 feet. In this example the v depth for catch basins were set so that the initial collector slope of 0.0025 ft/ft between the catch basin and the main line storm drain could be obtained (soffit elevations of main line and collector pipe are matched).

FIGURE 4.12
HGL PROFILE FOR INITIAL STORM DRAIN LAYOUT



2.3 The peak discharge at J010005 was calculated utilizing the following steps (refer to Table E.3):

2.3.1 Sum contributing drainage areas to catch basins 0105 and 0106.

$$A_T = 1.33 \text{ acres}$$

2.3.2 Calculate the weighted runoff coefficient, C_w:

$$C_w = \frac{A_1 C_1 + A_2 C_2 + \ldots + A_{n+1} C_{n+1}}{A_1 + A_2 + \ldots A_{n+1}}$$

$$C_w = \frac{(0.47)(0.91) + (0.86)(0.91)}{0.47 + 0.86} = 0.91$$

2.3.3 Using the longest time of concentration (T_c = 10 minutes), for the contributing areas, and the 2-year design storm, determine rainfall intensity from rainfall intensity-duration-frequency relation graphs provided in the Hydrology Manual.

$$i = 2.8 \text{ in/hr}$$

2.3.4 Determine the design peak discharge, Q_d, using the Rational Method:

$$Q_d = C_w i A_T$$

$$Q_d = (0.91)(2.80)(1.33) = \underline{\mathbf{3.4\ cfs}}.$$

2.4 The initial storm drain size for storm drain segment 010005 was estimated utilizing the following steps:

2.4.1 Calculate the initial size of storm drain segment 010005 using the peak discharges determined in Step 2.3.4, pipe slope estimated in Step 2.2, the assumption that the pipe is flowing full and the following equation:

$$D = 1.33\left(\frac{nQ}{\sqrt{S}}\right)^{3/8}$$

$$D = 1.33\left(\frac{(0.013)(3.4)}{\sqrt{0.0025}}\right)^{3/8} = 1.27\ feet$$

$D = 15.2\ inches$ - use minimum pipe diameter of 18 inches.

Assuming full flow, velocity $= \frac{Q}{A} = \frac{3.4}{1.77} = 1.92\ ft/\text{sec}$

Velocity is less than desired 5 ft/sec cleansing velocity. Check that the velocity of flow from one half the design peak discharge is greater than the minimum velocity of 2 ft/sec (criteria may vary from community to community). To check for minimum velocity a computer program was utilized to facilitate a solution using Manning's equation. Estimated pipe size, slope, and one half the design peak discharge was used to estimate velocity.

Velocity at half design peak discharge = 2.65 ft/sec.

2.5 The estimated travel time in storm drain segment 010005 was determined utilizing the following steps:

2.5.1 Using the velocity, V, calculated in Step 2.4, determine the travel time (T_{cd}) between J010010 and J010005 using the following equation:

$$T_{cd} = \frac{L}{V\frac{60}{m}}$$

where:

T_{cd} = time in drain, ft
V = velocity, ft/sec
$60/m$ = 60 sec per 1 min

Travel time = $\frac{190\ ft}{(1.92\ \text{ft/sec})(60\ \text{sec/min})}$ = **<u>1.65 min</u>**.

2.6 Calculate peak discharge at J010010.

2.6.1 Sum contributing drainage areas to catch basins 0105, 0106 and 0108.

A_T = acres

2.6.2 Calculate the weighted runoff coefficient, C_w using procedures listed in Step 2.3.2.

$$C_w = 0.91$$

2.6.3 Using the longest time of concentration, T_c = 11.65 minutes, (T_c from Step 2.3.3 plus the travel time from Step 2.5.1 as compared to the T_c estimated for the contributing drainage area to Catch Basin 108), and the 2-year design storm, determine rainfall intensity from rainfall intensity-duration-frequency relation graphs provided in the Hydrology Manual.

i = 2.6 in/hr

2.6.4 Determine the design peak discharge, Q_d, using the Rational Method:

$$Q_d = C_w i A_T$$

$$Q_d = (0.91)(2.60)(2.00) =$$ **4.7 cfs**.

2.7 The initial storm drain size for storm drain segment 010010 was estimated utilizing the following steps:

2.7.1 Calculate the preliminary size of storm drain segment 010010 using the peak discharges determined in Step 2.6.4, pipe slope estimated in Step 2.2 and procedures presented in Step 2.4. The selected pipe size shall be sufficient to convey the design peak discharge.

$D = 18$ inches

$V = 2.65$ ft/sec

Velocity at half design peak discharge = 2.9 ft/sec

2.8 Using procedures in Step 2.5 estimate travel time for storm drain segment 010010.

2.8.1 Travel time = 0.06 min

2.9 Calculate peak discharge at J010015.

2.9.1 Sum contributing drainage areas to catch basins 0105, 0106, 0107 and 0108.

$A_T = 2.47$ acres.

2.9.2 Calculate the weighted runoff coefficient, C_w using procedures listed in Step 2.3.2.

$$C_w = 0.91$$

2.9.3 Using the longest time of concentration, T_c = 11.71 minutes (T_c from Step 2.3.3 plus the travel time from Step 2.5.1 and 2.8.1 as compared to the T_c estimated for the contributing drainage area to Catch Basin 0107), and the 2-year design storm, determine rainfall intensity from rainfall intensity-duration-frequency relation graphs provided in the Hydrology Manual.

$i = 2.6$ in/hr

2.9.4 Determine the design peak discharge, Q_d, using the Rational Method:

$$Q_d = C_w i A_T$$

$Q_d = (0.91)(2.60)(2.47)$ = **5.8 cfs**.

2.10 The initial storm drain size for storm drain segments 010015, 010020, 010025 and 010030 was estimated using the following steps:

2.10.1 There are no inlets draining to the storm drain segment 010015, 010020, 010025 and 010030 therefore use the peak discharge determined in Step 2.9, pipe slope estimated in Step 2.2 and procedures presented in Step 2.4.

Results of the evaluation are presented below.

Pipe Segment	Pipe Size, inches	Capacity	Velocity, ft/sec	Velocity at 1/2 Design Peak, ft/sec
010015	18[1]	Full Flow	3.3	3.0
010020	18[1]	Full Flow	3.3	3.0
010025	18[1]	Full Flow	3.3	3.0
010030	24	Full Flow (Assume)	1.9	3.0

1. 18 inch diameter pipe is used to avoid conflicts with existing utilities. Appropriateness of the 18 inch diameter pipe will be checked while determining the hydraulic grade line for the system.

Step 3. Estimate design peak discharge for storm drain segments 010035 and 010040 (existing storm drain).

3.1 Estimate design peak discharge for storm drain segment 010035.

3.1.1 Sub-basin 0111 drains to pipe segment 010035. Design discharge for segment 010035 was estimated in Step 1.

Q_d = **102.0 cfs**

3.2 To determine the longest time of concentration for the overall drainage area draining to storm drain segment 010040 (existing storm drain drainage area versus proposed storm drain drainage area) the flow travel time for storm drain segments 010015, 010020, 010025, 010030 and 010035 needs to be determined. Using procedures in Step 2.5.1 estimate travel time for storm drain segments and then the respective time of concentration for each storm drain segment.

3.2.1 Proposed Storm drain segment:

3.2.1.1 Summation of travel time for storm drain segments 010015, 010020, 010025, and 010030 = 2.80 minutes

3.2.1.2 Add above flow travel time to time of concentration estimated in Step 2.9.3.

Time of concentration = 14.41 minutes.

3.2.2 Existing 48 inch storm drain segment 010035.

3.2.2.1 Travel time for storm drain segment 010035 = 0.62 minutes.

3.2.2.2 Add above flow travel time to time of concentration estimated in Step 1 (Sub-basin 0111 = 15.00 minutes).

Time of concentration = 15.62 minutes.

3.3 Calculate peak discharge at J010040.

3.3.1 Sum contributing drainage areas to catch basins 0105, 0106, 0107, 0108 and Sub-basin 0111.

$A_T = 52.47$ acres

3.3.2 Calculate the weighted runoff coefficient, C_w using procedures listed in Step 2.3.2.

$C_w = 0.853$

3.3.3 Using the longest time of concentration, T_c = 15.62 minutes (controlling T_c from Step 43.2), and the 2-year design storm, determine the rainfall intensity from rainfall intensity-duration-frequency relation graphs provided in the Hydrology Manual.

$i = 2.35$ in/hr

3.3.4 Determine the design peak discharge, Q_d, using the Rational Method:

$Q_d = C_w i A_T$

$Q_d = (0.853)(2.35)(52.47)$ = **105.2 cfs**.

TABLE E.3
MAINLINE DESIGN DISCHARGE AND PIPE PARAMETERS

Main Storm Drain														
Runoff/Mainline Discharge							Storm Drain - Normal Flow							
Conveyance ID	Contributing Area (A_T)	Composite C (C_w)	Composite Area (CA)	Time of Concentration (min)	Rainfall Intensity (in/hr)	Design Discharge (cfs)	Invert Elevation (Inlet) (ft)	Invert Elevation (Outlet) (ft)	Size (in)	Slope (ft/ft)	Velocity (fps)	Length (ft)	Time in Drain (min)	Manning's n
1	2	3	4	5	6	7	8	9	8	9	10	11	12	13
010005	1.33	0.91	1.21	**10.00**	2.80	**3.40**	1270.79	1270.31	**1 - 18 Pipe**	**0.0025**	1.92	190	**1.65**	0.013
010010	2.00	0.91	1.82	**11.65**	2.60	**4.70**	1270.31	1270.29	**1 - 18 Pipe**	**0.0025**	2.66	10	**0.06**	0.013
010015	2.47	0.91	2.25	**11.71**	2.60	**5.80**	1270.29	1270.24	**1 - 18 Pipe**	**0.0025**	3.28	20	**0.10**	0.013
010020	2.47	0.91	2.25	**11.71**	2.60	**5.80**	1270.24	1270.05	**1 - 18 Pipe**	**0.0025**	3.28	75	**0.38**	0.013
010025	2.47	0.91	2.25	**11.71**	2.60	**5.80**	1270.05	1269.80	**1 - 18 Pipe**	**0.0025**	3.28	100	**0.51**	0.013
010030	2.47	0.91	2.25	**11.71**	2.60	**5.80**	1269.30	1268.80	**1 - 24 Pipe**	**0.0025**	1.85	200	**1.81**	0.013
010035	50.00	0.85	42.50	**15.00**	2.40	**102.00**	1268.30	1266.80	**1 - 48 Pipe**	**0.0050**	8.12	300	**0.62**	0.013
010040	52.47	0.85	44.76	**15.62**	2.35	**105.20**	1266.80	1265.50	**1 - 48 Pipe**	**0.0163**	8.37	80		0.013

Step 4. Hydraulic Grade Line Evaluation Procedure

Design peak discharges and initial pipe sizes to be used in calculating the hydraulic grade line for the proposed and existing storm drain system have been determined in Steps 2 and 3. The next step is to set up a calculation sheet to aid in the hydraulic grade line calculations.

The general procedure for hydraulic grade line calculations is to establish the downstream control elevation and proceed upstream from one point of interest to another point of interest (i.e. from one junction to another, from one junction to a structure or from one junction to the beginning of a bend).

Table E.3 is an example of a hydraulic grade line calculation sheet. The calculation sheet aids the designer in keeping data organized. In this example the first row of data, is for storm drain segment 010005. In descending order, each following row lists data in a downstream direction for each storm drain segment. Since the proposed storm drain (storm drain segment 010035) connects into an existing storm drain, a row should be left blank to separate the data for the two storm drain systems. The row following the blank row is for storm drain segment 010035, the next row for storm drain segment 010040 and the last row for the analysis is for the outlet.

The hydraulic grade line computational procedure is as follows:

4.1 Calculate starting/controlling water surface elevation.

Review stormwater storage facilities requirements in Chapter 9. Tailwater conditions are a function of the storage-discharge relationship of the given facility.

4.1.1 This example uses a maximum ponding depth of 4.9 ft

4.1.2 Estimated starting water surface elevation (refer to Section 4.4.1 of the drainage design manual) = 1265.0 + 4.90 = **1269.90 ft**

4.2 Calculate outlet headloss.

4.2.1 Soffit elevation at outlet pipe is = 1265.5+ 4.0 = **1269.50 ft**. The starting water surface elevation is greater then the soffit elevation at the outlet, therefore use full flow conditions.

4.2.2 Using Equation (4.16) calculate the headloss at the outlet.

$$h_o = \frac{V_o^2}{2g} \tag{4.16}$$

$$V_o = \frac{Q}{A}$$

$$A = \pi\frac{D^2}{4}$$

$$A = \pi\frac{4^2}{4} = 12.57 \text{ sq ft}$$

$$V_o = \frac{105.2}{12.57} = 8.37 \text{ ft/sec}$$

$$h_o = \frac{8.37^2}{(2)(32.2)} = \mathbf{1.09\ ft}$$

4.2.3 Enter the headloss of 1.09 ft in the appropriate column of the calculation sheet.

4.2.4 Sum the headlosses for the storm drain segment, and calculate the hydraulic and energy grade lines and list in the appropriate column.

HGL = 1269.90 ft + 1.09 ft (exit loss) = **1270.99**
EGL = 1269.90 ft + 1.09 ft (exit loss) + 1.09 ft (velocity head) = **1272.08 ft**

4.3 Calculate headlosses for storm drain segment 010040.

4.3.1 Using Equation (4.4), calculate the friction slope:

$$S_f = K\frac{V^2}{2gR^{4/3}} \tag{4.4}$$

Calculate the *K* value using Equation (4.5):

where:

n (Manning's Roughness) = 0.013

g = acceleration due to gravity = 32.2 ft/sec^2

$$K = \frac{2gn^2}{2.21} \tag{4.5}$$

$$K = \frac{(2)(32.2)(0.013)^2}{2.21} = 0.0049 \text{ ft}$$

Calculate the hydraulic radius:

$$R = \frac{A}{P} \text{ (Normal flow conditions),}$$

$$R = \frac{D}{P} \text{ (Full flow conditions)}$$

where:

A = area of flow in pipe, sq ft

P = wetted perimeter of pipe, ft

D = diameter of pipe, ft

Since storm drain segment 010040 is in full flow condition:

$$R = \frac{48/12}{4} = 1.0\,ft$$

Calculate the velocity:

$$V = \frac{Q}{A}$$

$$V_o = \frac{105.2}{12.57} = 8.37 \text{ ft/sec}$$

Solving Equation (4.4):

$$S_f = 0.0049\frac{8.37^2}{(2)(32.2)(1^{4/3})} = 0.0053\, ft/ft$$

4.3.2 The headloss due to friction is calculated using Equation (4.6):

$$h_f = S_f L \qquad (4.6)$$

$$h_f = (0.0053)(80)\text{ ft} = \underline{\mathbf{0.42\ ft}}$$

4.3.3 The HGL at the upstream end of pipe 010040 (downstream end of J010040) is equal to (Tailwater Elevation) + (headloss due to friction for pipe segment 010040).

$$HGL = 1270.99\, ft + 0.42\, ft = \underline{\mathbf{1271.41\ ft}}$$

4.3.4 Calculate junction headloss at structure J010040.

4.3.4.1 The first step is to determine if the HGL is above or below the soffit at the junction. If the HGL is above the soffit proceed assuming full flow conditions. Verify by checking that the HGL at the upstream end of pipe segment 010040 is high enough to inundate the junction soffit.

Storm drain segment 010040's soffit elevation at junction J010040 ≈1266.80 ft (invert elevation) + 4 ft (pipe diameter) = **1270.80 ft**

From Step 4.3.3 HGL at downstream end of J010040 = **1271.41 ft**

Assume full flow conditions.

4.3.4.2 Calculate junction loss utilizing Equation (4.10b).

$$h_j = \frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3Cos\theta°)}{(A_1 + A_2)g} + \frac{V_1^2}{2g} - \frac{V_2^2}{2g} \qquad (4.10b)$$

where:

$Q_2 = Q_{010040}$ = design peak discharge for storm drain segment 010040 = 105.2 cfs

$Q_1 = Q_{010035}$ =difference in design peak discharge between storm drain segments 010040 and 010030 = 99.4 cfs

$Q_3 = Q_{010030}$ = design peak discharge for storm drain segment 010030 = 5.8 cfs

$A_2 = A_{010040}$ = full flow area for storm drain segment 010040 = 12.57 sq ft

$A_1 = A_{010035}$ = full flow area for storm drain segment 010035 = 12.57 sq ft

$A_3 = A_{010030}$ = full flow area for storm drain segment 010030 = 3.14 sq ft

$V_2 = V_{010040}$ = full flow velocity for Q_{010040} = 8.37 ft/sec

$V_1 = V_{010035}$ = full flow velocity for Q_{010035} = 7.91 ft/sec

$V_3 = V_{010030}$ = full flow velocity for Q_{010030} = 1.85 ft/sec

$$h_j = \frac{(2)((105.2)(8.37) - (99.4)(7.91) - (5.8)(1.85)(\cos 90°))}{(12.57 + 12.57)(32.2)} + \frac{7.91^2}{(2)(32.2)} - \frac{8.37^2}{(2)(32.2)}$$

h_j = **<u>0.11 ft</u>**

4.3.5 Record friction and junction headlosses and summation of headlosses for storm drain segment 010040 in appropriate calculation sheet columns. Calculate HGL and EGL and record.

Total Head Loss = 0.42 ft + 0.11 ft = **0.53 ft**
Downstream HGL Elevation = **1270.99 ft**
Upstream HGL Elevation = **1271.52 ft**
Downstream EGL Elevation = **1272.08 ft**
Upstream EGL Elevation = **1272.61 ft**
Upstream Soffit Elevation = 1266.80 ft + 4.0 ft = **1270.80 ft**

4.4 Calculate headlosses for storm drain segment 010035.

4.4.1 Using procedures from Step 4.3.1 and 4.32 calculate friction slope and headloss for storm drain segment 010035. Based on HGL elevation at J010040 storm drain segment 010035 starts in full flow.

$$S_f = 0.0049 \frac{8.12^2}{(2)(32.2)(1^{4/3})}$$ = **<u>0.0050 ft/ft</u>**

The headloss due to friction is calculated using Equation (4.6):

$$h_f = S_f L \qquad (4.6)$$

$h_f = (0.0050)(300)\, ft$ = **<u>1.50 ft</u>**

4.4.2 Calculate manhole headlosses at M010035 using Equation (4.11).

$$h_{mh} = 0.05\frac{V^2}{2g} \quad (4.11)$$

$$h_{mh} = 0.05\frac{8.12^2}{(2)(32.2)} = \underline{\mathbf{0.05\ ft}}$$

4.4.3 Record friction and manhole headlosses and summation of headlosses for storm drain segment 010035 in appropriate calculation sheet columns. Calculate HGL and EGL elevations and record.

Total Head Loss = 1.50 ft + 0.05 ft = **1.55 ft**
Downstream HGL Elevation = **1271.52 ft**
Upstream HGL Elevation = **1273.07 ft**
Downstream EGL Elevation = **1272.61 ft**
Upstream EGL Elevation = **1274.09 ft**
Upstream Soffit Elevation = 1268.30 ft+ 4.0 ft = **1272.30 ft**

4.5 Start HGL calculations for storm segment 010030 (proposed storm drain segment). HGL for proposed storm drain segment commences at J010040 with an HGL elevation of 1271.52 ft determined in step 4.3.5. Full flow conditions exist.

4.5.1 Using procedures from Step 4.3.1 and 4.32 calculate friction slope and headloss for storm drain segment 010030. Based on HGL elevation at J010040 and the upstream soffit elevation, storm drain segment 010030 starts in full flow.

$$S_f = 0.0049\frac{1.85^2}{(2)(32.2)(0.5^{4/3})} = 0.0007\ ft/ft$$

The headloss due to friction is calculated using Equation (4.6):

$$h_f = S_f L \quad (4.6)$$

$$h_f = (0.0007)(200)\ ft = \underline{\mathbf{0.14\ ft}}$$

4.5.2 Calculate transition headlosses at S010030. Structure S010030 is an abrupt transition (expansion) from a 18 inch pipe (010025) to a 24 inch pipe (010030).

Determine if the HGL elevation at the inlet of storm drain segment 010030 indicates that the storm drain is flowing full. In the example, pipes, 010030 and 010025 are flowing full (pressure flow conditions).

4.5.2.1 Calculate the transition headloss using Equation (4.9) and Table 4.3b:

$$h_t = k_e\frac{V^2}{2g} \quad (4.9)$$

4.5.2.2 To use Table 4.3b, first calculate $\frac{D_2}{D_1} = \frac{24}{18} = 1.33$ (say 1.4)

where:

D_1 is equal to the upstream pipe diameter.
D_2 is equal to the downstream pipe diameter.

Second, calculate the smallest pipe segment velocity (storm drain segment 010025 is estimated to be flowing full).

$$V_{010025} = \frac{Q_{010025}}{A_{010025}}$$

$$V_{010025} = \frac{5.8\ cfs}{1.77\ ft^2} = \underline{\mathbf{3.28\ ft/sec}}$$

Use Table 4.3b to determination of the sudden expansion coefficient:

k_e = 0.25 (hand calculated)
k_e = 0.20 (4-way interpolation, used for this example)

4.5.2.3 Calculate the transition headloss:

$$h_t = k_e \frac{V_{010025}^{\ 2}}{(2)(32.2)}$$

$$h_t = 0.20 \frac{3.28^2}{(2)(32.2)} = \underline{\mathbf{0.03\ ft}}$$

4.5.3 Record friction and transition headlosses and summation of headlosses for storm drain segment 010030, in appropriate calculation sheet columns. Calculate HGL and EGL elevations and record.

Total Head Loss = 0.14 ft + 0.03 ft = **0.17 ft**
Downstream HGL Elevation = **1271.52 ft** (refer to Step 4.5)
Upstream HGL Elevation = **1271.69 ft**
Downstream EGL Elevation = **1272.61 ft**
Upstream EGL Elevation = **1271.74 ft**
Upstream Soffit Elevation = 1269.30 ft + 2.0 ft = **1271.30 ft**

4.6 Calculate headlosses for storm drain segment 010025.

4.6.1 Using procedures from Step 4.3.1 and 4.3.2 calculate friction slope and headloss for storm drain segment 010025. Based on HGL elevation at S010030 storm drain segment 010025 starts in full flow.

$$S_f = 0.0049\frac{3.28^2}{(2)(32.2)(0.375^{4/3})} = \underline{\mathbf{0.0030\ ft/ft}}$$

The headloss due to friction is calculated using Equation (4.6):

$$h_f = S_f L \qquad (4.6)$$

$$h_f = (0.0030)(100\ ft) = \underline{\mathbf{0.30\ ft}}$$

4.6.2 Calculate manhole bend losses at M010025. Manhole M010025 is used to turn/ bend flow from pipe 010020 to pipe 010025. Use Figure 4.10 to estimate bend loss coefficient and Equation (4.12) to estimate headloss due to a bend.

4.6.2.1 Using the results of Step 4.6.1 calculate HGL elevation immediately downstream of M010025 to determine if the manhole is in pressure flow. In the example, pipes, 010025 and 010020 are flowing full.

4.6.2.2 Using bend angle identified in the schematic (62 degrees) and Figure 4.10, determine the bend loss coefficient (k_b):

k_b = 0.69 (hand calculated)

k_b = 0.68 (4-way interpolation, used for this example)

4.6.2.3 Calculate the bend loss at manhole headloss using Equation (4.12):

$$h_{mh} = k_b\frac{V^2}{2g} \qquad (4.12)$$

$$V_{010020} = \frac{Q_{010020}}{A_{010020}}$$

$$V_{010020} = \frac{5.8\ cfs}{1.77\ ft^2} = \underline{\mathbf{3.28\ ft/sec}}$$

$$h_{mh} = k_b\frac{V_{010020}^2}{(2)(32.2)}$$

$$h_{mh} = 0.68\frac{3.28^2}{(2)(32.2)} = \underline{\mathbf{0.11\ ft}}$$

4.6.3 Record friction and manhole bend headlosses and summation of headlosses for storm drain segment 010025, in appropriate calculation sheet columns. Calculate HGL and EGL elevations and record.

Total Head Loss = 0.30 ft + 0.11 ft = **0.41 ft**
Downstream HGL Elevation = **1271.69 ft**
Upstream HGL Elevation = **1272.10 ft**

Downstream EGL Elevation = **1271.74 ft**
Upstream EGL Elevation = **1272.27 ft**
Upstream Soffit Elevation = 1270.05 ft + 1.50 ft = **1271.55 ft**

4.7 Calculate headlosses for storm drain segment 010020.

4.7.1 Using procedures from Step 4.3.1 and 4.3.2 calculate friction slope and headloss for storm drain segment 010020. Based on HGL elevation at M010025, storm drain segment 010020 starts in full flow.

$$S_f = 0.0049\frac{3.28^2}{(2)(32.2)(0.375^{4/3})} = \underline{\mathbf{0.0030\ ft/ft}}$$

The headloss due to friction is calculated using Equation (4.6):

$$h_f = S_f L \qquad (4.6)$$

$$h_f = (0.0030)(75\,ft) = \underline{\mathbf{0.23\ ft}}$$

4.7.2 Calculate manhole bend losses at M010020. Manhole M010020 is used to turn/bend flow from storm drain segment 010020 to storm drain segment 010015. Use Figure 4.10 to estimate bend loss coefficient and Equation (4.12) to estimate headloss due to a bend.

4.7.2.1 Using the results of step 4.7.1, calculate HGL elevation immediately downstream of M010020 to determine if the manhole is in pressure flow. In the example storm drain segments, 010020 and 010015 are flowing full.

4.7.2.2 Using bend angle identified in the schematic (28 degrees) and Figure 4.10, determine the bend loss coefficient (k_b):

$$(k_b) = 0.19$$

4.7.2.3 Calculate the manhole headloss using Equation (4.12):

$$h_{mh} = k_b\frac{V^2}{2g} \qquad (4.12)$$

$$V_{010015} = \frac{Q_{010015}}{A_{010015}}$$

$$V_{010015} = \frac{5.8\,cfs}{1.77\,ft^2} = \underline{\mathbf{3.28\ ft/sec}}$$

$$h_{mh} = k_b\frac{{V_{010015}}^2}{(2)(32.2)}$$

$$h_{mh} = 0.19\frac{3.28^2}{(2)(32.2)} = \underline{\mathbf{0.03\ ft}}$$

4.7.3 Record friction and manhole bend headlosses and summation of headlosses for storm drain segment 010020, in appropriate calculation sheet columns. Calculate HGL and EGL elevations and record.

Total Head Loss = 0.23 ft + 0.03 ft = **0.26 ft**
Downstream HGL Elevation = **1272.10 ft**
Upstream HGL Elevation = **1272.36 ft**
Downstream EGL Elevation = **1272.27 ft**
Upstream EGL Elevation = **1272.53 ft**
Upstream Soffit Elevation = 1270.24 ft + 1.50 ft = **1271.74 ft**

4.8 Calculate headlosses for storm drain segment 010015.

4.8.1 Using procedures from Step 4.3.1 and 4.3.2 calculate friction slope and headloss for storm drain segment 010015. Based on HGL elevation at M010020 storm drain segment 010015 starts in full flow.

$$S_f = 0.0049\frac{3.28^2}{(2)(32.2)(0.375^{4/3})} = \underline{\mathbf{0.0030\ ft/ft}}$$

The headloss due to friction is calculated using Equation (4.6):

$$h_f = S_f L \qquad (4.6)$$

$$h_f = (0.0030) \times (20\ ft) = \underline{\mathbf{0.06\ ft}}$$

4.8.2 Calculate junction headloss at structure J010015.

4.8.2.1 The first step, is to determine if the HGL is above or below the soffit at the junction. If the HGL is above the soffit, proceed assuming full flow conditions. Verify by checking that the HGL at the upstream end of pipe segment 010015 is high enough to inundate the junction soffit.

Storm drain segment, 010010's soffit elevation at junction J010015 = 1270.29 ft + 1.5 ft (pipe diameter) = **1271.79 ft**.

From Step 4.7.3 and 4.8.1, the HGL at downstream end of J010015 = 1272.36 ft + 0.06 ft = **1272.42 ft**. Junction is submerged. Assume full flow conditions.

4.8.2.2 Calculate junction loss utilizing Equation (4.10b).

$$h_j = \frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3Cos\theta^\circ)}{(A_1 + A_2)g} + \frac{V_1^2}{2g} - \frac{V_2^2}{2g} \qquad (4.10b)$$

where:

$Q_2 = Q_{010015}$ = design peak discharge for storm drain segment 010015 = 5.8 cfs

$Q_1 = Q_{010010}$ = design peak discharge for storm drain segment 010010 = 4.7 cfs

$Q_3 = Q_{P0107}$ = difference in design peak discharge between storm drain segments 010015 and 010010 = 1.6 cfs (peak flow at inlet 107 = 1.6 cfs)

$A_2 = A_{010015}$ = full flow area for storm drain segment 010015 = 1.77 sq ft

$A_1 = A_{010010}$ = full flow area for storm drain segment 010010 = 1.77 sq ft

$A_3 = A_{P0107}$ = full flow area for storm drain segment P0107 = 1.23 sq ft

$V_2 = V_{010015}$ = full flow velocity for Q_{010015} = 3.28 ft/sec

$V_1 = V_{010010}$ = full flow velocity for Q_{010010} = 2.66 ft/sec

$V_3 = V_{P0107}$ = full flow velocity for Q_{P0107} = 1.63 ft/sec

$$h_j = \frac{(2)((5.8)(3.28) - (4.7)(2.66) - (1.6)(1.63)(\cos 90°))}{(1.77 + 1.77)(32.2)} + 0.11 - 0.17$$

h_j = **<u>0.05 ft</u>**

4.8.3 Record friction and junction headlosses and summation of headlosses for storm drain segment 010015 in appropriate calculation sheet columns. Calculate HGL and EGL and record.

Total Head Loss = 0.06 ft + 0.05 ft = **0.11 ft**
Downstream HGL Elevation = **1272.36 ft**
Upstream HGL Elevation = **1272.47 ft**
Downstream EGL Elevation = **1272.53 ft**
Upstream EGL Elevation = **1272.64 ft**
Upstream Soffit Elevation = 1270.29 ft + 1.50 ft = **1271.79 ft**

4.9 Calculate headlosses for storm drain segment 010010.

4.9.1 Using procedures from Step 4.3.1 and 4.3.2 calculate friction slope and headloss for storm drain segment 010010. Based on HGL elevation at J010015 storm drain segment 010010 starts in full flow.

$$S_f = 0.0049 \frac{2.66^2}{(2)(32.2)(0.375^{4/3})}$$ = **<u>0.0020 ft/ft</u>**

The headloss due to friction is calculated using Equation (4.6):

$$h_f = S_f L \qquad (4.6)$$

$$h_f = (0.0020)(10\,ft) = \underline{\mathbf{0.02\ ft}}$$

4.9.2 Calculate junction headloss at structure J010010.

4.9.2.1 The first step, is to determine if the HGL is above or below the soffit at the junction. If the HGL is above the soffit proceed assuming full flow conditions. Verify by checking that the HGL at the upstream end of pipe segment 010005 is high enough to inundate the junction soffit.

Storm drain segment 010005's soffit elevation at junction J010010 = 1270.31 ft + 1.5 ft (pipe diameter) = **1271.81 ft**

From Step 4.8.3 and 4.9.1 HGL at downstream end of J010010 = 1272.47 ft + 0.02 ft = **1272.49 ft**. The junction is submerged. Assume full flow conditions.

4.9.2.2 Calculate junction loss utilizing Equation (4.10b).

$$h_j = \frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3Cos\theta°)}{(A_1 + A_2)g} + \frac{V_1^2}{2g} - \frac{V_2^2}{2g} \qquad (4.10b)$$

where:

$Q_2 = Q_{010010}$ = design peak discharge for storm drain segment 010010 = 4.7 cfs

$Q_1 = Q_{010005}$ = difference in design peak discharge between storm drain segment 010010 and P0108 = 3.0 cfs

$Q_3 = Q_{P0108}$ = design peak discharge for storm drain segment P010 = 1.7 cfs

$A_2 = A_{010010}$ = full flow area for storm drain segment 010010 = 1.77 sq ft

$A_1 = A_{010005}$ = full flow area for storm drain segment 010005 = 1.77 sq ft

$A_3 = A_{P0108}$ = full flow area for storm drain segment P0108 = 1.23 sq ft

$V_2 = V_{010010}$ = full flow velocity for Q_{010010} = 2.66 ft/sec

$V_1 = V_{010005}$ = full flow velocity for Q_{010005} = 1.69 ft/sec

$V_3 = V_{P0108}$ = full flow velocity for Q_{P0108} = 1.38 ft/sec

$$h_j = \frac{2((4.7)(2.66) - (3.0)(1.69) - (1.7)(1.38)(\cos 90°))}{(1.77 + 1.77)(32.2)} + 0.04 - 0.11$$

h_j = **0.06 ft**

4.9.3 Record friction and junction headlosses and summation of headlosses for storm drain segment 010010 in appropriate calculation sheet columns. Calculate HGL and EGL and record.

Total Head Loss = 0.02 ft + 0.06 ft = **0.08 ft**
Downstream HGL Elevation = **1272.47 ft**
Upstream HGL Elevation = **1272.55 ft**
Downstream EGL Elevation = **1272.64 ft**
Upstream EGL Elevation = **1272.66 ft**
Upstream Soffit Elevation = 1270.31 ft + 1.5 ft = **1271.81 ft**

4.10 Calculate headlosses for storm drain segment 010005.

4.10.1 Using procedures from Step 4.3.1 and 4.3.2 calculate friction slope and headloss for storm drain segment 010005. Based on HGL elevation at J010010 storm drain segment 010005 starts in full flow.

$$S_f = 0.0049\frac{1.92^2}{(2)(32.2)(0.375^{4/3})}$$ = **0.0010 ft/ft**

The headloss due to friction is calculated using Equation (4.6):

$$h_f = S_f L \qquad (4.6)$$

$$h_f = (0.0010)(190\ ft)$$ = **0.19 ft**

4.10.2 Record friction and junction headlosses and summation of headlosses for storm drain segment 010005 in appropriate calculation sheet columns. Calculate HGL and EGL and record.

Total Head Loss = **0.19 ft**
Downstream HGL Elevation = **1272.55 ft**
Upstream HGL Elevation = **1272.74 ft**
Downstream EGL Elevation = **1272.66 ft**
Upstream EGL Elevation = **1272.80 ft**
Upstream Soffit Elevation = 1270.79 ft + 1.5 ft = **1272.29 ft**

Step 5. Connector Pipe Hydraulic Grade Line Evaluation Procedures

Design peak discharges and initial pipe sizes to be used in calculating the hydraulic grade line for proposed connector pipes draining catch basins have been determined in steps 1 and 2. Two types of headlosses are primarily associated with connector pipes segments, losses due to friction and inlet headlosses. The hydraulic grade line and energy grade line in the main storm drain at the junction is used as the starting HGL for the connector pipe. THe HGL and EGL at the upstream end of the connector pipe is computed using the velocity in the connector pipe.

5.1 Calculate headlosses for connector pipe P0107.

5.1.1 Determine starting water surface elevation. From Step 4.8.3 starting HGL at J010015 is equal to 1272.47 ft.

5.1.2 Using procedures from Step 4.3.1 and 4.3.2 calculate friction slope and headloss for connector pipe P0107. Based on HGL elevation at J010015 connector pipe P0107 starts in full flow. Note catch basin at 0107 also intercepts 0.4 cfs overflow from 0105 for a total interception of 1.6 cfs.

$$S_f = 0.0049\frac{1.30^2}{(2)(32.2)(0.31^{4/3})} = \underline{\mathbf{0.0006\ ft/ft}}$$

The headloss due to friction is calculated using Equation (4.6):

$$h_f = S_f L \qquad (4.6)$$

$$h_f = (0.0006)(36\ ft) = \underline{\mathbf{0.02\ ft}}$$

5.1.3 Calculate inlet headloss for inlet number 0107.

5.1.3.1 To calculate the inlet headloss for 0107, Table 5.1 from Chapter 5 must be used to determine the entrance loss coefficient. For the example the type of structure used to determine the coefficient was “Pipe, Concrete: headwall, square edge”, (k_{en}):

k_{en} = 0.5

5.1.3.2 Equation (4.17) is then used to calculate the headloss at the inlet:

$$h_i = (1 + k_{en})\frac{V^2}{2g} \qquad (4.17)$$

$$h_i = (1 + 0.5)\frac{1.30^2}{(2)(32.2)} = \underline{\mathbf{0.04\ ft}}$$

5.1.4 Record friction and inlet headlosses and summation of headlosses for connector pipe P0107 in appropriate calculation sheet (see Table E.5) columns. Calculate HGL and EGL and record. Check to verify that there is 1 ft of freeboard between the EGL and the inlet elevation.

Total Head Loss = 0.02 ft + 0.04 ft = **0.06 ft**
Downstream HGL Elevation = **1272.47 ft**
Upstream HGL Elevation = **1272.53 ft**
Downstream EGL Elevation = **1272.64 ft**

Upstream EGL Elevation = **1272.57 ft**
Curb opening inlet/gutter Elevation = **1273.73 ft**

5.2 Calculate headlosses for connector pipe P0108.

5.2.1 Determine starting water surface elevation. From Step 4.9.2 starting HGL at J010010 is equal to 1272.55 ft.

5.2.2 Using procedures from Step 4.3.1 and 4.3.2 calculate friction slope and headloss for connector pipe P0108. Based on HGL elevation at J010010 connector pipe P0108 starts in full flow.

$$S_f = 0.0049\frac{1.38^2}{(2)((32.2)(0.31^{4/3}))} = \underline{\mathbf{0.0007\ ft/ft}}$$

The headloss due to friction is calculated using Equation (4.6):

$$h_f = S_f L \qquad (4.6)$$

$$h_f = (0.0007)(35\,ft) = \underline{\mathbf{0.02\ ft}}$$

5.2.3 Calculate inlet headloss for inlet number 0108.

5.2.3.1 To calculate the inlet headloss for 0108, Table 5.1 from Chapter 5 must be used to determine the entrance loss coefficient. For the example the type of structure used to determine the coefficient was “Pipe, Concrete: headwall, square edge”, (k_{en}):

k_{en} = 0.5

5.2.3.2 Equation (4.17) is then used to calculate the headloss at the inlet:

$$h_i = (1 + k_{en})\frac{V^2}{2g} \qquad (4.17)$$

$$h_i = (1 + 0.5)\frac{1.38^2}{(2)(32.2)} = \underline{\mathbf{0.04\ ft}}$$

5.2.4 Record friction and inlet headlosses and summation of headlosses for connector pipe P0108 in appropriate calculation sheet (see Table E.5) columns. Calculate HGL and EGL and record.

Total Head Loss = 0.02 ft + 0.04 ft = **0.06 ft**
Downstream HGL Elevation = **1272.55 ft**
Upstream HGL Elevation = **1272.61 ft**
Downstream EGL Elevation = **1272.66 ft**

Upstream EGL Elevation = **1272.64 ft**
Curb opening inlet/gutter Elevation = **1273.73 ft**

5.3 Calculate headlosses for connector pipe P0106.

5.3.1 Determine starting water surface elevation. From Step 4.10.2 starting HGL at J010005 is equal to 1272.74 ft.

5.3.2 Using procedures from Step 4.3.1 and 4.3.2 calculate friction slope and headloss for connector pipe P0106. Based on HGL elevation at J010005 connector pipe P0106 starts in full flow.

$$S_f = 0.0049\frac{1.79^2}{(2)(32.2)(0.31^{4/3})} = \underline{\mathbf{0.0012\ ft/ft}}$$

The headloss due to friction is calculated using Equation (4.6):

$$h_f = S_f L \qquad (4.6)$$

$$h_f = (0.0012)(38\,ft) = \underline{\mathbf{0.05\ ft}}$$

5.3.3 Calculate inlet headloss for inlet number 0106.

5.3.3.1 To calculate the inlet headloss for 0106, Table 5.1 from Chapter 5 must be used to determine the entrance loss coefficient. For the example, the type of structure used to determine the coefficient was “Pipe, Concrete: headwall, square edge”, (k_{en}):

k_{en} = 0.5

5.3.3.2 Equation (4.17) is then used to calculate the headloss at the inlet:

$$h_i = (1 + k_{en})\frac{V^2}{2g} \qquad (4.17)$$

$$h_i = (1 + 0.5)\frac{1.79^2}{(2)(32.2)} = \underline{\mathbf{0.07\ ft}}$$

5.3.4 Record friction and inlet headlosses and summation of headlosses for connector pipe P0106 in appropriate calculation sheet (see Table E.5) columns. Calculate HGL and EGL and record.

Total Head Loss = 0.05 ft + 0.07 ft = **0.12 ft**
Downstream HGL Elevation = **1272.74 ft**
Upstream HGL Elevation = **1272.86 ft**
Downstream EGL Elevation = **1272.80 ft**
Upstream EGL Elevation = **1272.91 ft**
Curb opening inlet/gutter Elevation = **1274.24 ft**

5.4 Calculate headlosses for connector pipe P0105.

5.4.1 Determine starting water surface elevation. From Step 4.10.2 starting HGL at J010005 is equal to 1272.74 ft.

5.4.2 Using procedures from Step 4.3.1 and 4.3.2 calculate friction slope and headloss connector pipe P0105. Based on HGL elevation at J010005 connector pipe P0105 starts in full flow. Note catch basin at 0105 only intercepts 0.8 cfs.

$$S_f = 0.0049\frac{0.65^2}{(2)(32.2)(0.31^{4/3})} = \underline{\mathbf{0.0002\ ft/ft}}$$

The headloss due to friction is calculated using Equation (4.6):

$$h_f = S_f L \tag{4.6}$$

$$h_f = (0.0002) \times (36\,ft) = \underline{\mathbf{0.01\ ft}}$$

5.5 Calculate inlet headloss for inlet number 0105.

5.5.1 To calculate the inlet headloss for 0105, Table 5.1 from Chapter 5 must be used to determine the entrance loss coefficient. For the example the type of structure used to determine the coefficient was “Pipe, Concrete: headwall, square edge”, (k_{en}):

k_{en} = 0.5

5.5.2 Equation (4.17) is then used to calculate the headloss at the inlet:

$$h_i = (1 + k_{en})\frac{V^2}{2g} \tag{4.17}$$

$$h_i = (1 + 0.5)\frac{0.65^2}{(2)(32.2)} = \underline{\mathbf{0.01\ ft}}$$

5.5.3 Record friction and inlet headlosses and summation of headlosses for connector pipe P0105 in appropriate calculation sheet (see Table E.5) columns. Calculate HGL and EGL and record.

Total Head Loss = 0.00 ft + 0.01 ft = **0.01 ft**
Downstream HGL Elevation = **1272.74 ft**
Upstream HGL Elevation = **1272.76 ft**
Downstream EGL Elevation = **1272.80 ft**
Upstream EGL Elevation = **1272.77 ft**
Curb opening inlet/gutter Elevation = **1274.23 ft**

Step 6. Confirm adequate cover over all pipes and check freeboard at all catch basins, manholes and junctions.

Step 7. Complete Table E.4 **and** Table E.5.

SUMMARIZED RESULTS:

Hydraulic grade line calculation summary sheets are provided as Table E.4 and Table E.5. Figure 4.13 displays the final HGL profile calculated for the proposed storm drain.

FIGURE 4.13
FINAL HGL PROFILE FOR PROPOSED STORM DRAIN

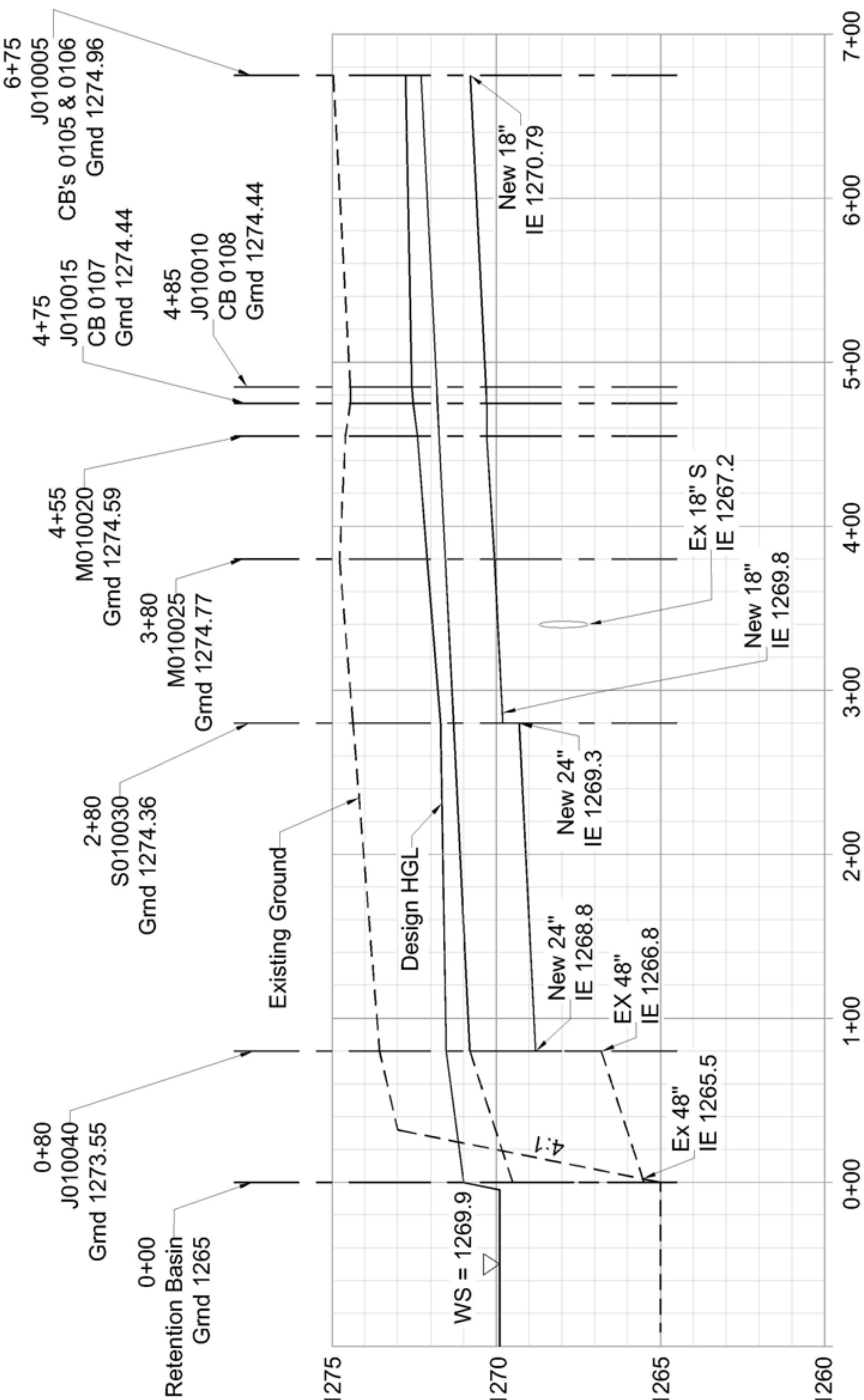


TABLE E.4
HYDRAULIC GRADE LINE CALCULATION SHEET

Conveyance ID	Hydraulic Data											
	Invert Elevation (Inlet) (ft)	Invert Elevation (Outlet) (ft)	Size (in)	Manning's n	Flow Area (ft^2)	Design Discharge (cfs)	Velocity (fps)	$v^2/2g$ (ft)	K	R (ft)	Slope (ft/ft)	S_f (ft/ft)
1	2	3	4	5	6	7	8	9	10	11	12	13
010005	**1270.79**	**1270.31**	**1 - 18 Pipe**	**0.013**	**1.77**	**3.40**	**1.92**	**0.06**	**0.0049**	**0.38**	**0.0025**	**0.0010**
010010	1270.31	1270.29	1 - 18 Pipe	0.013	1.77	4.70	2.66	0.11	0.0049	0.38	0.0025	0.0020
010015	1270.29	1270.24	1 - 18 Pipe	0.013	1.77	5.80	3.28	0.17	0.0049	0.38	0.0025	0.0030
010020	1270.24	1270.05	1 - 18 Pipe	0.013	1.77	5.80	3.28	0.17	0.0049	0.38	0.0025	0.0030
010025	1270.05	1269.80	1 - 18 Pipe	0.013	1.77	5.80	3.28	0.17	0.0049	0.38	0.0025	0.0030
010030	1269.30	1268.80	1 - 24 Pipe	0.013	3.14	5.80	1.85	0.05	0.0049	0.50	0.0025	0.0007
			Note: Begin Proposed Storm Drain HGL Evaluation									
010035	**1268.30**	**1266.80**	**1 - 48 Pipe**	**0.013**	**12.57**	**102.00**	**8.11**	**1.02**	**0.0049**	**1.00**	**0.0050**	**0.0050**
010040	**1266.80**	**1265.50**	**1 - 48 Pipe**	**0.013**	**12.57**	**105.20**	**8.37**	**1.09**	**0.0049**	**1.00**	**0.0163**	**0.0053**
OUTLET			Outlet									

Table E.4
Hydraulic Grade Line Calculation Sheet (Continued)

Conveyance ID	Hydraulic Data Cont.		Head Losses							HGL		EGL		Crown Elevation	
	Length	Average S_f	h_o	h_f	h_j	h_{mh}	h_b	h_t	Total Head Loss	Inlet	Outlet	Inlet	Outlet	Inlet	Outlet
	(ft)	(ft/ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
1	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28
010005	**190**			**0.19**					**0.19**	**1272.74**	**1272.55**	**1272.80**	**1272.66**	**1272.50**	**1272.02**
									0.00						
010010	10			0.02	0.06				0.08	1272.55	1272.47	1272.66	1272.64	1272.02	1272.00
010015	20			0.06	0.05				0.11	1272.47	1272.36	1272.64	1272.53	1272.00	1271.95
010020	75			0.23			0.03		0.26	1272.36	1272.10	1272.53	1272.27	1271.95	1271.76
010025	100			0.30			0.11		0.41	1272.10	1271.69	1272.27	1271.74	1271.76	1271.51
010030	200			0.14				0.03	0.17	1271.69	1271.52	1271.74	1272.61	1271.55	1271.05
									0.00						
010035	**300**			**1.50**		**0.05**			**1.55**	**1273.07**	**1271.52**	**1274.09**	**1272.61**	**1272.72**	**1271.22**
010040	**80**			**0.42**	**0.11**				**0.53**	**1271.52**	**1270.99**	**1272.61**	**1272.08**	**1271.22**	**1269.92**
OUTLET			1.09						1.09	1270.99	1269.90	1272.08	1269.90		

TABLE E.5
CONNECTOR PIPE SUMMARY

Connector Pipe											
Connector ID	Length Of Lateral	Lateral Size	Catch Basin Depth	Inlet ID	Entrance Loss, Ke	h_i	h_f	h_{total}	HGL at Inlet	Catch Basin Inlet Elevation	Head Between 1' Below Lip of Gutter to Inlet H.G.L[1]
	(ft)	(ft)	(ft)			(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
1	2	3	4	5	6	7	8	9	10	11	12
P0105	36	15	3.50	0105	0.50	0.01	0.01	0.02	1272.76	1274.23	0.47
P0106	**38**	**15**	**3.50**	**0106**	**0.50**	**0.07**	**0.05**	**0.12**	**1272.86**	**1274.24**	**0.38**
P0108	35	15	3.60	0108	0.50	0.04	0.02	0.06	1272.61	1273.73	0.12
P0107	36	15	3.58	0107	0.50	0.04	0.02	0.06	1272.53	1273.73	0.20

Note: [1] (Column 11)-(Column 10) – 1 ft.

4.7 REFERENCES

American Iron and Steel Institute, 1990, *Modern Sewer Design*.

American Society of Civil Engineers and the Water Environment Federation, 1992, *Design and Construction of Urban Stormwater Management Systems*.

California Department of Transportation, 1985, *Hydraulic Design and Procedures Manual*.

City of Phoenix, 2005, *City of Phoenix Standard Drawings,* http://phoenix.gov/streets/mag-suppl.html

Clark County Regional Flood Control District, 1990, *Hydrologic Criteria and Drainage Design Manual*.

Los Angeles County Flood Control District, 1982, *Hydraulic Design Manual*.

Maricopa Association of Governments, 1998, *Uniform Standard Details for Public Works Construction*.

Orange County Flood Control District, 1972, *Design Manual, Channel Hydraulics and Structures*.

United States Department of Transportation (USDOT), 2001, *Urban Drainage Design Manual, Hydraulic Engineering Circular No. 22*, FHWA-NHI-01-021. [USDOT Hydraulics WEB Site]

University of Missouri, 1958, *Pressure Changes at Storm Drain Junctions, Engineering Series Bulletin No. 41*, Engineering Experiment Station.

4.8 APPENDIX 4-A PRESSURE MOMENTUM ANALYSIS

The following pressure plus momentum method for evaluating junctions are offered to aid the designer in situations in which crowns of pipes at a storm drain junction are not matching. The method is taken from Orange County Flood Control District Design Manual, *July 1972*.

Junctions should be analyzed by the specific force (pressure plus momentum, P+M) method if the incremental increase in flow is more than 10 percent of the flow in the main channel or if the incremental increase, regardless of magnitude, could adversely affect the system. Structures flowing at slightly supercritical velocities are especially susceptible to adverse affects from side inflows.

The P+M method (based on Newton's second law of motion) has been expanded from the Corps of Engineers open channel analysis to include all junctions.

The general equilibrium equation is:

$$P_2 + M_2 = P_1 + M_1 + M_3 \cos\theta + P_i + P_w - P_f$$

where:

P_1 = hydrostatic pressure on section 1,

P_2 = hydrostatic pressure on section 2,

P_i = horizontal component of hydrostatic pressure on invert,

P_s = horizontal component of hydrostatic pressure on soffit,

P_w = axial component of hydrostatic pressure on walls,

P_f = retardation force of friction,

M_1 = momentum of moving mass of water entering junction at section 1,

M_2 = momentum of moving mass of water leaving junction at section 2,

$M_3 \cos\theta$ = axial component of momentum of the moving mass of water entering the junction at section 3.

The expression for pressure acting on an area is:

$$P = wAy \text{ (lbs)}$$

where:

w = unit weight of water, lbs/ft^3,

A = cross sectional area of flow, sq ft,

y = average depth, ft.

and for momentum per unit time is:

$$M = wQV/g \text{ (lbs/s)}$$

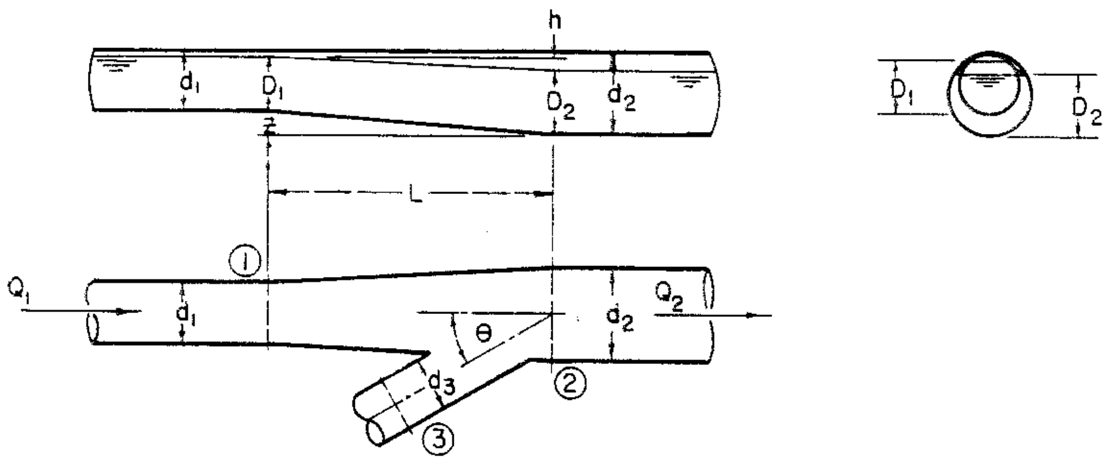
However, since the unit weight of water (w) appears in all terms of the general equilibrium equation it may be omitted and the dimension for P+M becomes feet to the third power.

Since most applications of junction analysis involve relatively small elevation changes simplifying assumption have been made that cosines of the invert slope equal unity and tangents and sines of the friction slope are equal.

The designer should recognize that components of wall and invert pressures may be either positive or negative and should be used accordingly.

Often when a confluence is within a transition from trapezoidal to rectangular shape (or reverse), a portion of the invert and wall pressures are of negative sign. These can be measured by superimposing the end areas of the sections over each other and developing a graphical representation of the negative areas. By adding algebraically the component Ay's, a reasonable approximation of the wall and invert pressures is obtained.

FIGURE 4.A1
CIRCULAR CONDUIT FLOWING PARTIALLY FULL, PIPE INLET
(Orange County Flood Control District, 1972)



$$P_1 = C_1 d_1^3 \qquad P_2 = C_2 d_2^3$$

$$M_1 = K_1 \left(\frac{Q_1}{d_1}\right)^2 \qquad M_2 = K_2 \left(\frac{Q_2}{d_2}\right)^2$$

$$M_3 \cos\Theta = \frac{(Q_2 - Q_1)^2}{25.2 d_3^2} (\cos\Theta)$$

$$P_i = 0$$

$${}^{*}P_w = A_2\bar{y}_2 - A_1\bar{y}_1 + \frac{h}{2}(A_2 + A_1) + \frac{(h)^2}{12}(T_2 - T_1)$$

$$P_f = \frac{L(s_1 + s_2)}{4}(A_1 + A_2)$$

For tabulated values of C and K, see Page 13.

See King "Handbook of Hydraulics", for $A_1, \bar{y}$ and T.

* Where $h = z + D_1 - D_2$, The term $\frac{(h)^2}{12}(T_2 - T_1)$ is usually negligible.

TABLE 4.A1
PRESSURE PLUS MOMENTUM FACTORS FOR PARTIALLY FULL CIRCULAR CONDUITS

(Orange County Flood Control District, 1972)

$\frac{D}{d}$	K	C	F	$\frac{D}{d}$	K	C	F
.01	23.919	.0000	9188.	.51	.0773	.0873	.0958
.02	8.403	.0000	1134.	.52	.0753	.0914	.0912
.03	4.507	.0001	326.	.53	.0736	.0956	.0869
.04	2.961	.0002	140.9	.54	.0719	.0998	.0829
.05	2.115	.0003	71.9	.55	.0703	.1042	.0793
.06	1.620	.0005	42.1	.56	.0687	.1087	.0758
.07	1.285	.0007	26.5	.57	.0672	.1133	.0726
.08	1.058	.0010	17.97	.58	.0658	.1179	.0696
.09	0.888	.0013	12.68	.59	.0645	.1227	.0668
.10	0.760	.0017	9.28	.60	.0632	.1276	.0641
.11	0.662	.0021	7.03	.61	.0620	.1326	.0617
.12	0.582	.0026	5.45	.62	.0608	.1376	.0594
.13	0.518	.0032	4.31	.63	.0597	.1428	.0572
.14	0.466	.0038	3.48	.64	.0586	.1428	.0551
.15	0.421	.0045	2.84	.65	.0575	.1534	.0532
.16	0.383	.0053	2.36	.66	.0565	.1589	.0514
.17	0.351	.0061	1.982	.67	.0559	.1644	.0496
.18	0.324	.0070	1.681	.68	.0547	.1700	.0480
.19	0.299	.0080	1.438	.69	.0538	.1758	.0465
.20	0.278	.0091	1.242	.70	.0530	.1816	.0450
.21	0.259	.0103	1.080	.71	.0521	.1875	.0437
.22	0.243	.0115	0.946	.72	.0514	.1935	.0424
.23	0.228	.0128	0.833	.73	.0506	.1996	.0411
.24	0.215	.0143	0.740	.74	.0499	.2058	.0400
.25	.2026	.0157	0.659	.75	.0492	.2121	.0389
.26	.1916	.0173	0.589	.76	.0485	.2185	.0379
.27	.1817	.0190	0.530	.77	.0479	.2249	.0369
.28	.1727	.0207	0.479	.78	.0473	.2314	.0359
.29	.1645	.0226	0.435	.79	.0467	.2380	.0351
.30	.1569	.0255	0.395	.80	.0462	.2447	.0342
.31	.1493	.0266	0.361	.81	.0456	.2515	.0334
.32	.1435	.0287	0.331	.82	.0451	.2584	.0327
.33	.1376	.0309	0.304	.83	.0446	.2653	.0320
.34	.1320	.0332	0.280	.84	.0441	.2723	.0313
.35	.1269	.0356	0.259	.85	.0437	.2794	.0307
.36	.1221	.0381	0.240	.86	.0433	.2865	.0301
.37	.1177	.0407	0.222	.87	.0429	.2938	.0295
.38	.1135	.0434	0.207	.88	.0425	.3011	.0290
.39	.1096	.0462	.1931	.89	.0421	.3084	.0285
.40	.1060	.0491	.1804	.90	.0418	.3158	.0280
.41	.1026	.0520	.1689	.91	.0414	.3233	.0276
.42	.0993	.0551	.1585	.92	.0411	.3308	.0272
.43	.0963	.0583	.1489	.93	.0408	.3384	.0266
.44	.0934	.0616	.1402	.94	.0406	.3460	.0265
.45	.0907	.0650	.1321	.95	.0403	.3537	.0261
.46	.0882	.0684	.1248	.96	.0401	.3615	.0259
.47	.0857	.0720	.1180	.97	.0399	.3692	.0256
.48	.0834	.0757	.1118	.98	.0398	.3770	.0254
.49	.0813	.0795	.1060	.99	.0397	.3848	.0253
.50	.0792	.0833	.1007	1.00	.0396	.3927	.0252

Tabulated Values

$$M = k(Q/d)^2 \qquad P = C \cdot d^3$$

$$h_v = F(Q/d^2)^2$$

Page intentionally left blank

5 CULVERTS & BRIDGES

TABLE OF CONTENTS

5.1 SYMBOLS

The following symbols will be used in equations throughout Chapter 5.

θ_j = Angle between outfall and lateral at a junction, degrees

a = Angle of approach, degrees

α_e = Coefficient

α_u = Unit coefficient constant, 180 lb/ft^2

A = Cross sectional area, sq ft

B = Width of culvert opening, ft

C_h = Drop height adjustment coefficient at culvert outlet

C_r = Road embankment overtopping discharge coefficient

C_s = Slope correction coefficient

d	=	Inlet bevel, in
ρ	=	fluid density of water, 1.94 slugs/ft^3
d_c	=	Critical depth, ft
d_s	=	Depth of scour hole, ft
d_{50}	=	Diameter of a rock particle for which 50% of the gradation is finer by weight (other percentages may also be used)
D	=	Pipe culvert diameter or box culvert depth, ft
$D.I.$	=	Discharge intensity
EL_o	=	Invert elevation at the outlet, ft
EL_{ho}	=	Outlet control headwater elevation, ft
$FALL$	=	Difference between invert elevation and original streambed elevation, ft
g	=	Acceleration due to gravity, 32.2 ft/sec^2
H	=	Sum of inlet loss, friction loss, and velocity head in a culvert, ft
H_b	=	Head loss through a bend of a culvert, ft
H_j	=	Head loss through a junction, ft
H_t	=	Head loss due to turning flow at a headwall, ft
H_v	=	Velocity head, ft
HW	=	Depth from inlet invert to upstream total energy grade line, ft
HW_r	=	Flow depth above the roadway, ft
h_o	=	Height of hydraulic grade line above outlet invert, ft
h_t	=	Height of tailwater above crown of submerged road, ft
K_b	=	Bend loss coefficient
K_e	=	Entrance loss coefficient
K_t	=	Submergence factor
L	=	Actual length of culvert, ft
L_1	=	Adjusted culvert length, ft
L_a	=	Length of apron, ft
L_r	=	Width of roadway crest over the roadway, ft
L_x	=	Length of overflow sections along embankment normal to flow, ft
n	=	Manning's n-value
n_1	=	Desired Manning's n-value
PI	=	Plasticity Index from Atterberg limits
Q	=	Rate of flow, cfs
Q_o	=	Rate of flow overtopping roadway, cfs
R_c	=	Hydraulic radius at the end of the culvert (assuming full flow)
S	=	Slope, ft/ft
S_v	=	Saturated shear strength, lb/ft^2
TW	=	Tailwater depth measured from culvert outlet invert, ft

t	=	Time, minutes
τ_c	=	critical tractive shear stress, lb/ft^2
V	=	Velocity, ft/sec
V_a	=	Approach channel velocity, ft/sec
W_a	=	Width of apron, ft
y'	=	Change in hydraulic grade line through the junction, ft
y_e	=	Equivalent depth, ft
y_s	=	Depth of scour, ft
σ	=	Material standard deviation

5.2 INTRODUCTION

Culverts and bridges are structures that convey stormwater under roads. Their purpose is to prevent water from the more frequent storm events from overtopping and crossing the road as such conditions inhibit safe passage of vehicles. The intent of this chapter is to provide guidance for the design of culverts. This includes the necessary design aids/examples and guidance for treatment of culvert inlets and outlets. Some brief guidelines are presented to follow when using inverted siphons. The design of bridges requires special training and experience in regard to hydraulic analyses, design of flow training works, and estimates of pier and abutment scour. Therefore, only an overview of the hydraulic analyses for bridge openings is presented.

5.3 CULVERTS

The charts and procedures for culvert design used in this manual are taken from the *Federal Highway Administration, Hydraulic Design Series Number 5, Hydraulic Design of Highway Culverts* (USDOT, FHWA, HDS-5, 1985). Culvert designers use this reference liberally as it is the result of years of research and experience in culvert design and at this time represents the state of the art.

5.3.1 Use of Culverts

Culverts are primarily used for conveying runoff through a roadway embankment. They are normally aligned with a watercourse or engineered drainage channel. Culverts are typically used for smaller drainageways. They may also serve as outfall structures for storm drain systems. Bridges are generally used for larger drainageways such as large washes and rivers.

5.3.2 Culvert Design Criteria

Sizing

Minimum culvert sizing shall be in accordance with the appropriate jurisdictional standards.

Minimum Velocity

Culverts should be designed to provide adequate velocity to self-clean during partial depth flow events. Debo and Reese (1995) suggest a minimum velocity of 2.5 feet per second for partial flow depths. Greater velocities are recommended for installations where sediment loads are heavy. Alternatively, a sediment trap can be utilized where culvert velocities are lower or excessive sediment deposition is expected.

Maximum Velocity

As a practical limit, outlet velocities should be kept below 15 feet per second unless special conditions exist. The maximum velocity should be consistent with channel stability requirements at the culvert outlet. As outlet velocities increase, the need for channel stabilization at the culvert outlet increases. If culvert outlet velocities exceed permissible velocities for the outlet channel lining material, suitable outlet protection must be provided. Outlet velocities may exceed permissible downstream channel velocities by up to 10 percent without providing outlet protection if the culvert tailwater depth is greater than the culvert critical depth of flow under design flow conditions. Table 6.2 and Table 6.3 outline the permissible velocities for several channel lining materials.

Materials

The selection of a culvert material may depend upon structural strength, hydraulic roughness, durability, and corrosion and abrasion resistance. The culvert materials that should be considered are concrete (reinforced and non-reinforced), corrugated aluminum, corrugated steel, and PVC. Culverts may also be lined with other materials to inhibit corrosion and abrasion. Linings are not recommended to reduce hydraulic resistance because culvert linings have a short life span and are seldom reapplied as part of normal culvert maintenance. When linings are applied, the culvert sizing should neglect the reduced roughness from the lining material.

Minimum Cover

Minimum cover of fill over culverts must be provided to maintain the structural integrity of the structure under anticipated loading conditions. Culvert manufacturers provide minimum cover requirements for prefabricated pipe. A general rule of thumb for estimating minimum cover requirements is to provide one-eighth of the barrel diameter or span, with a minimum of 1 foot. The top of culverts should not extend into the roadway subgrade. Minimum cover should be measured from the top of subgrade, which is the bottom of the pavement structural section.

Depth for Road Crossing

The allowable headwater depth, allowable flow across the street, and design storm frequency requirements should be verified with each jurisdictional entity's policies and standards.

Regardless of the size of the culvert, street crossings shall be designed to convey the 100-year storm runoff under and/or over the road to an area downstream of the crossing to which the flow would have gone in the absence of the street crossing. Flows up to and including the 100-year frequency event should not cause increased flooding to adjacent property or buildings, unless a drainage easement is acquired for those areas. The ponded headwater elevation should be delineated on a contour map, or other surveying methods used to identify the area inundated by the ponded water.

In general, dip sections are not recommended, however, for flows crossing broad shallow washes where the construction of a culvert is not practical, the road may be dipped to allow the entire flow to cross the road. Use of dip sections for specific, individual cases must be approved by the governing agency. The pavement through the dip section should be concrete and should have a one way slope in the direction of flow with curbs and medians flush with the pavement. Upstream and downstream cutoff walls and aprons should be provided to minimize the effects of headcutting and erosion.

Scour and Sedimentation

Possible aggradation or degradation at culvert crossings must be examined in the design of culverts.

An ideal culvert design should pass drainage water through it without upsetting the delicate balance between hydraulics and sediment transport.

An effective culvert design should minimize scour and deposition. For example, suitable outlet protection should be provided to minimize scour. To minimize sedimentation problems, inlets should not be depressed below the natural channel flowline. In addition, multi-barrel installations tend to reduce the channel velocity, particularly in low flow situations. Where multi-barrel installations are necessary, provisions should be made to handle sedimentation with minimal maintenance.

Skewed Channels

A good culvert design is one that limits the hydraulic and environmental stress placed on an existing natural watercourse. This stress can be minimized by designing a culvert that closely conforms to the natural stream in alignment and grade. Often the culvert barrel must be skewed with respect to the roadway centerline to accomplish this goal. Alterations to the normal inlet alignment are often necessary as well.

The alignment of a culvert barrel with respect to a line perpendicular to the roadway centerline at the point of crossing is referred to as the barrel skew angle. A culvert aligned normal to the roadway centerline has a zero skew angle. Directions (right or left) must accompany the barrel skew angle (Figure 5.1). Some advantages of following a natural stream alignment include: reduction of entrance losses, equal depths of scour at the footings, less sedimentation, and less excavation for installation.

The angle from the culvert face to a line normal to the culvert barrel is referred to as the inlet skew angle (Figure 5.2). The structural integrity of circular sections is compromised when the inlet is skewed due to the loss of a portion of the full circular section where the culvert barrel extends beyond the full section. Although concrete headwalls help stabilize the pipe section, structural considerations should not be overlooked in the design of skewed inlets. Culverts which have a barrel skew angle often have an inlet skew angle as well. This is because headwalls are generally constructed parallel to a roadway centerline to avoid warping of the embankment fill.

In cases where the culvert barrel cannot be aligned with the channel flowline, such as when runoff is directed along a roadway embankment to a suitable crossing location, the flow enters the culvert barrel at an angle. The approach angle should be limited to a maximum of 90 degrees. When high velocities exist, inlet losses resulting from turning the flow into the culvert should be considered. If backwater computations are not employed and the approach channel velocity is 6 feet per second or greater, the following equation should be used to estimate the loss. The loss should be added to the other inlet losses in the culvert design computation, if they aren't included in the appropriate nomographs.

$$H_t = \left(\frac{V_a^2}{2g}\right) \sin a \qquad (5.1)$$

FIGURE 5.1
BARREL SKEW ANGLE
(USDOT, FHWA, HDS-5, 1985)

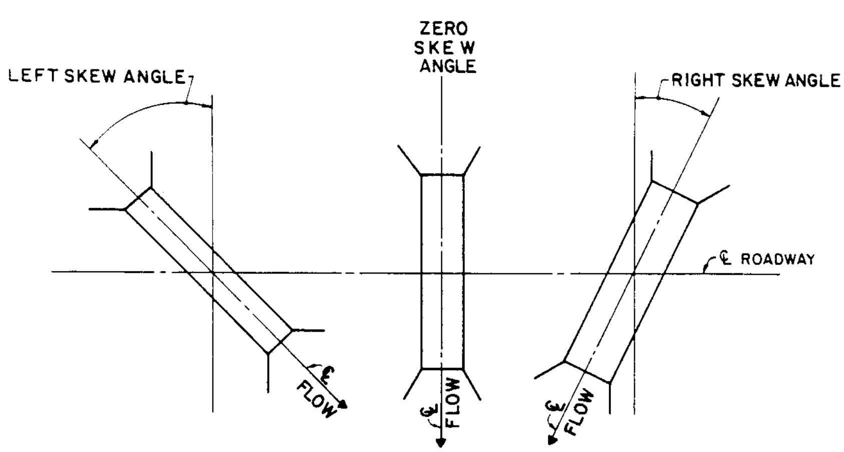


FIGURE 5.2
INLET SKEW ANGLE
(USDOT, FHWA, HDS-5, 1985)

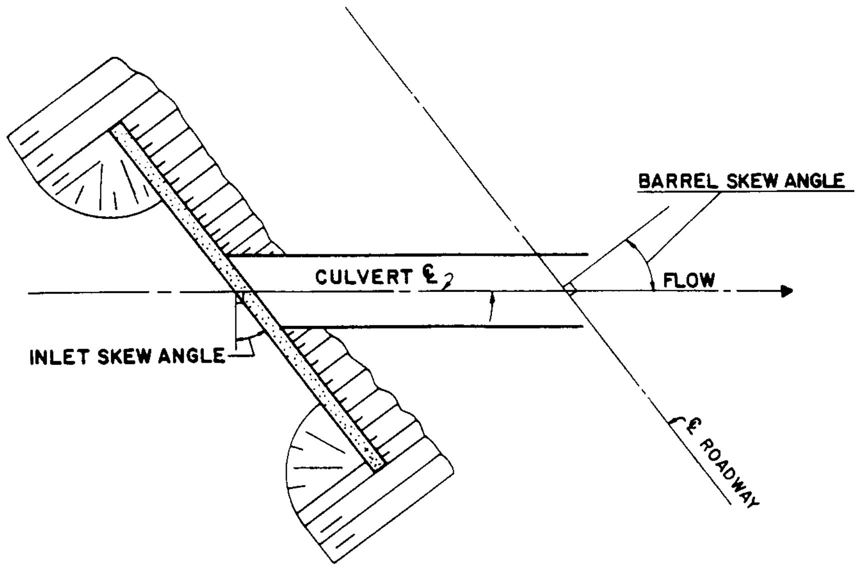
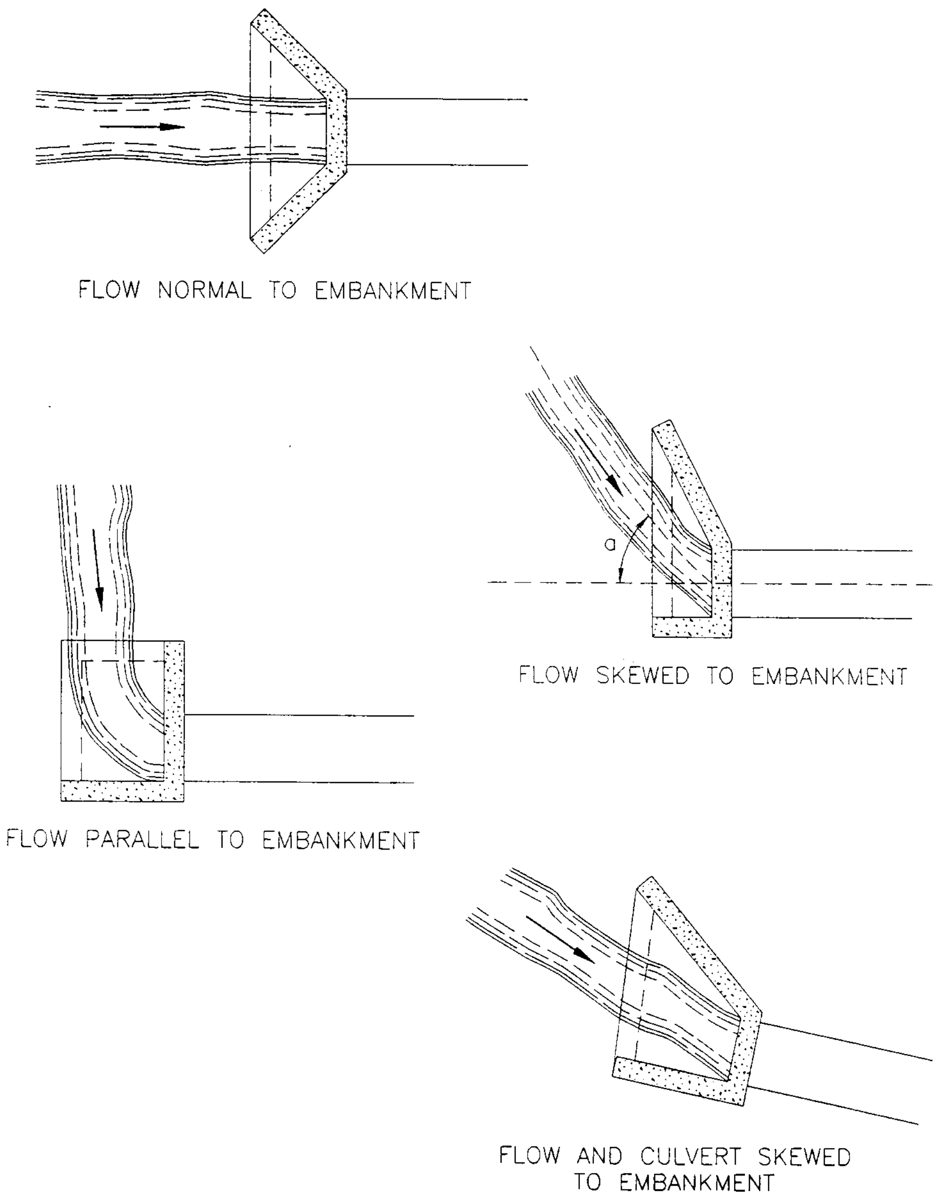


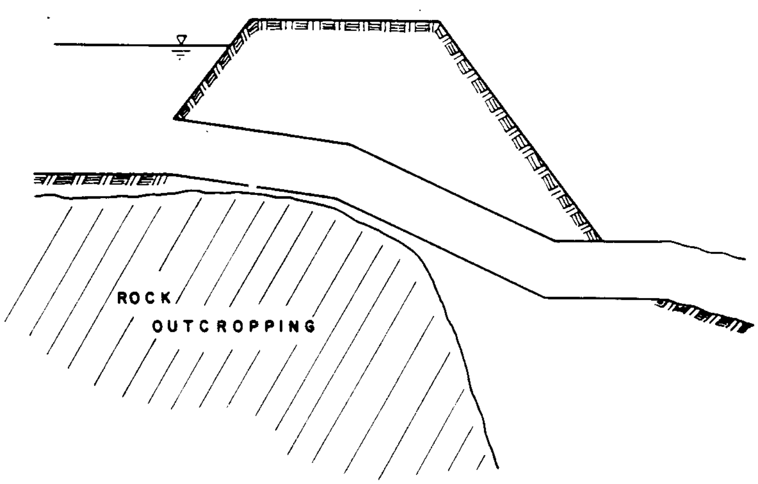
FIGURE 5.3
TYPICAL HEADWALL/WINGWALL CONFIGURATIONS FOR SKEWED CHANNELS
(USDOT, FHWA, HDS-5, 1985)



Bends

A straight culvert alignment is desirable to avoid clogging, increased construction costs, and reduced hydraulic efficiency. However, site conditions may require a change of alignment, either horizontally or vertically. When considering a nonlinear culvert alignment, particular attention should be given to erosion, sedimentation, and debris control. Vertical bends are permitted when they transition from a flatter to a steeper slope, but should not transition from steeper to flatter slopes because of the potential for sediment deposition in the flatter reach.

FIGURE 5.4
"BROKEN BACK" CULVERT
(USDOT, FHWA, HDS-5, 1985)



In designing a nonlinear culvert, the energy losses due to the bends must be considered. If the culvert operates in inlet control, no increase in headwater occurs unless the bend losses cause the culvert to flow under outlet control. If the culvert operates in outlet control, an increase in energy losses and headwater will result due to the bend losses. To minimize these losses, the culvert should be curved or have bends not exceeding 15 degrees at intervals of not less than 50 feet. Under these conditions, bend losses can be ignored.

If these conditions cannot be met, analysis of bend losses is required. Bend losses are a function of the velocity head in the culvert barrel. To calculate bend losses, use the following equation:

$$H_b = K_b \frac{V^2}{2g} \tag{5.2}$$

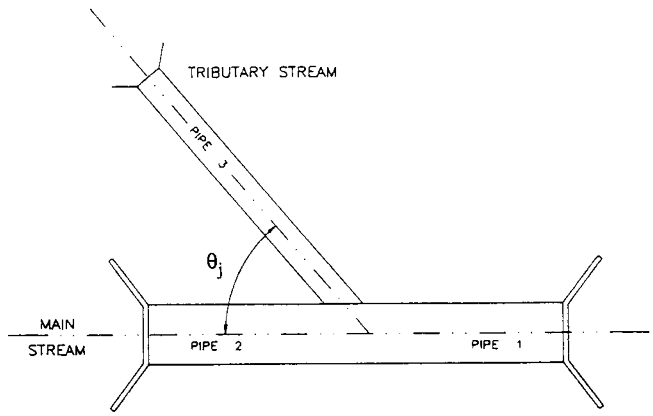
H_b is added to the other outlet losses. See Chapter 4, Storm Drains, to determine loss coefficients (K_b) for bend losses in conduits flowing full.

The broken back culvert, shown in Figure 5.4, has four possible control sections: the inlet, the outlet, and the two bends. The upstream bend may act as a control section, with the flow passing through critical depth just upstream of the bend. In this case, the upstream section of the culvert operates in outlet control and the downstream section operates in inlet control. Outlet control calculation procedures can be applied to the upstream barrel, assuming critical depth at the bend, to obtain a headwater elevation. This elevation is then compared with the inlet and outlet control headwater elevations for the overall culvert. The controlling flow condition produces the highest headwater elevation. Control at the lower bend is very unlikely. That possible control section can be ignored except for the bend losses in outlet control.

Junctions

Flow from two or more separate culverts or storm drains may be combined at a junction into a single culvert barrel. For example, a tributary and a main stream intersecting at a roadway crossing can be accommodated by a culvert junction (Figure 5.5).

FIGURE 5.5
CULVERT JUNCTION
(USDOT, FHWA, HDS-5, 1985)



Loss of head may be important in the hydraulic design of a culvert containing a junction. Attention should be given to streamlining the junction to minimize turbulence and head loss. Also, timing of peak flows from the two branches should be considered in analyzing flow conditions and control. When possible, the tributary flow should be released downstream of the culvert barrel. When this is not practical, the following procedure should be used to estimate the losses.

For a culvert barrel operating in outlet control and flowing full, the junction loss is calculated using the equations given below. The loss is then added to the other outlet control losses.

$$H_j = y' + H_{V1} - H_{V2} \tag{5.3}$$

The equation for y' is based on momentum considerations and is as follows:

$$y' = \frac{Q_2V_2 - Q_1V_1 - Q_3V_3\cos\theta_j}{0.5(A_1 + A_2)g} \tag{5.4}$$

The subscripts 1, 2, and 3 refer to the outlet pipe, the upstream pipe, and the lateral pipe respectively.

Trashracks and Access Barriers

For trashracks with approach velocities less than 3 feet per second, it is not necessary to include a head loss for the trashrack; however, for velocities greater than 3 feet per second, such computations are required. See Hydraulic Structures, Chapter 8, Section 8.6.4.

Flotation and Anchorage

Flotation is the term used to describe the failure of a culvert due to the uplift forces caused by buoyancy. The buoyant force is produced from a combination of high head on the outside of the inlet and the large region of low pressure on the inside of the inlet caused by flow separation. As a result, a large bending moment is exerted on the end of the culvert. This problem has been noted in the case of culverts under high head, with shallow cover, on steep slopes, and with projecting inlets. The phenomenon can also be caused by debris blocking the culvert end or by damage to the inlet. The resulting uplift may cause the inlet ends of the barrel to rise and bend. Occasionally, the uplift force is great enough to dislodge the embankment. Generally, flexible barrel materials are more vulnerable to failure of this type because of their light weight and lack of resistance to longitudinal bending. Large, projecting, or mitered corrugated metal culverts are the most susceptible.

A number of precautions can be taken by the designer to guard against flotation. Steep slopes (1 to 1 or steeper) of adequate height, which are protected against erosion by slope paving or headwalls, help inlet and outlet stability. When embankment fill heights are less than 1.5 times the pipe diameter or fill slopes are flatter than 1 to 1, the designer may consider other applications such as concrete encasement, concrete headwalls, and tie bars to guard against failures caused by flotation. Limiting headwater buildup also helps prevent flotation. It is desirable to limit design headwater depths to 1.5 times the culvert height.

Safety

Culverts shall be designed to conform to the safety protocols identified in the introduction to this manual.

Inlets

Culvert inlets are used to transition the flow from a headwater condition upstream of the culvert into the culvert barrel. Losses caused by the inlets have been studied extensively for several types of inlets. The inlet control nomographs in Section 5.3.4 give the required headwater depth to pass the design discharge through several types of culvert entrances. The hydraulic capacity of a culvert may be improved by appropriate inlet selection. Since the channel is usually wider than the culvert barrel, the culvert inlet edge represents a flow contraction and may be the primary flow control. The provision of a more gradual flow transition will lessen the energy loss and thus create a more hydraulically efficient inlet condition. Design charts for improved inlets are contained in *Hydraulic Design of Highway Culverts* (USDOT, FHWA, HDS No. 5, September 1985). It should be noted that improving culvert inlets will cause the greatest increase in culvert capacity when the culvert is operating in inlet control.

The hydraulic performance of culverts operating in inlet control can be improved by changing the inlet geometry of the headwall. Improvements include bevel-edged, side-tapered, and slope-tapered inlets. The advantage of these improvements is to convert an inlet control culvert closer to outlet control by using more of the barrel capacity.

A beveled-edge provides a decrease in flow contraction losses at the inlet and the entrance loss coefficient, K_e is normally reduced to 0.2, which can increase the culvert capacity by as much as 20 percent. Bevels are required on all culverts with headwalls and should be constructed as shown in Figure 5.6.

Side-tapered inlets have an enlarged face area accomplished by tapering sidewalls as shown in Figure 5.7. It provides an increase in flow capacity of 25 to 40 percent over square-edged inlets. There are two types of control sections for side-tapered inlets; face and throat control. The advantages of side-tapered inlets under throat control are; reduced flow contraction at the throat and increased head at the throat control section.

Slope-tapered inlets provide additional head at the throat section as shown in Figure 5.8. This type of inlet can have over 100 percent greater capacity than a conventional culvert with square edges. The degree of increased capacity depends upon the drop between the face and the throat section. Both the face and the throat are possible control sections. The inlet face should be designed with a greater capacity than the throat to promote flow control at the throat and therefore greater potential capacity of the culvert. This type of inlet may not be appropriate for flows containing high sediment loads; caution should be excised for this design condition.

Prefabricated steel inlet end sections (Figure 5.9) are available for corrugated steel pipe that perform about as well as a square-edged headwall inlet with an entrance loss coefficient of 0.5.

When there is a potential for inlet uplift failure or inlet damage from other sources, concrete headwalls are recommended. In some cases, such as when concrete encasement of the pipe is utilized, metal end sections such as the one shown in Figure 5.9 may be acceptable.

FIGURE 5.6
INLET BEVEL DETAIL
([USDOT](), FHWA, HDS-5, 1985)

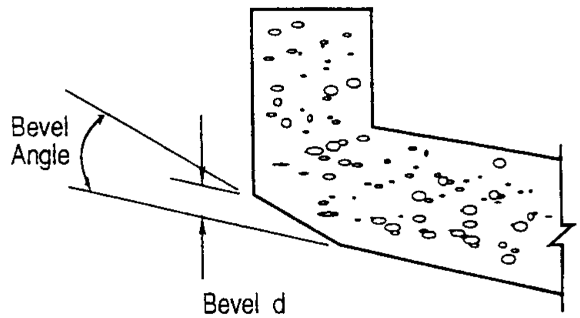


FIGURE 5.7
SIDE-TAPERED INLET
([USDOT](), FHWA, HDS-5, 1985)

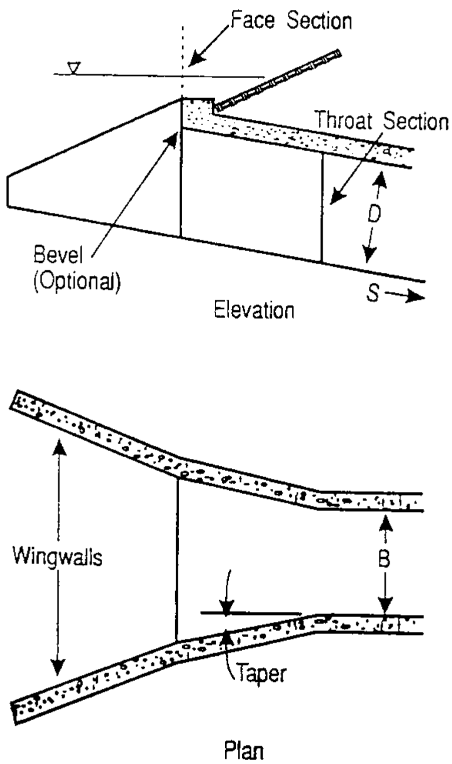


FIGURE 5.8
SLOPE-TAPERED INLET
(USDOT, FHWA, HDS-5, 1985)

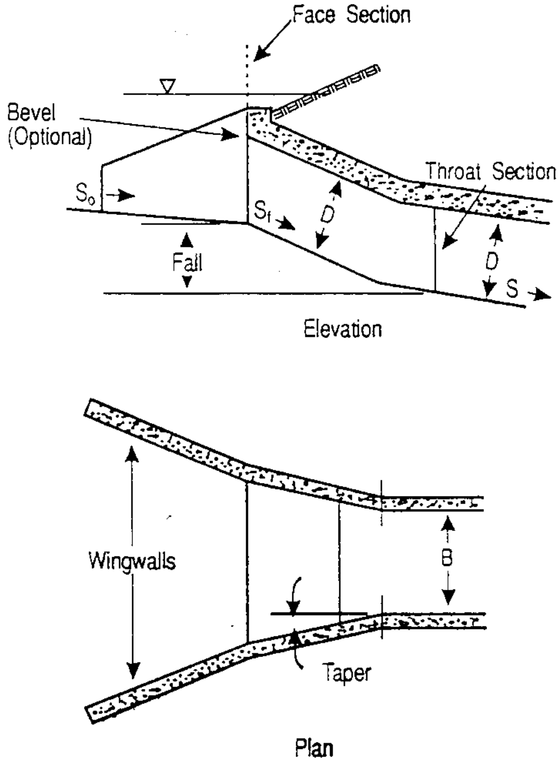
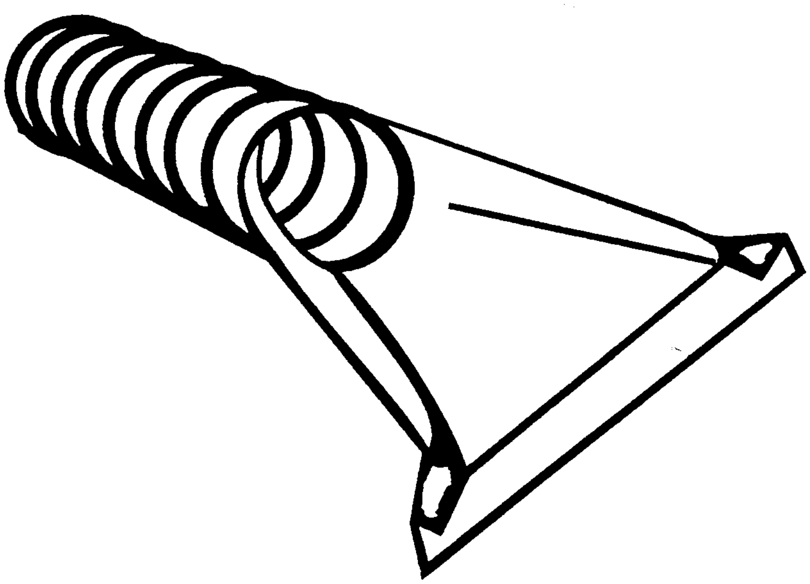


FIGURE 5.9
PREFABRICATED CULVERT END SECTION



Outlets

The receiving channel at culvert outlets must be protected from high culvert outlet velocities caused by the flow constriction that is inherent in culvert operation. If the culvert outlet velocity is greater than the allowable velocity for the receiving channel, protective measures must be provided.

Projecting culvert outlets are not permitted unless approved by the appropriate governing agency.

The minimum requirement is to provide a preformed metal or concrete end section, or a headwall (with or without a wingwall configuration) with a cutoff wall provided at the end of the apron. Culvert outlet designs are presented in Section 5.4. Energy dissipation structures, if needed are presented in Chapter 8, Hydraulic Structures, Section 8.4.

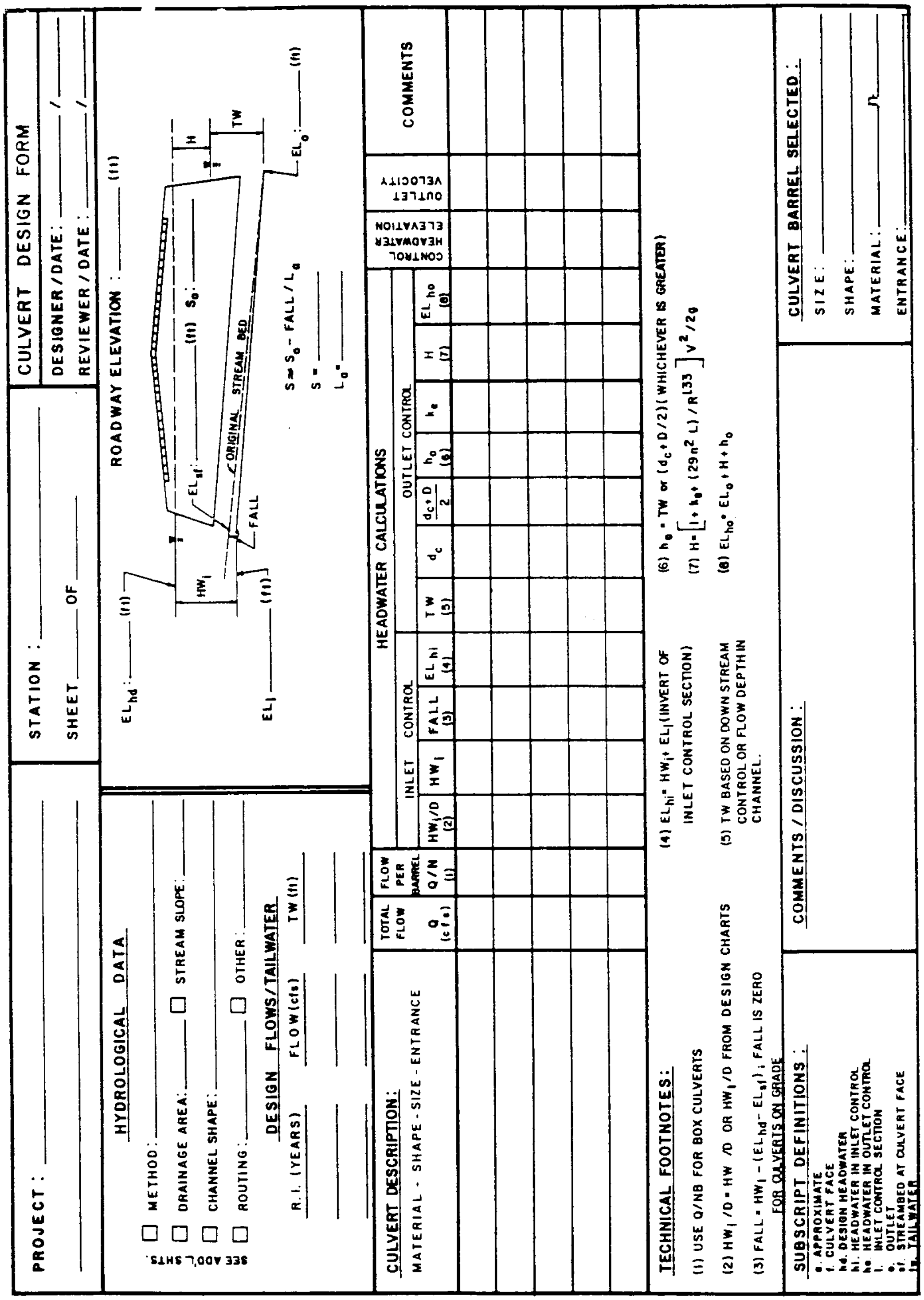
5.3.3 Design Procedures

Culvert Design Method

This design method provides a convenient and organized procedure for designing culverts, considering inlet and outlet control.; however, it is recommended that this procedure only be applied by individuals possessing a solid understanding of culvert hydraulics.

The first step in the design process is to summarize all known data for the culvert at the top of the Culvert Design Form (Figure 5.10). This includes establishing a maximum design headwater elevation, considering roadway overflow, roadway subgrade elevation, the finished floor elevation of any upstream structures, right-of-way or easement requirements for the backwater ponding elevation, and any potential flow diversions. This information will have been collected or calculated prior to performing the actual culvert design. The next step is to select a preliminary culvert material, shape, size and entrance type. The user then enters the design flow rate and proceeds with the inlet control calculations.

FIGURE 5.10
CULVERT DESIGN FORM
(USDOT, FHWA, HDS-5, 1985)



Inlet Control

The inlet control calculations determine the headwater elevation required to pass the design flow through the selected culvert configuration if the culvert is operating in inlet control. The inlet control nomographs in Section 5.3.4 are used in the design process. For the following discussion, refer to the schematic inlet control nomograph shown in Figure 5.11.

1. Locate the selected culvert size (point 1) and flow rate (point 2) on the appropriate scales of the inlet control nomograph. (Note that for box culverts, the flow rate per foot of barrel width is used.)

2. Using a straightedge, extend a straight line from the culvert size (point 1) through the flow rate (point 2) and mark a point on the first headwater/culvert height (HW/D) scale (point 3). The first HW/D scale is also a turning line.

3. If another HW/D scale is required, extend a horizontal line from the first HW/D scale (the turning line) to the desired scale and read the result.

4. Multiply HW/D by the culvert height, D, to obtain the required headwater (HW) from the invert of the control section to the energy grade line. HW equals the required headwater depth. If trashracks are used, add trashrack losses to HW.

5. Calculate the inlet control headwater elevation.

   $$EL_{hi} = EL_i + HW$$

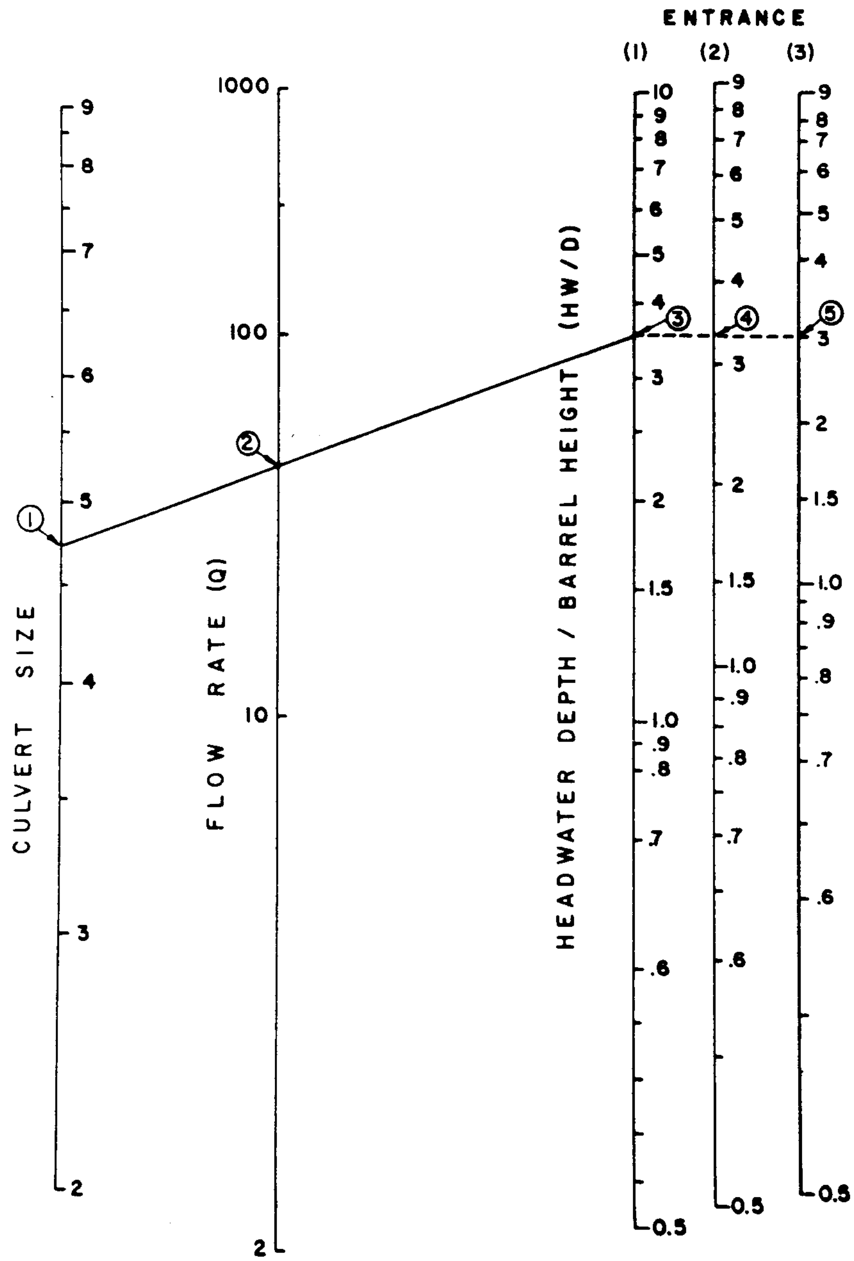
   where EL_i is the invert elevation at the inlet.

6. If the inlet control headwater elevation exceeds the design headwater elevation determined in the first step and tabulated on Figure 5.10, a new culvert configuration must be selected and the process repeated. Improvements to the inlet may suffice, or an enlarged barrel may be necessary, particularly if the outlet control headwater elevation calculated in the following section also exceeds the design headwater elevation.

Outlet Control

The outlet control calculations result in the headwater elevation required to convey the design discharge through the selected culvert if the culvert is operating in outlet control. The critical depth charts and outlet control nomographs of Section 5.3.4 are used in the design process. For illustration, refer to the schematic critical depth chart and outlet control nomograph shown in Figure 5.12 and Figure 5.13, respectively.

FIGURE 5.11
INLET CONTROL NOMOGRAPH (SCHEMATIC)
(USDOT, FHWA, HDS-5, 1985)



1. Determine the tailwater depth above the outlet invert (*TW*) at the design flow rate. This is obtained from backwater or normal depth calculations of the downstream channel, or from field observations. Field observations are important in determining tailwater depths. The area downstream of the culvert should be examined for features that may create backwater effects, i.e., channel control, another culvert, etc. If such features are found, appropriate backwater analysis techniques should be employed to determine the tailwater depth. When

culverts are in series, the headwater elevation from the downstream culvert should be checked to make sure that it doesn't back up water affecting the outlet conditions of the upstream culvert.

2. Enter the appropriate critical depth chart (Figure 5.12) with the flow rate and read the critical depth (d_c). If the computed d_c is greater than D, use D for critical depth. d_c cannot exceed the top of the culvert.

   (Note: The d_c curves are truncated for convenience when they converge. If an accurate d_c is required for d_c much greater than 0.9D, consult the *Handbook of Hydraulics* by Brater and King, 1976, or other hydraulic references.)

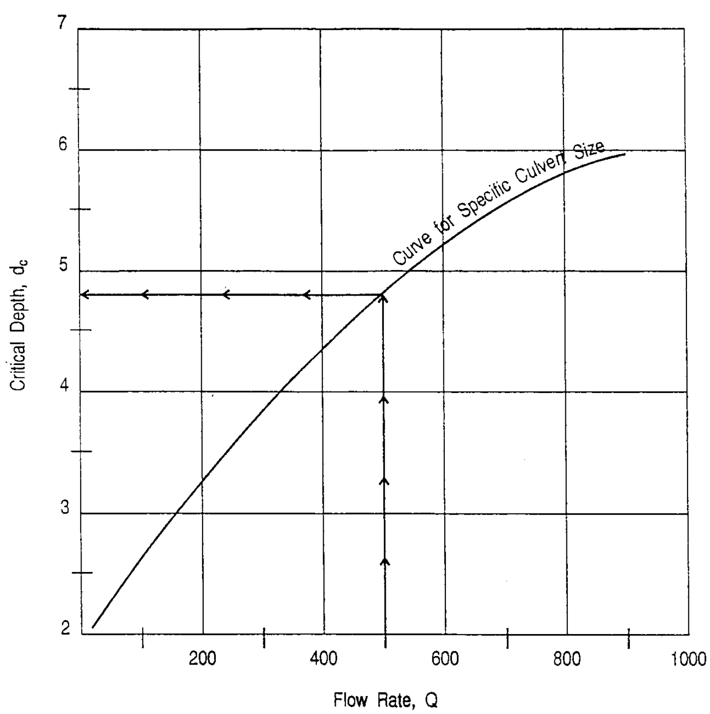
3. Calculate $(d_c + D)/2$

4. Determine the depth from the culvert outlet invert to the hydraulic grade line (h_o).

   $h_o = TW$ or $(d_c + D)/2$, whichever is larger

5. From Table 5.1 obtain the appropriate entrance loss coefficient, K_e, for the culvert inlet configuration.

FIGURE 5.12
CRITICAL DEPTH CHART (SCHEMATIC)
(USDOT, FHWA, HDS-5, 1985)



6. Determine the losses through the culvert barrel, H, using the outlet control nomograph (Figure 5.13) or appropriate equations if outside the range of the nomograph.

   a) If the Manning's n-value given in the outlet control nomograph is different than the Manning's n for the culvert, adjust the culvert length using the equation:

   $$L_1 = L\left(\frac{n_1}{n}\right)^2 \tag{5.5}$$

   Then use L_1 rather than the actual culvert length when using the outlet control nomograph.

   b) Using a straightedge, connect the culvert size (point 1) with the culvert length on the appropriate K_e scale (point 2). This defines a point on the turning line (point 3).

   c) Again using the straightedge, extend a line from the discharge (point 4) through the point on the turning line (point 3) to the Barrel Losses (H) scale. Read H, which is the energy loss through the culvert, including entrance, friction, and outlet losses.

   d) All other applicable losses should be added to H.

7. Calculate the outlet control headwater elevation.

   $$EL_{ho} = EL_o + H + h_o \tag{5.6}$$

   where EL_o is the invert elevation at the outlet.

8. If the outlet control headwater elevation exceeds the design headwater elevation determined in the first step, and tabulated on Figure 5.10, a new culvert configuration must be selected and the process repeated. Generally, an enlarged barrel will be necessary since inlet improvements are of limited benefit in outlet control.

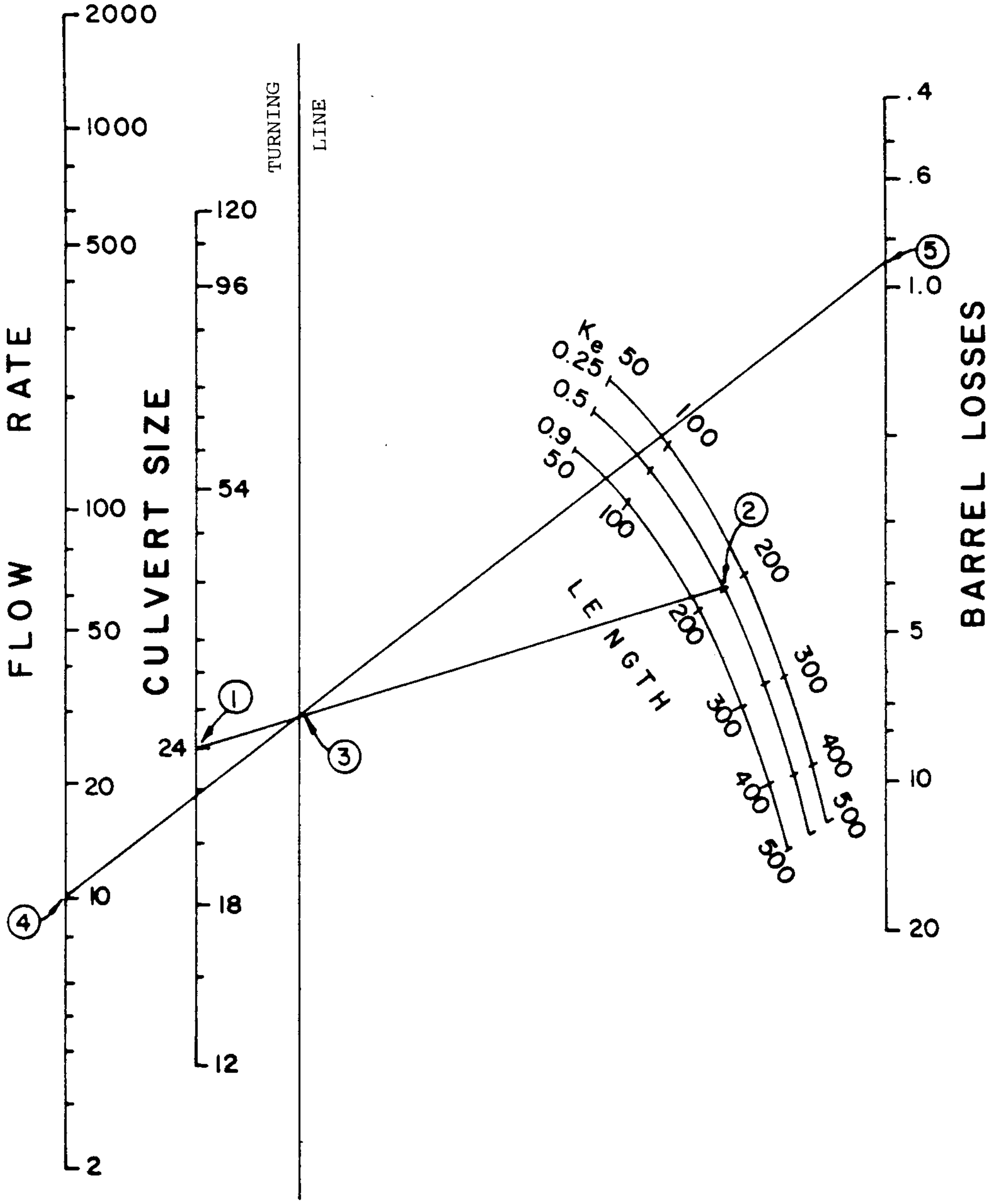
Evaluation of Results

Compare the headwater elevations calculated for inlet and outlet control. The higher of the two is designated the controlling headwater elevation. The culvert can be expected to operate with that higher headwater for at least part of the time.

The outlet velocity is calculated as follows:

1. If the controlling headwater is based on inlet control, determine the normal depth and velocity in the culvert barrel. The velocity at normal depth is assumed to be the outlet velocity (Figure 5.14). Normal depth for circular and rectangular culverts can be found using Figure 5.19.

FIGURE 5.13
OUTLET CONTROL NOMOGRAPH (SCHEMATIC)
(USDOT, FHWA, HDS-5, 1985)



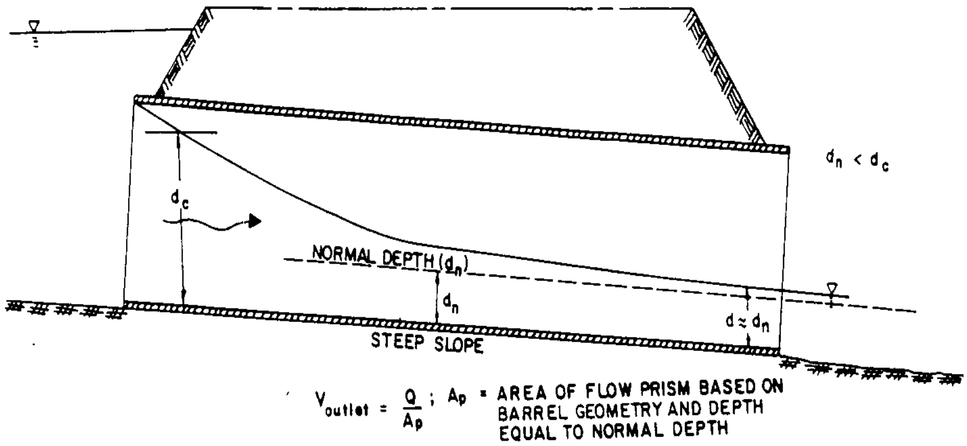
2. If the controlling headwater is in outlet control, determine the area of flow and velocity at the outlet based on the barrel geometry (see Figure 5.15) and the following:

   a) Critical depth, if the tailwater is below critical depth.

   b) The tailwater depth if the tailwater is between critical depth and the top of the barrel.

   c) The height of the barrel if the tailwater is above the top of the barrel.

FIGURE 5.14
OUTLET VELOCITY - INLET CONTROL
(USDOT, FHWA, HDS-5, 1985)



Repeat the design process until an acceptable culvert configuration is determined. Once the barrel is selected it must be fitted into the roadway cross section. The culvert barrel must have adequate cover, the length should be close to the approximate length, and the headwalls and wingwalls must be dimensioned.

If outlet control governs and the headwater depth (referenced to the inlet invert) is less than 1.2D, it is possible that the barrel flows partly full through its entire length. In this case, caution should be used in applying the approximate method of setting the downstream elevation based on the greater of tailwater or $(d_c + D)/2$. If an accurate headwater is necessary, backwater calculations should be used to check the result from the approximate method. If the headwater depth falls below 0.75D, the approximate method should not be used.

If the selected culvert will not fit the site, return to the culvert design process and select another culvert. After a selected culvert is found to meet the design conditions, document the design to this point. Culvert design documentation shall include a performance curve which displays culvert behavior over a range of discharges. Development of performance curves is presented later in this section, and Example 4 in Section 5.3.5 contains a performance curve calculation.

Additional design considerations including stage discharge ratings, roadway overtopping, and performance curves, are discussed in the following sections.

Stage Discharge Ratings

All reservoir routing procedures require three basic data inputs: 1) an inflow hydrograph; 2) a stage versus storage relationship; and 3) a stage versus discharge relationship. Stage, that is elevation above some base datum, is the parameter which relates storage to discharge providing the key to the storage routing solution.

Stage versus discharge data can be computed from culvert data and the roadway geometry as described below under Performance Curves. Discharge values for the selected culvert and over-topping flows are tabulated with reference to elevation. The combined discharge is utilized in the formulation of a performance curve.

Culverts are frequently used for detention basin outlet structures. The culvert design methods presented in this section can be used to develop the stage-discharge relationship for these structures. If the detention basin discharges into a storm drain system, procedures from Section 4.3 should be used to establish the hydraulic grade line for that stormdrain to check for outlet control.

Performance Curves

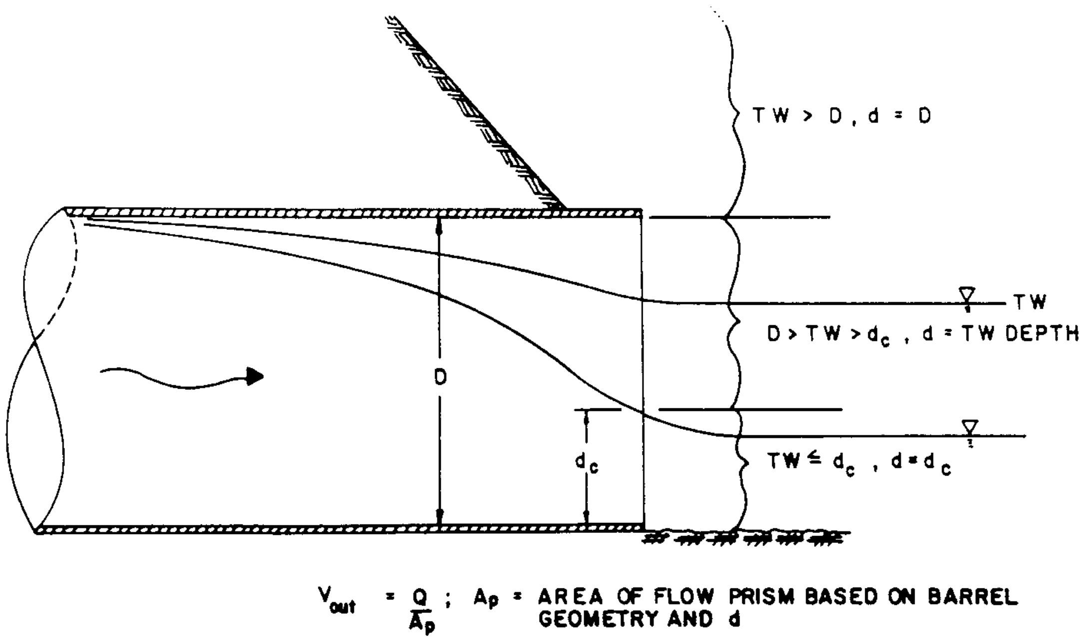
Performance curves are representations of flow rate versus headwater depth or stage for a culvert. Because a culvert has several possible control sections (inlet, outlet, throat), a given installation will have a performance curve for each control section and one for roadway overtopping. The overall culvert performance curve is made up of the controlling portions of the individual performance curves for each control section.

Inlet Control - The inlet control performance curves are developed using the inlet control nomographs of Section 5.3.4. The headwaters corresponding to the series of flow rates are determined and then plotted. The transition zone is inherent in the nomographs.

Outlet Control - The outlet control performance curves are developed using the outlet control nomographs of Section 5.3.4. Flows bracketing the design flow are selected. For these flows, the total losses through the barrel are calculated or read from the outlet control nomographs. The losses are added to the elevation of the hydraulic grade line at the culvert outlet to obtain the headwater.

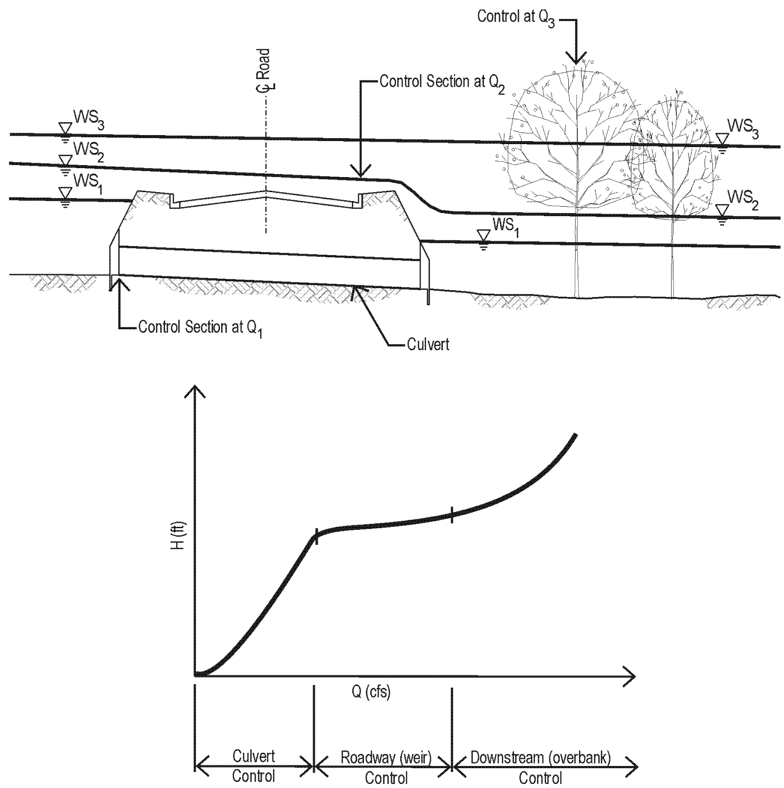
If backwater calculations are performed beginning at the downstream end of the culvert, friction losses are accounted for in the calculations. Adding the inlet loss to the energy grade line in the barrel at the inlet results in the headwater elevation for each flow rate. An example of development of a performance curve is contained in Example 4 in Section 5.3.5.

FIGURE 5.15
OUTLET VELOCITY – OUTLET CONTROL
(USDOT, FHWA, HDS-5, 1985)



Roadway Overtopping - A performance curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. Rather than using a trial and error procedure to determine the flow division between the overtopping flow and the culvert flow, an overall performance curve can be developed. The performance curve depicts the sum of the flow through the culvert and the flow across the roadway.

FIGURE 5.16
CULVERT PERFORMANCE CURVE WITH ROADWAY OVERTOPPING
(USDOT, FHWA, HDS-5, 1985)



The overall performance curve can be determined by performing the following steps:

1. Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. These flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters should be calculated. It is recommended that the 2-, 10-, 50- and 100-year flow rates be included in the range of flow rates considered.

2. Combine the inlet and outlet control performance curves to define a single performance curve for the culvert based on the controlling stage for each discharge.

3. When the culvert headwater stages exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and Equation (5.7a) or Equation to calculate flow rates across the roadway.

4. Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

Using the combined culvert performance curve, it is an easy matter to determine the headwater stage for any flow rate, or to visualize the performance of the culvert installation over a range of flow rates. When roadway overtopping begins, the rate of headwater increase will diminish. The headwater will rise very slowly from that point on. Figure 5.16 depicts an overall culvert performance curve with roadway overtopping. Example 4 in Section 5.3.5 illustrates the development of an overall culvert performance curve. The 100-year discharge should be identified on the performance curve and the corresponding depth of flow over the roadway.

The Federal Highway Administration's computer program, HY8 (USDOT, 1999), can be used in the development of performance curves. HY8 automates the design methods described in HDS-5 (USDOT, 1985), and HEC-14 (USDOT, 2006). The U.S. Army Corps of Engineers HEC-2 (USACE, 1990) and HEC-RAS computer programs (USACE, 2001a and 2001b) are also capable of analyzing culverts. The use of HY8 is preferred for design of culverts that are not subject to backwater conditions. HEC-RAS is preferred for modeling and design of culverts in river systems where backwater effects are of concern

Roadway overtopping will begin as the headwater rises to the elevation of the lowest point of the roadway. This type of flow is similar to flow over a broad crested weir. The length of the weir can be taken as the horizontal length along the roadway. The flow across the roadway is calculated from the broad crested weir equation:

$$Q_o = K_t C_r L_x (HW_r)^{1.5} \tag{5.7a}$$

The charts in provide estimates of the correction factors K_t *and* C_r.

If the elevation of the roadway crest varies, for instance where the crest is defined by a roadway sag vertical curve, the vertical curve can be approximated as a series of horizontal segments. The flow over each is calculated separately and the total flow across the roadway is the sum of the incremental flows for each segment (Figure 5.18). If the assumption of horizontal segments is invalid $(HW_{ra} > 1.5\,HW_{rb})$, the following formula may be used, assuming the value of C_r remains constant:

$$Q_o = \frac{2K_t C_r L_x (HW_{rb}^{5/2} - HW_{ra}^{5/2})}{5(HW_{rb} - HW_{ra})} \tag{5.7b}$$

where: HW_{ra} = flow depth above the roadway at the high end of the weir segment, ft.

HW_{rb} = flow depth above the roadway at the low end of the weir segment, ft.

Adapted from Hulsing (1968).

The total flow across the roadway then equals the sum of the roadway overflow plus the culvert flow. A performance curve must be plotted including both culvert flow and road overflow. The headwater depth for a specific discharge, such as the 100-year discharge can then be read from the curve. Design Example 4 in Section 5.3.5 illustrates this procedure.

FIGURE 5.17
DISCHARGE COEFFICIENT AND SUBMERGENCE FACTOR FOR ROADWAY OVERTOPPING
(USDOT, FWHA, HDS-5, 1985)

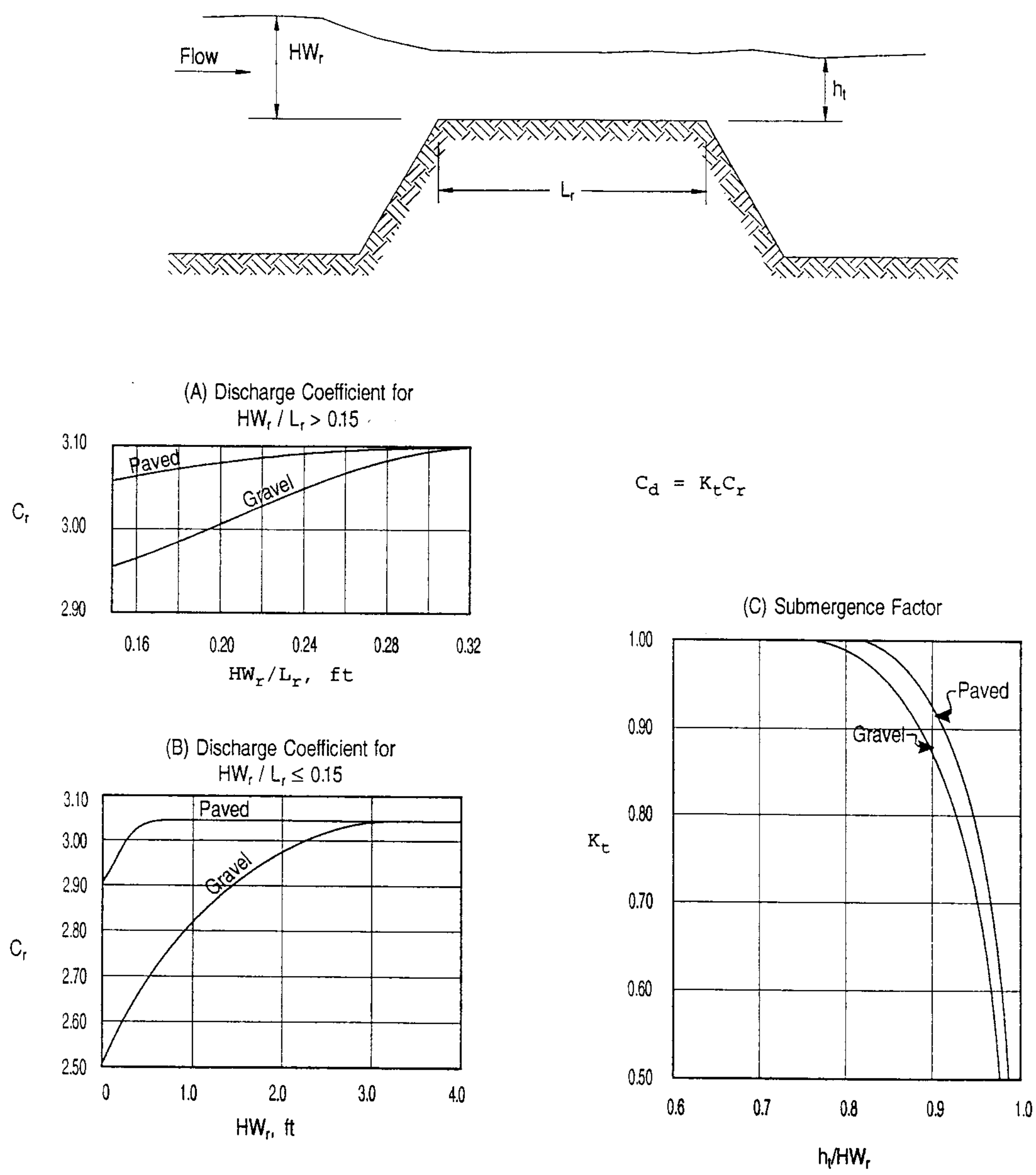
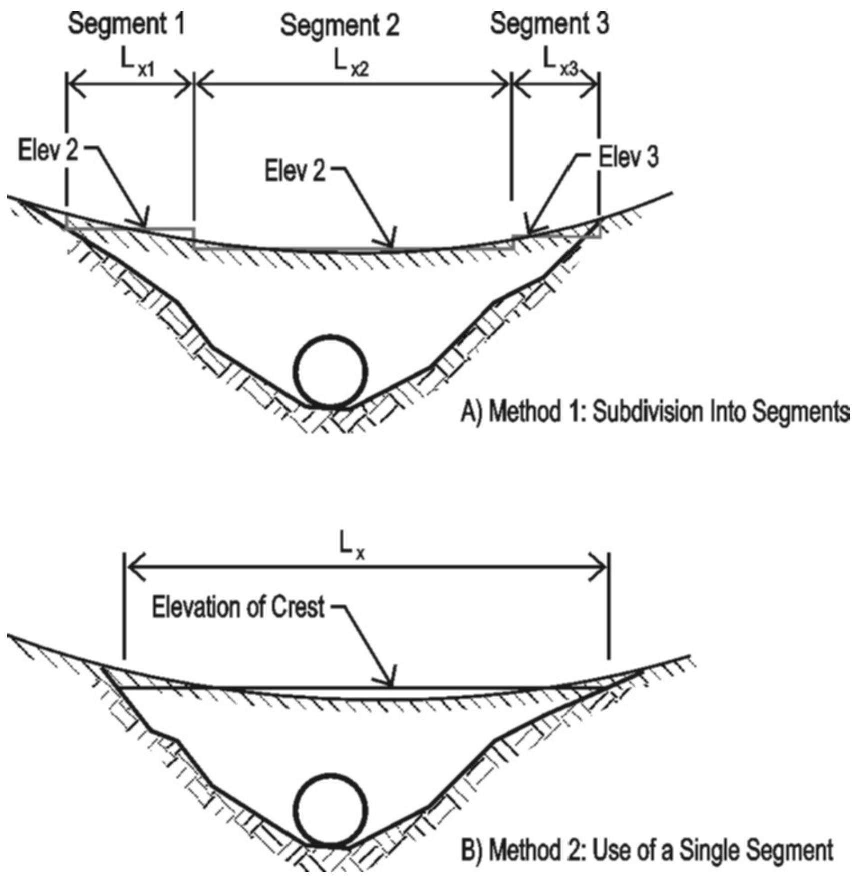


FIGURE 5.18
WEIR CREST LENGTH DETERMINATIONS FOR ROADWAY OVERTOPPING
(USDOT, FHWA, HDS-5, 1985)



5.3.4 Design Aids

Computer programs for culvert design are acceptable provided they are based on USDOT, FHWA, HDS-5, 1985.

The Culvert Design Form (Figure 5.10) has been formulated to guide the user through the design process. Summary blocks are provided at the top of the form for the project description, and the designer's identification. Summaries of hydrologic data are also included. At the top right is a small sketch of a culvert with blanks for inserting important dimensions and elevations.

The central portion of the design form contains lines for inserting the trial culvert description and calculating the inlet control and outlet control headwater elevations. Space is provided at the lower center for comments and at the lower right for a description of the culvert barrel selected. The design chart should be completely filled out, including consideration of inlet and outlet control. Table 5.1 and Figure 5.19 through Figure 5.38 should facilitate completion of the Culvert Design Form.

TABLE 5.1
ENTRANCE LOSS COEFFICIENTS
OUTLET CONTROL, FULL OR PARTLY FULL ENTRANCE HEAD LOSS
(USDOT, FHWA, HDS-5, 1985)

Type of Structure and Design of Entrance	Coefficient, K_e
Pipe, Concrete	
Projecting from fill, socket end (grove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (grove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12 D)	0.2
Mitered to conform to fill slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7^{o} or 45^{o} bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7^{o} or 45^{o} bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled on sides	0.2
Wingwalls at 30^{o} to 75^{o} to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwalls at 10^{o} to 25^{o} to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

FIGURE 5.19
CURVES FOR DETERMINING THE NORMAL DEPTH
(Chow, 1959)

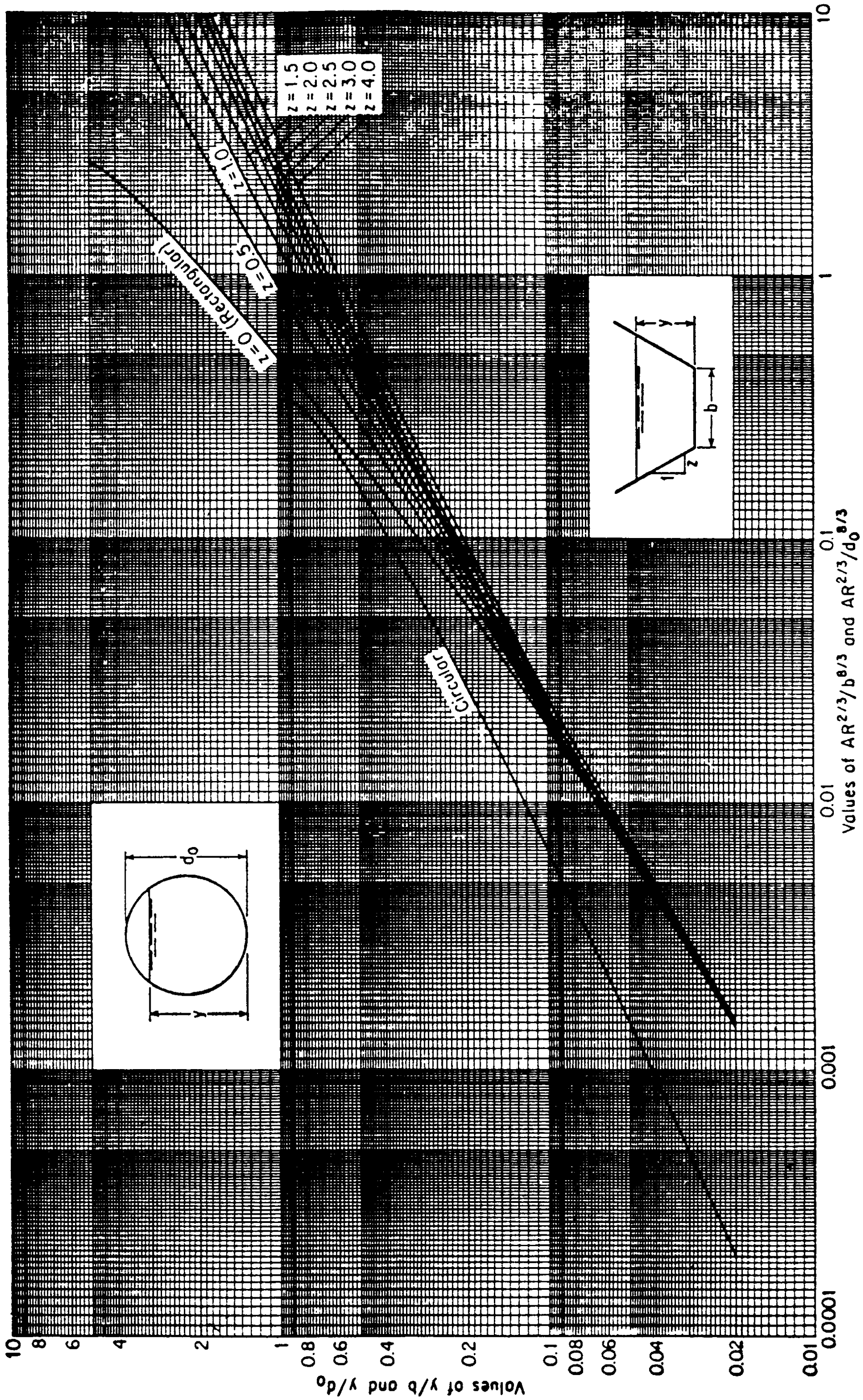


FIGURE 5.20
INLET CONTROL HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS
(USDOT, FHWA, HDS-5, 1985)

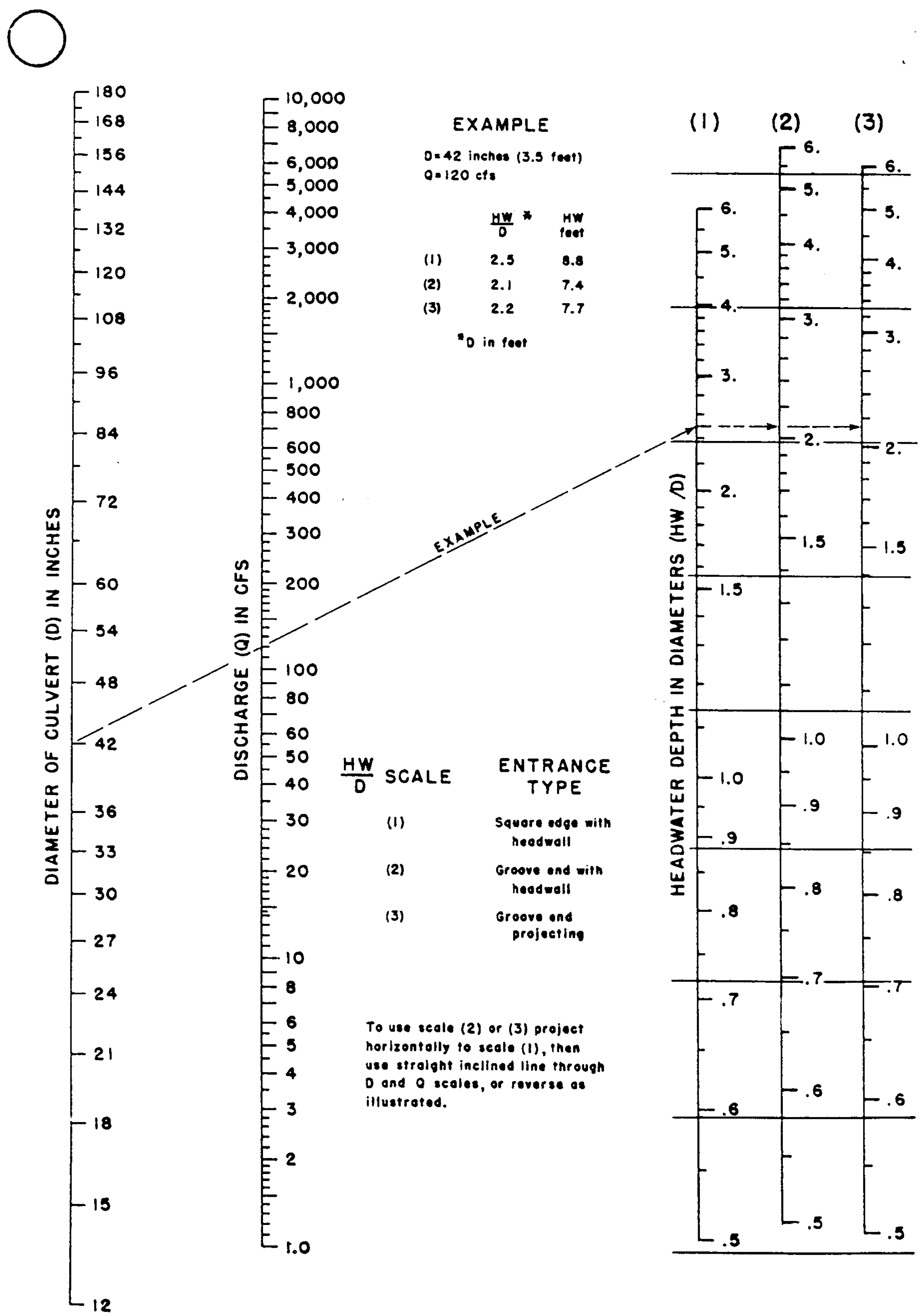


FIGURE 5.21
INLET CONTROL HEADWATER DEPTH FOR C.M. PIPE
(USDOT, FHWA, HDS-5, 1985)

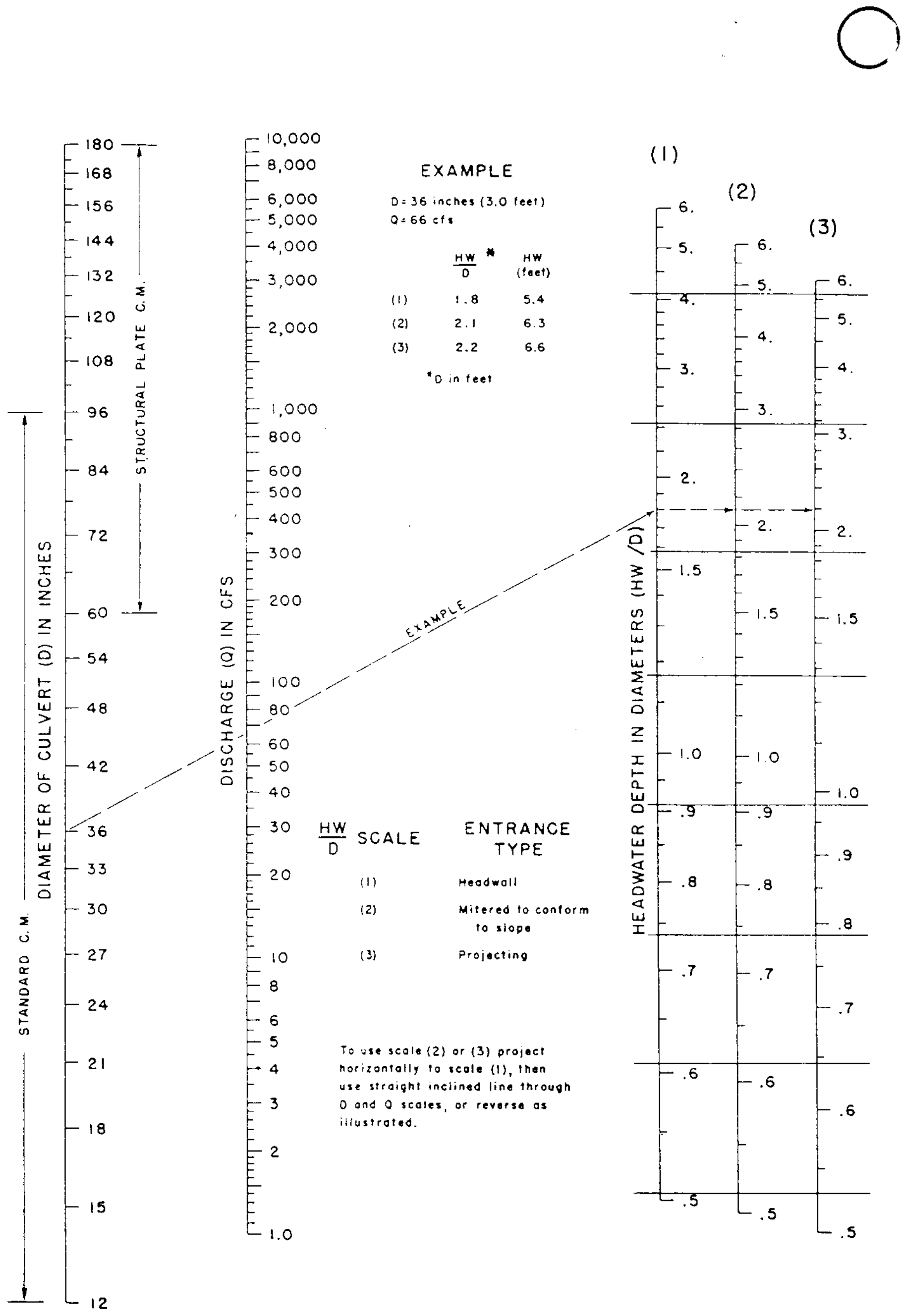


FIGURE 5.22
INLET CONTROL HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING
(USDOT, FHWA, HDS-5, 1985)

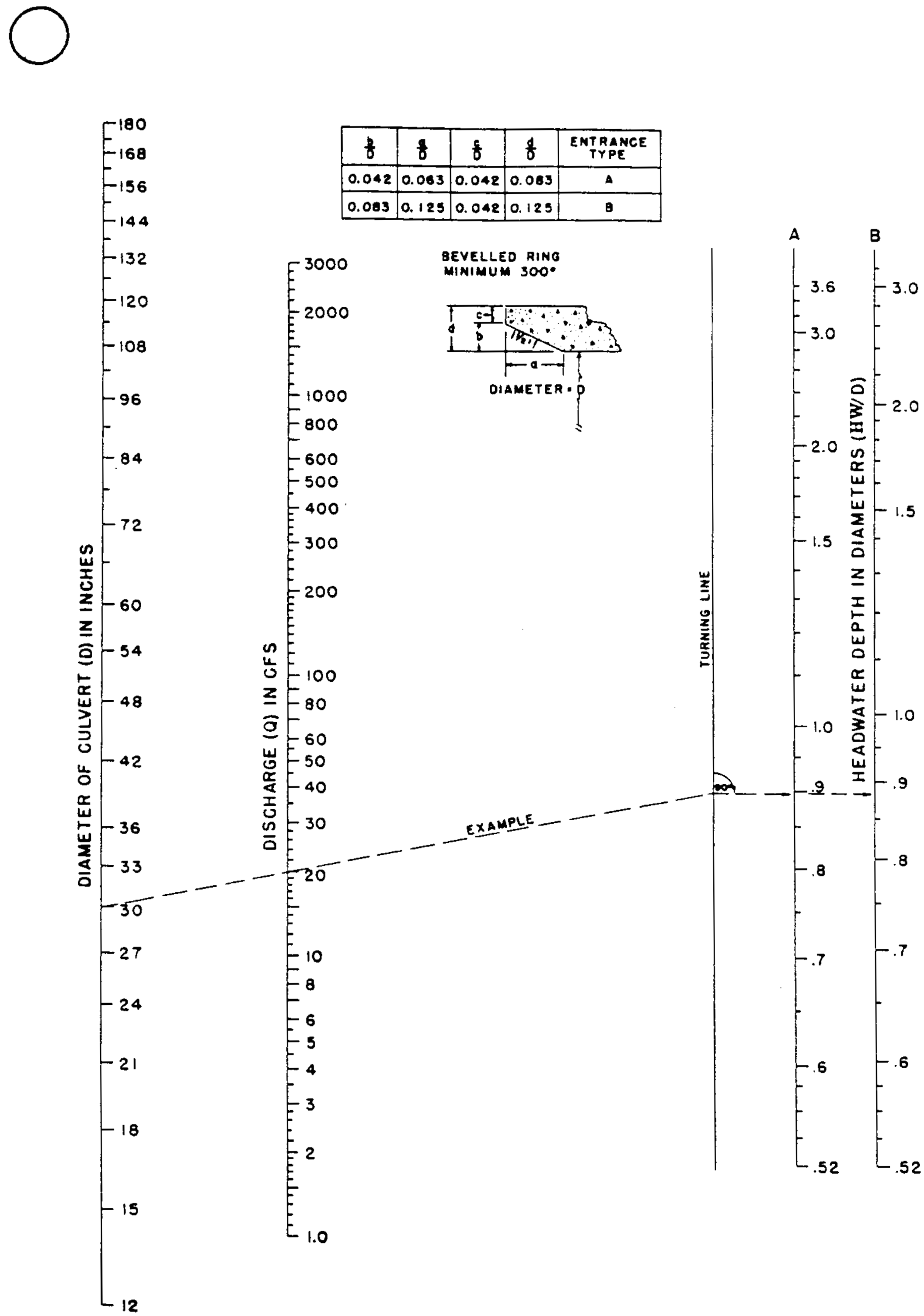


FIGURE 5.23
CRITICAL DEPTH FOR CIRCULAR PIPE
(USDOT, FHWA, HDS-5, 1985)

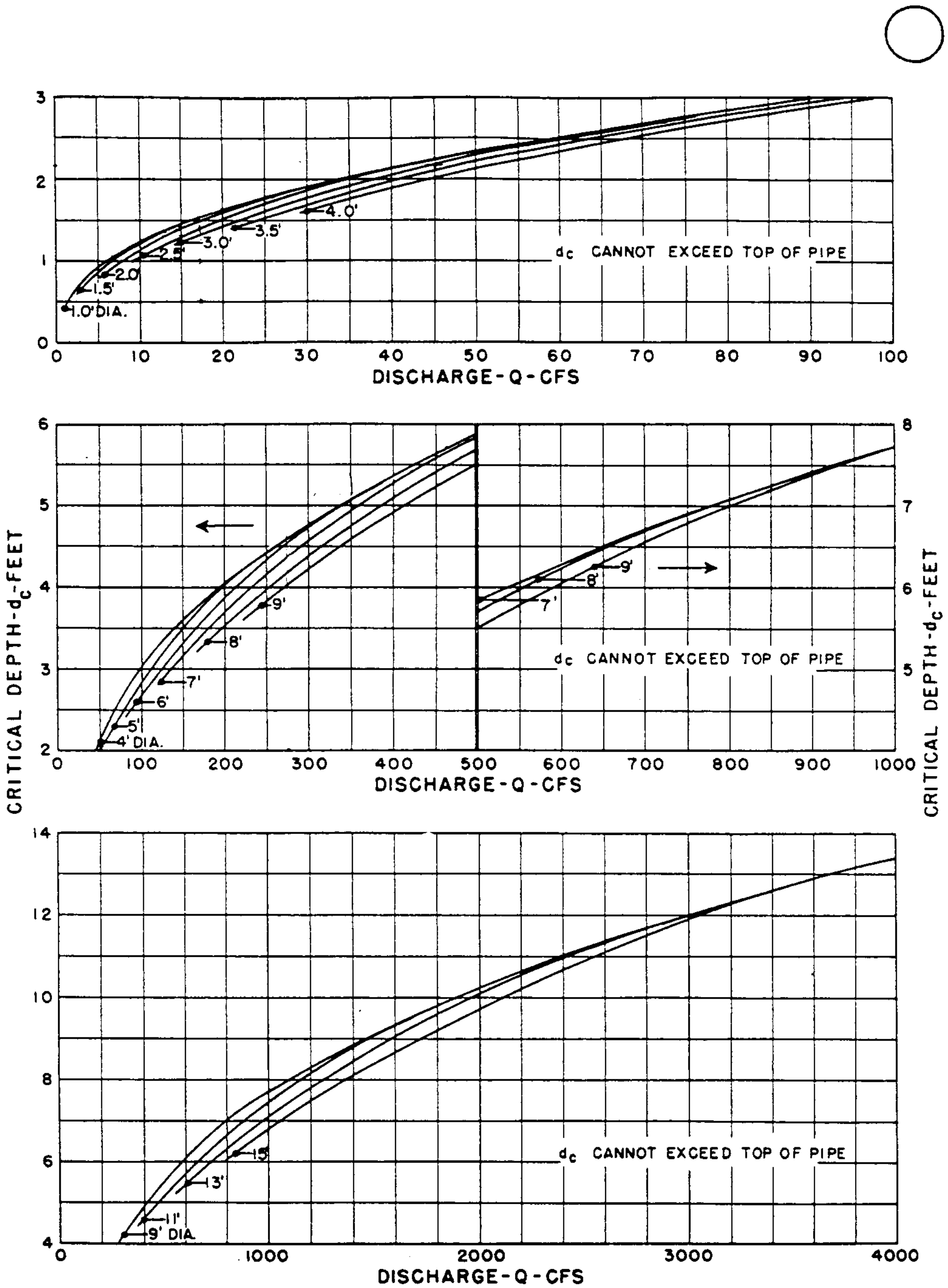


FIGURE 5.24
HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL
n = 0.012
(USDOT, FHWA, HDS-5, 1985)

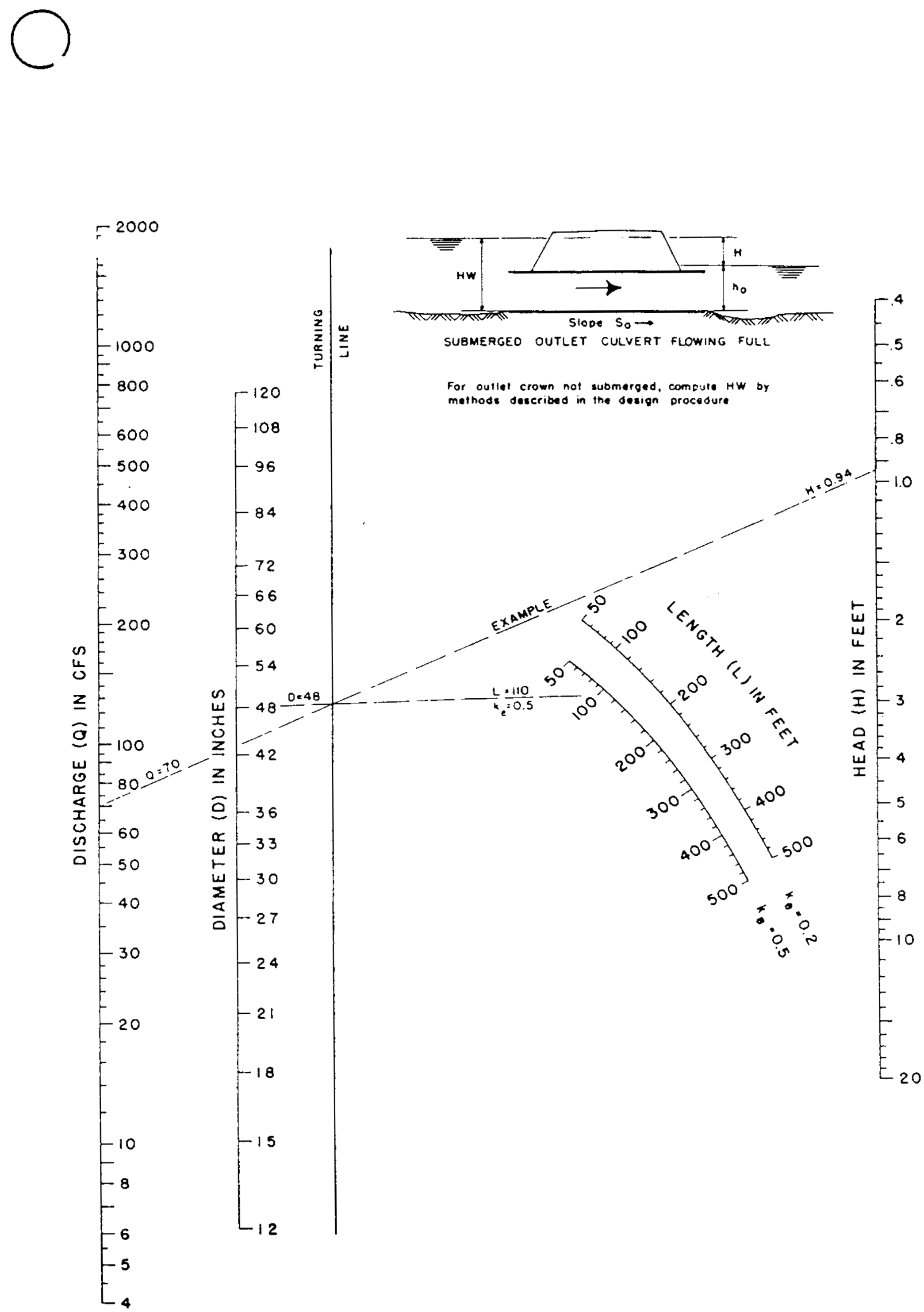


FIGURE 5.25
HEAD FOR C.M. PIPE CULVERTS FLOWING FULL
n = 0.024
(USDOT, FHWA, HDS-5, 1985)

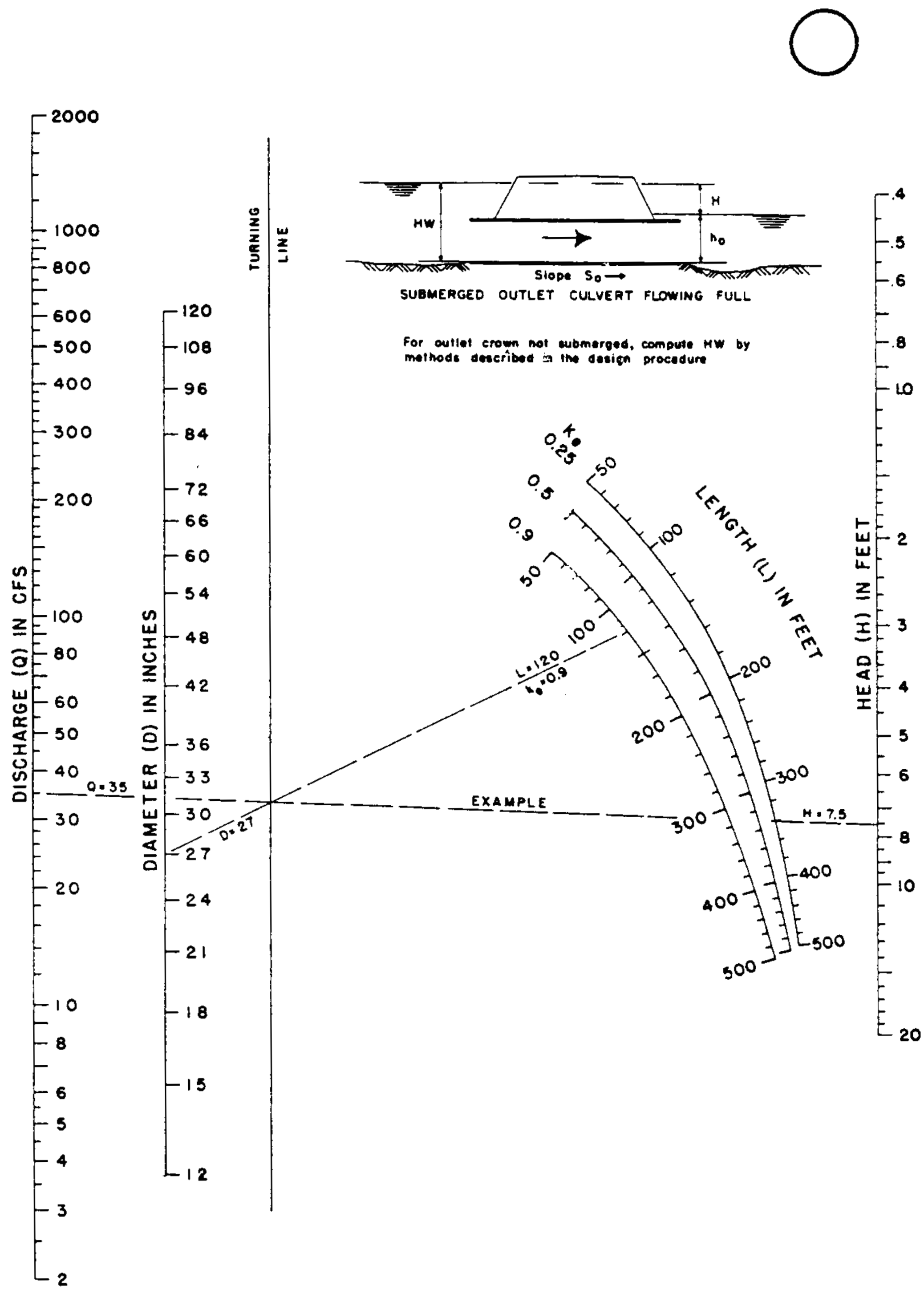


FIGURE 5.26
INLET CONTROL HEADWATER DEPTH FOR BOX CULVERTS
(USDOT, FHWA, HDS-5, 1985)

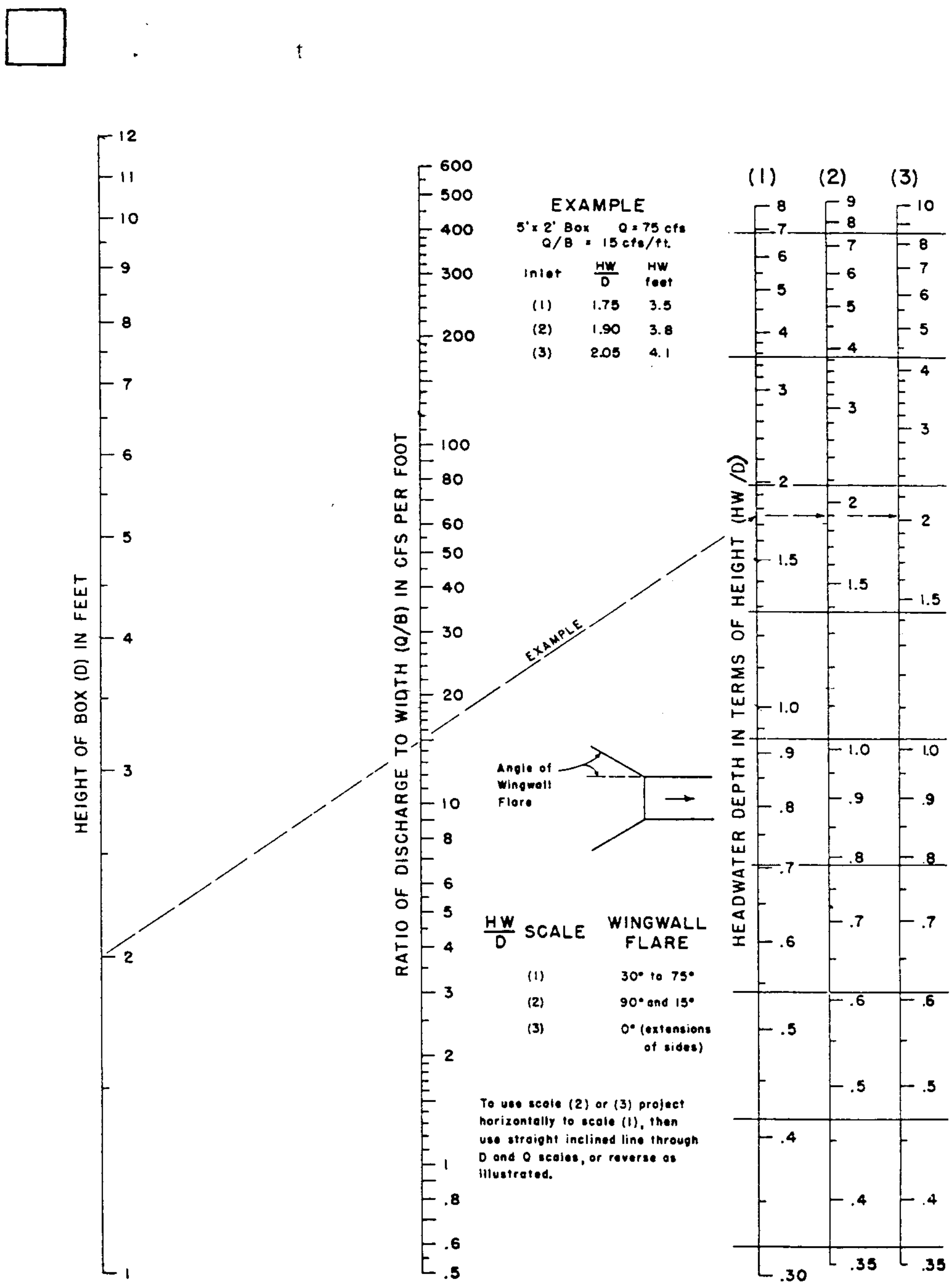


FIGURE 5.27

INLET CONTROL HEADWATER DEPTH FOR RECTANGULAR BOX CULVERT (FLARED WINGWALLS)

Flare Wingwalls (18^{o} to 33.7^{o}, and 45^{o}) and Beveled Edge at the Top of the Inlet

(USDOT, FHWA, HDS-5, 1985)

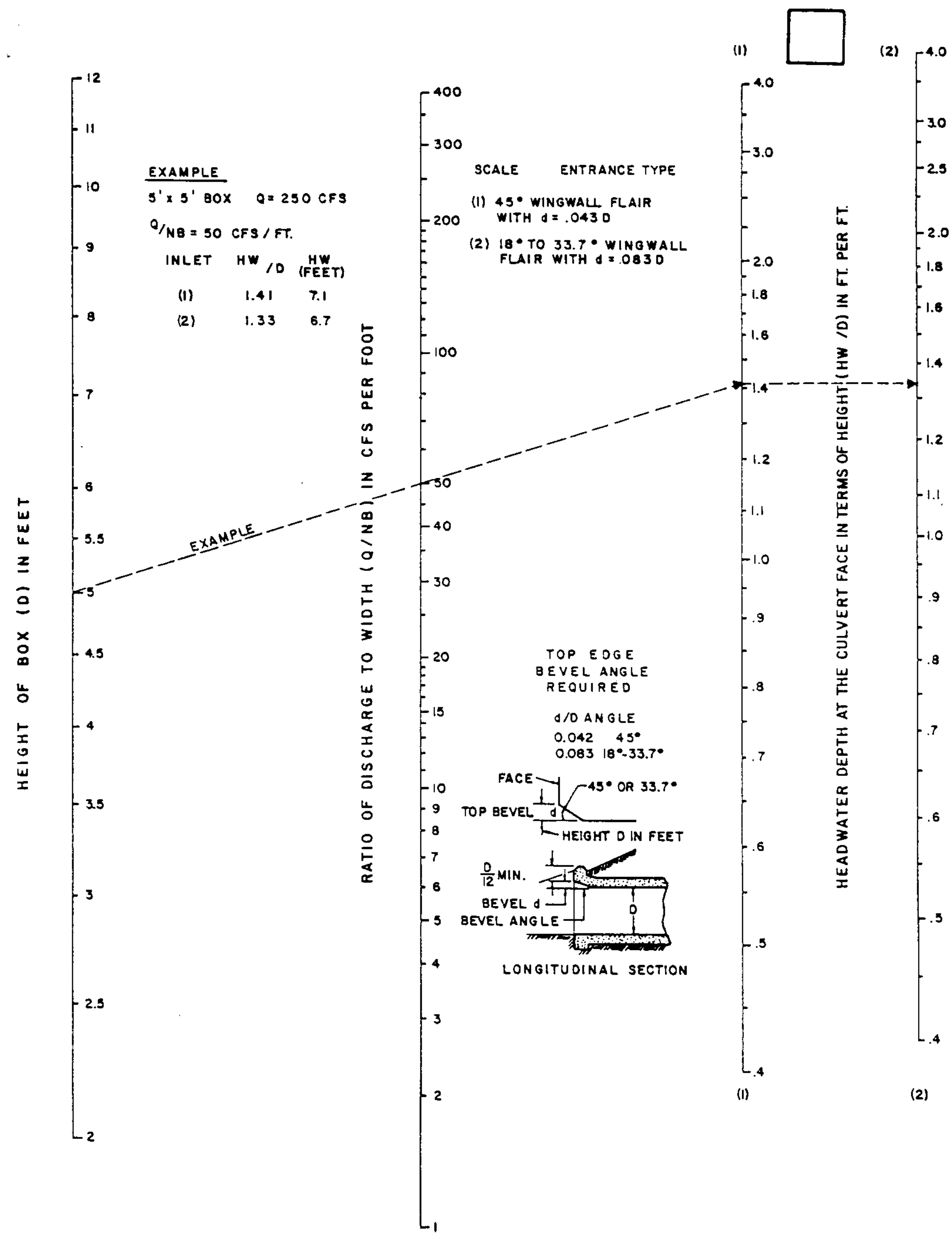


FIGURE 5.28

INLET CONTROL HEADWATER DEPTH FOR RECTANGULAR BOX CULVERT (90° HEADWALL)

90° Headwall - Chamfered or Beveled Inlet Edges

(USDOT, FHWA, HDS-5, 1985)

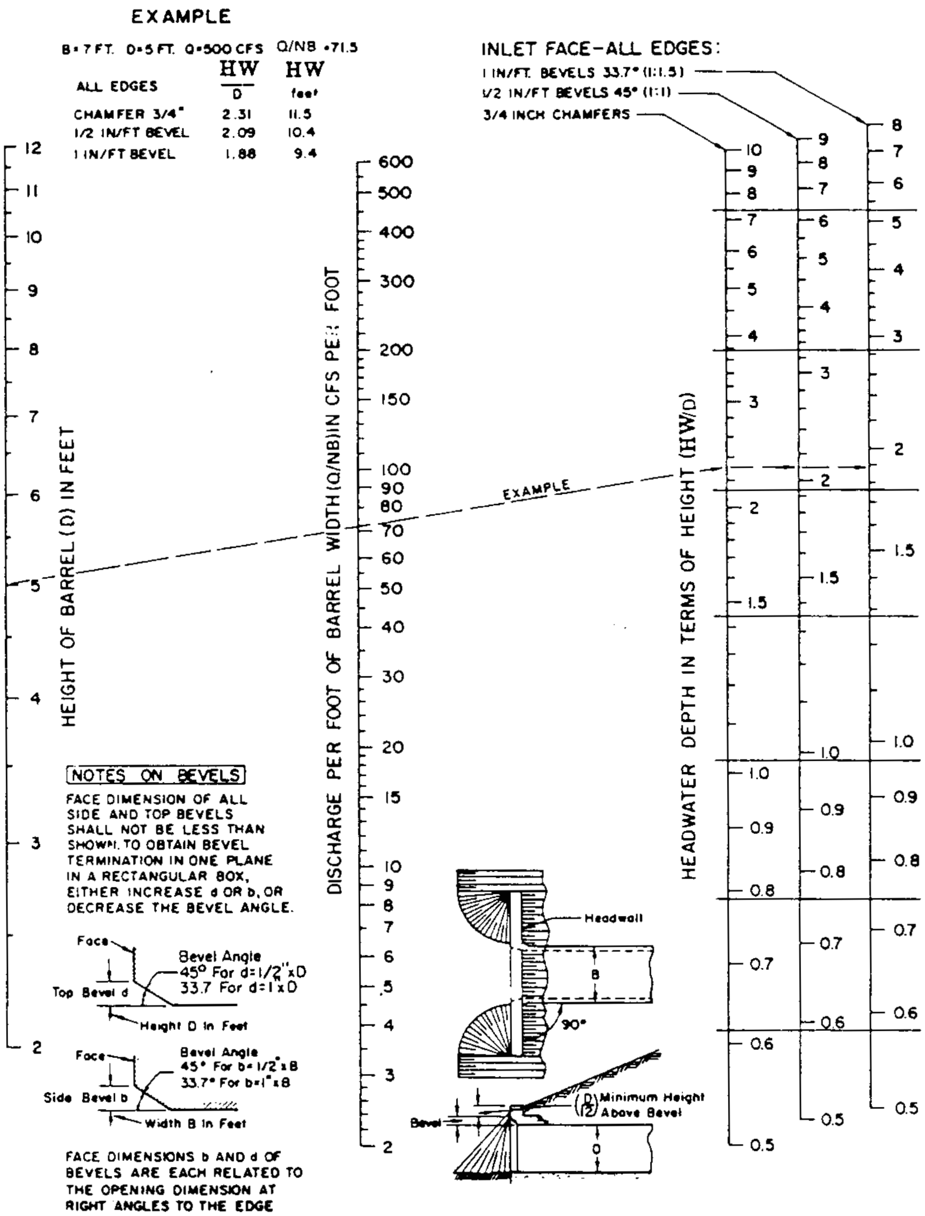
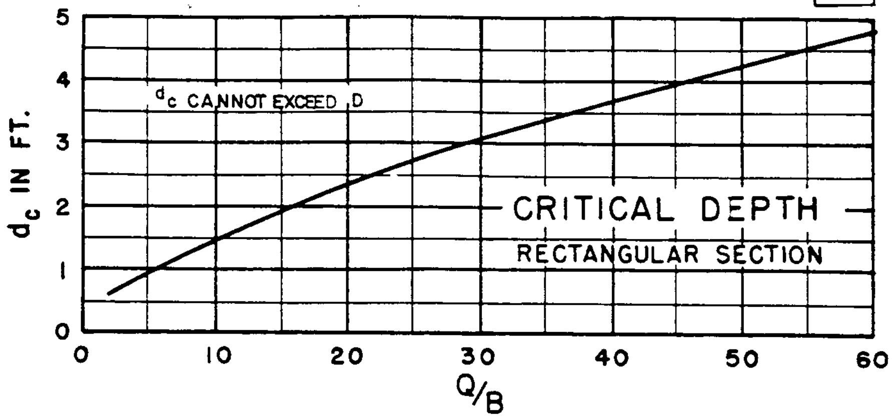


FIGURE 5.29
CRITICAL DEPTH RECTANGULAR SECTION
(USDOT, FHWA, HDS-5, 1985)



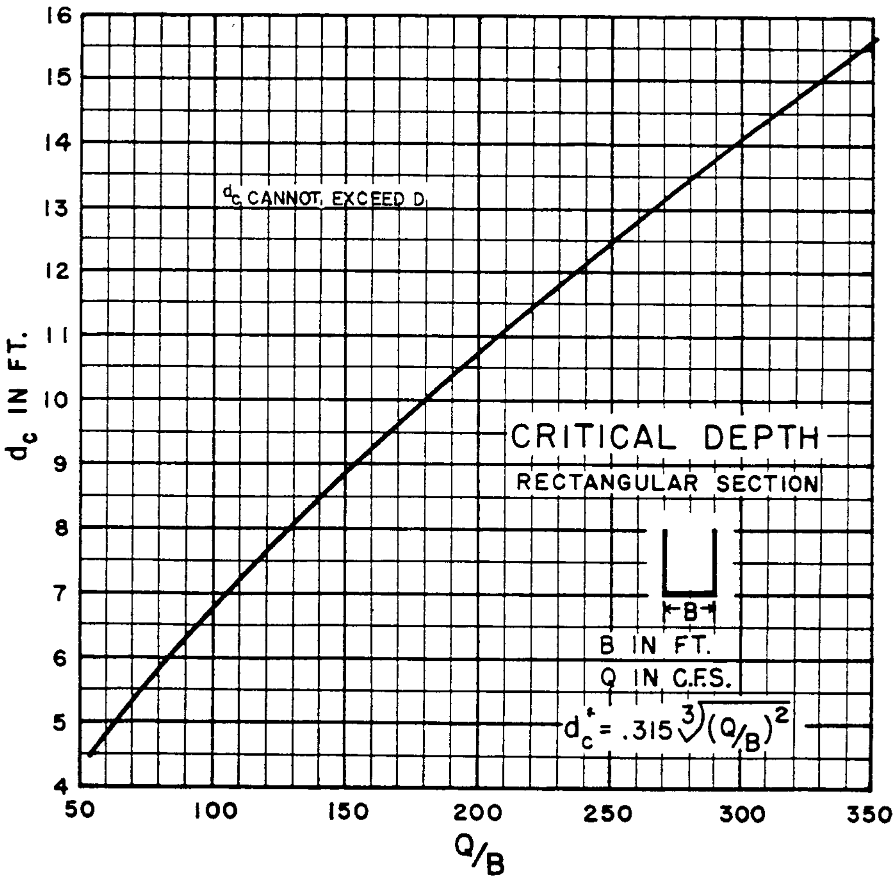


FIGURE 5.30
HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL
***n* = 0.012**
(USDOT, FHWA, HDS-5, 1985)

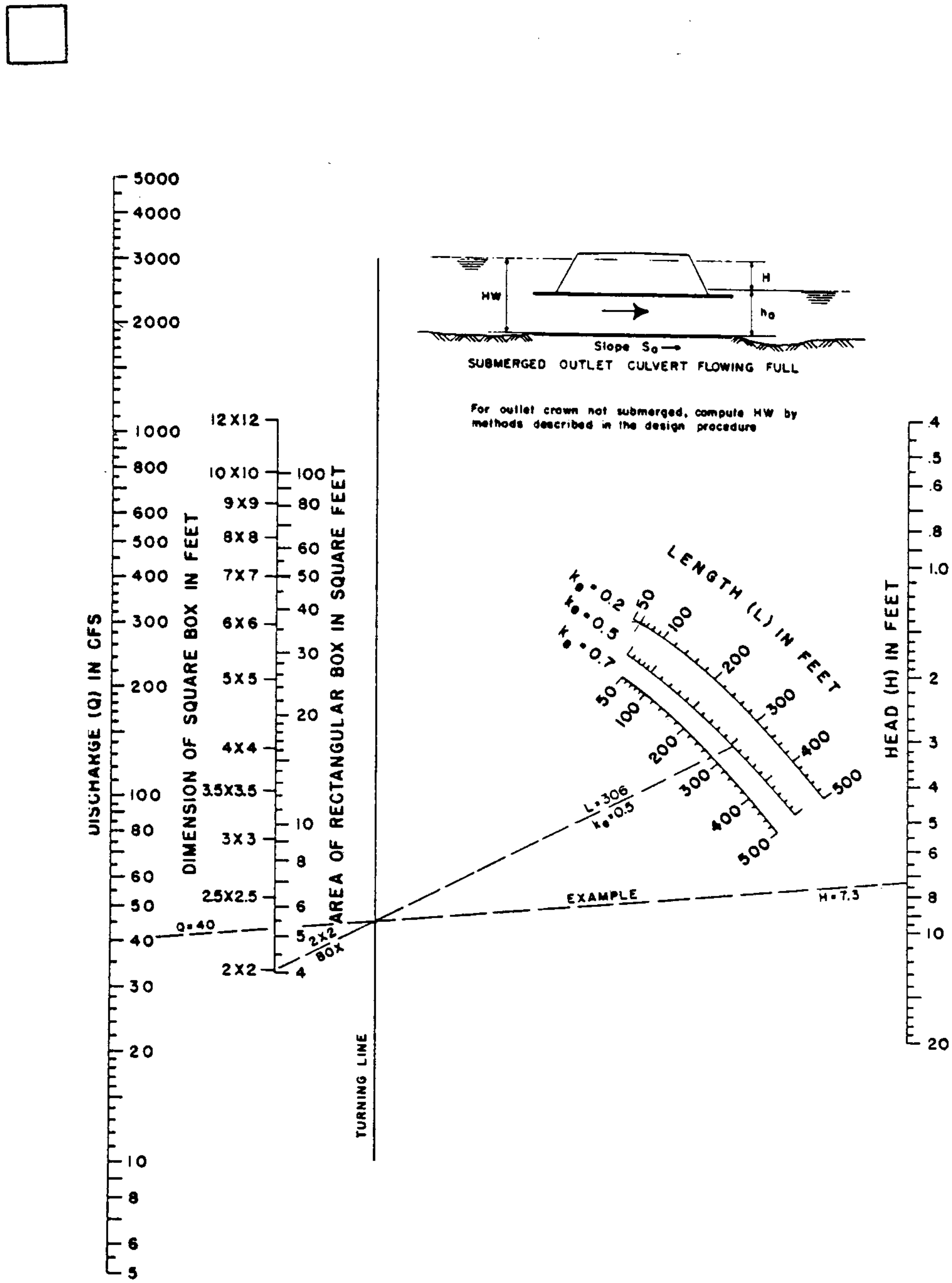


FIGURE 5.31
INLET CONTROL HEADWATER DEPTH FOR OVAL CONCRETE PIPE - LONG AXIS HORIZONTAL
(USDOT, FHWA, HDS-5, 1985)

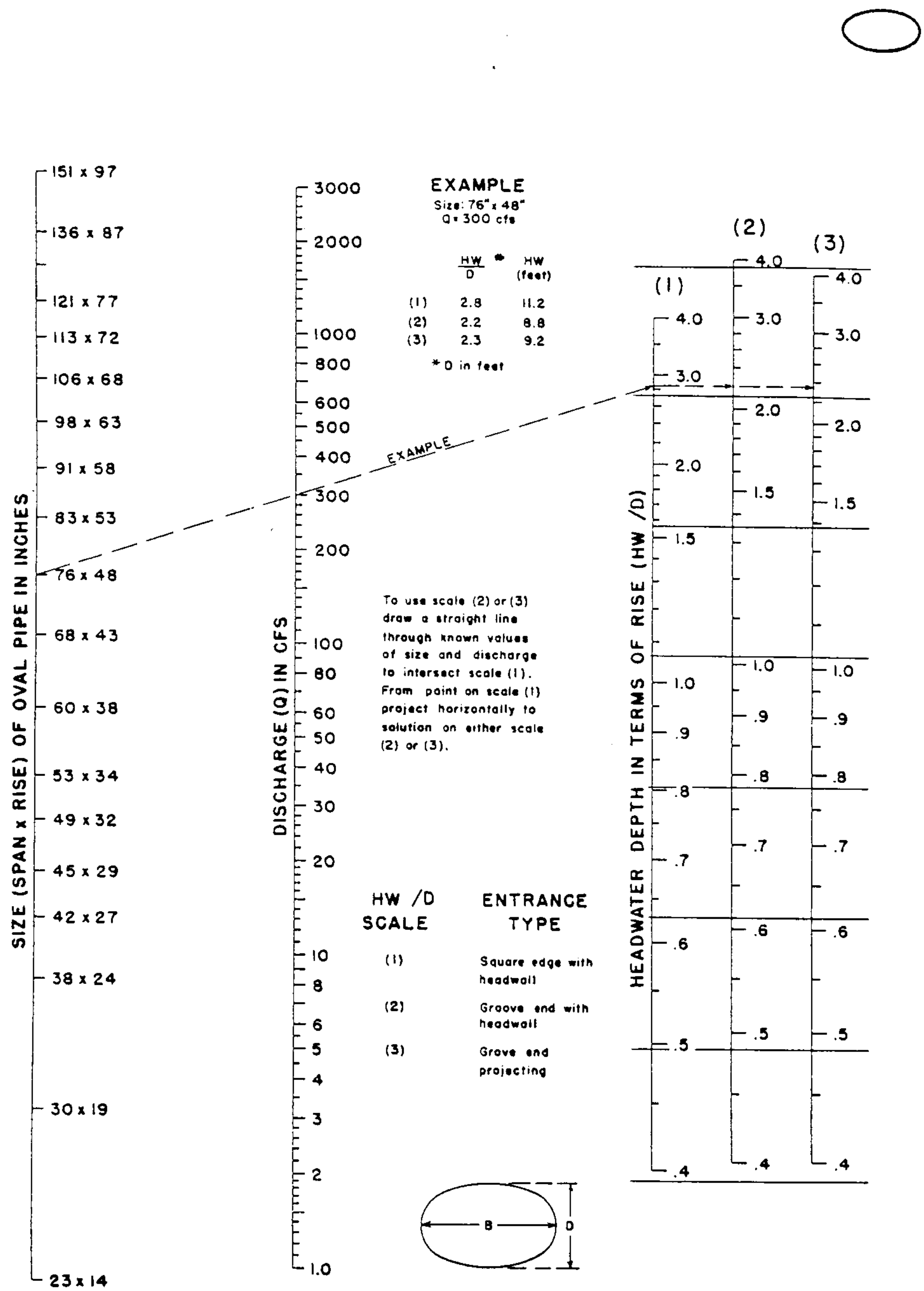


FIGURE 5.32
INLET CONTROL HEADWATER DEPTH FOR OVAL CONCRETE PIPE - LONG AXIS VERTICAL
(USDOT, FHWA, HDS-5, 1985)

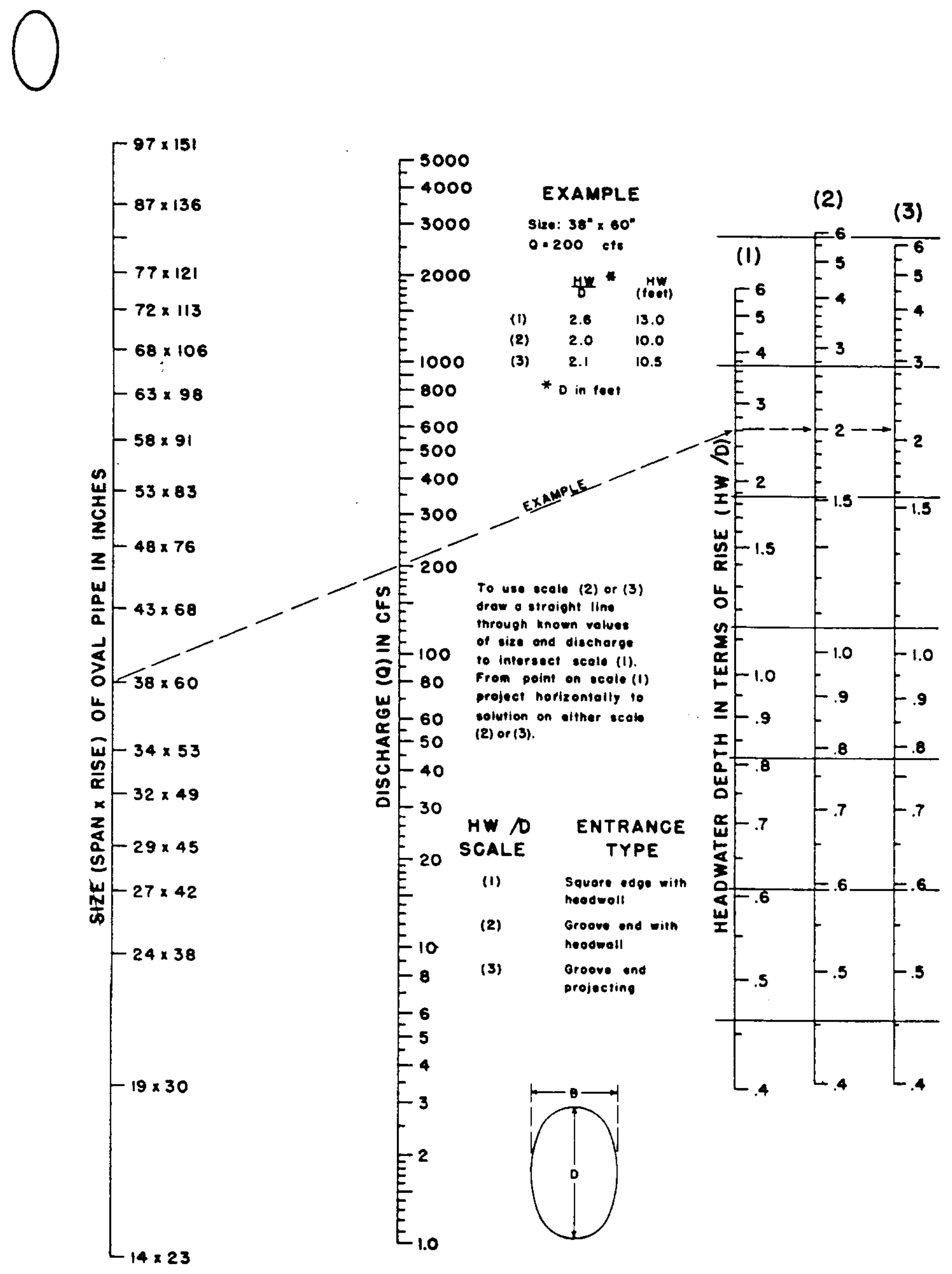


FIGURE 5.33
CRITICAL DEPTH FOR AN OVAL CONCRETE PIPE - LONG AXIS HORIZONTAL
(USDOT, FHWA, HDS-5, 1985)

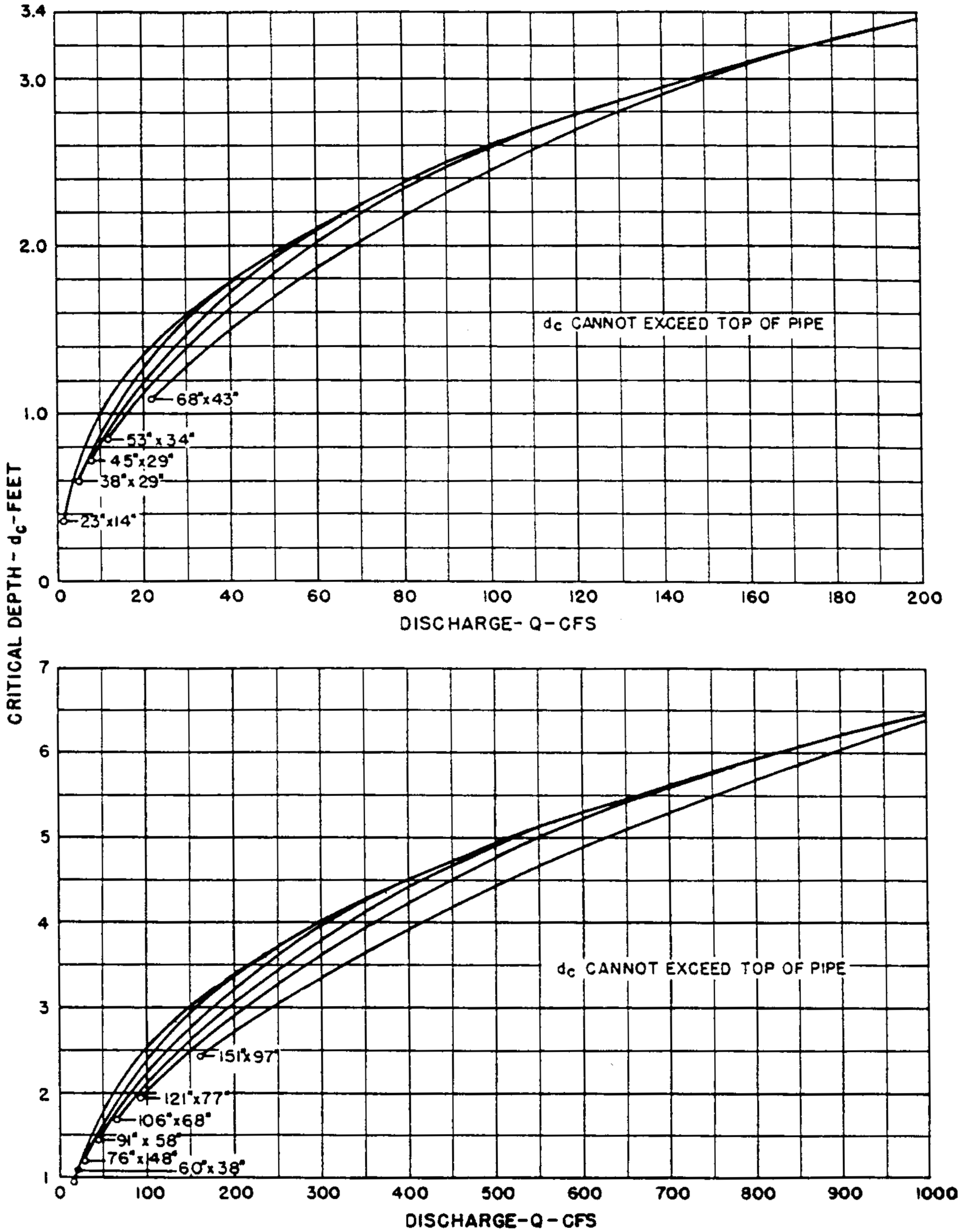


FIGURE 5.34
CRITICAL DEPTH FOR AN OVAL CONCRETE PIPE - LONG AXIS VERTICAL
(USDOT, FHWA, HDS-5, 1985)

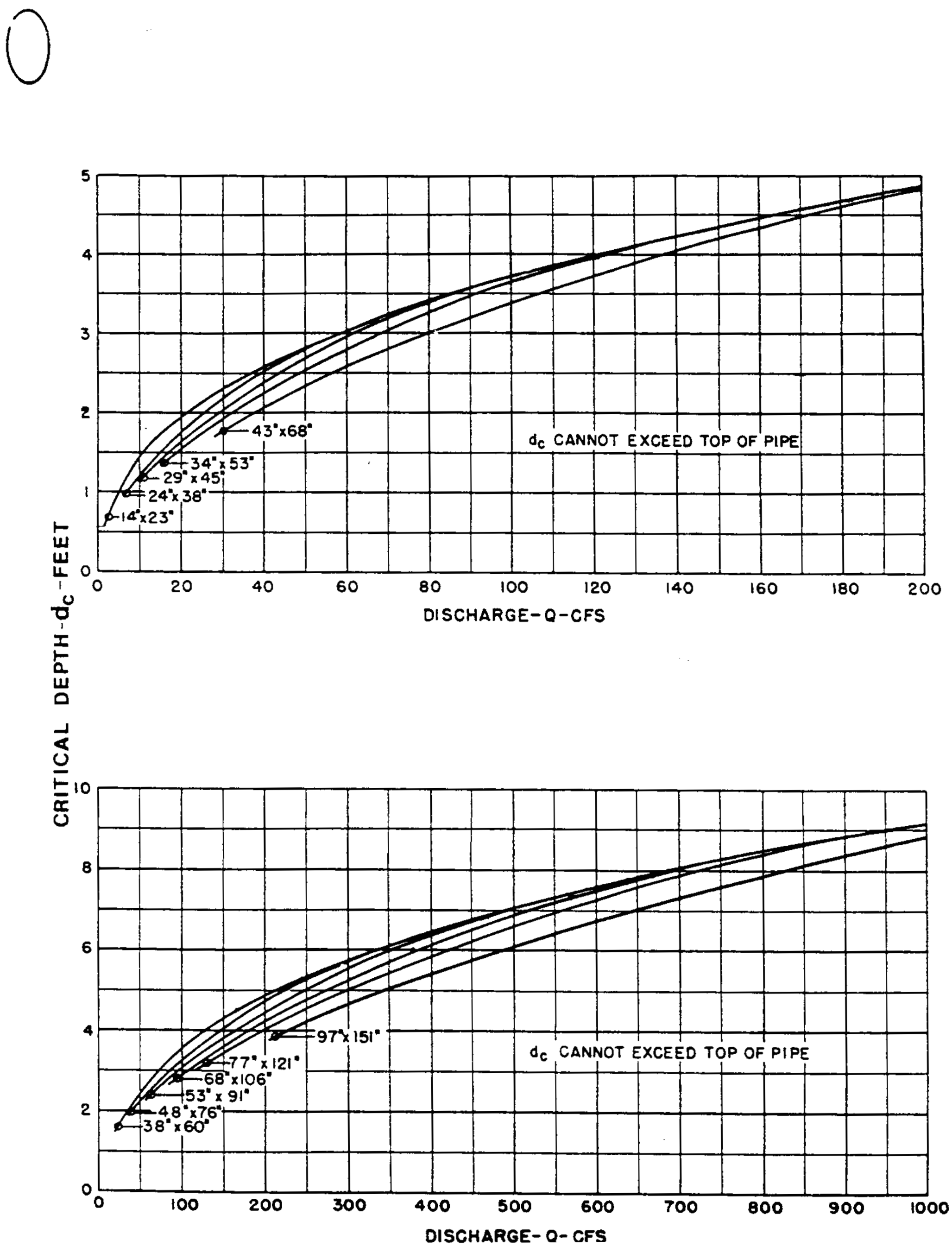


FIGURE 5.35
HEAD FOR CONCRETE PIPE FLOWING FULL - LONG AXIS HORIZONTAL OR VERTICAL
***n* = 0.012**
(USDOT, FHWA, HDS-5, 1985)

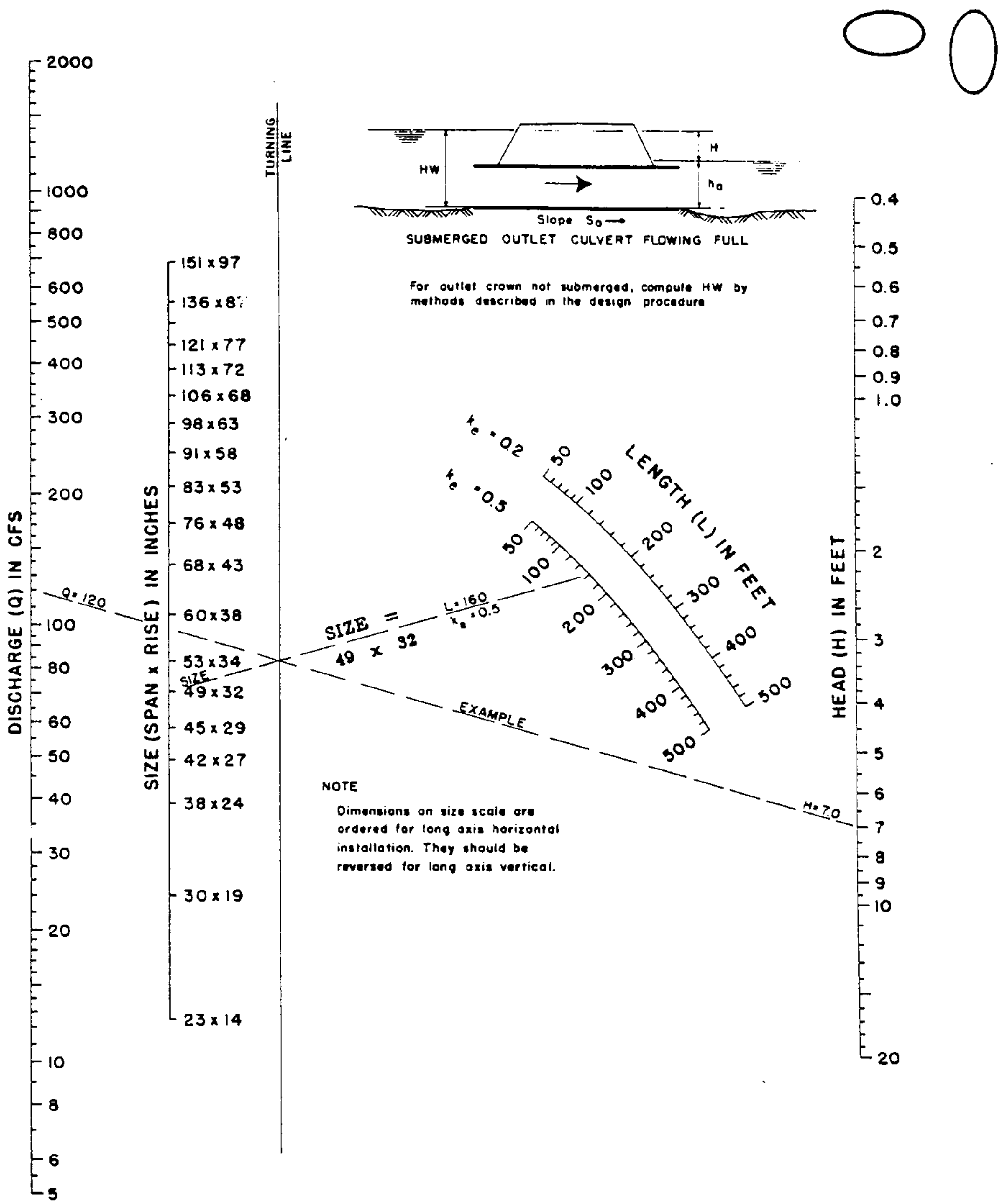


FIGURE 5.36
HEADWATER DEPTH FOR C.M. PIPE - ARCH CULVERT WITH INLET CONTROL
(USDOT, FHWA, HDS-5, 1985)

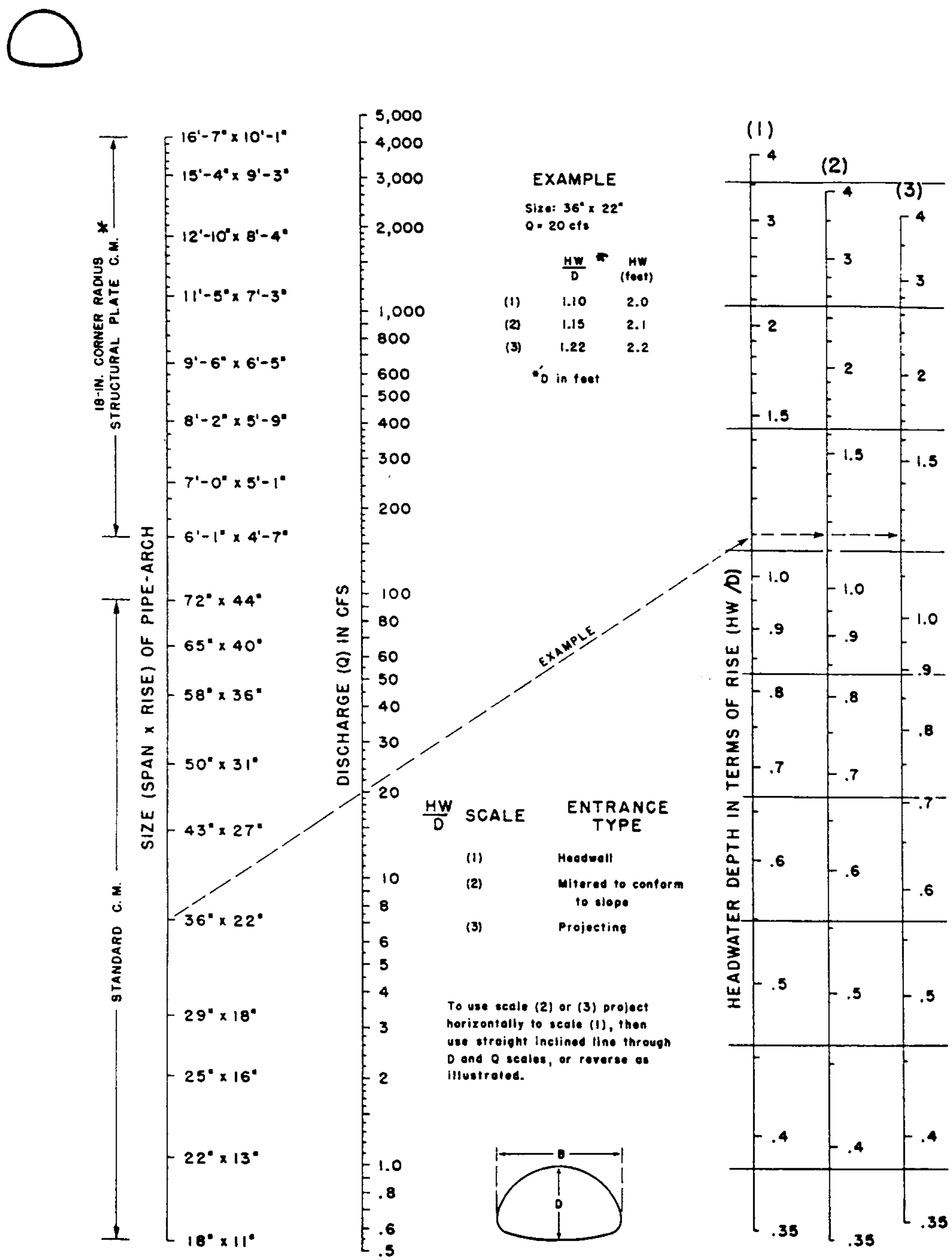
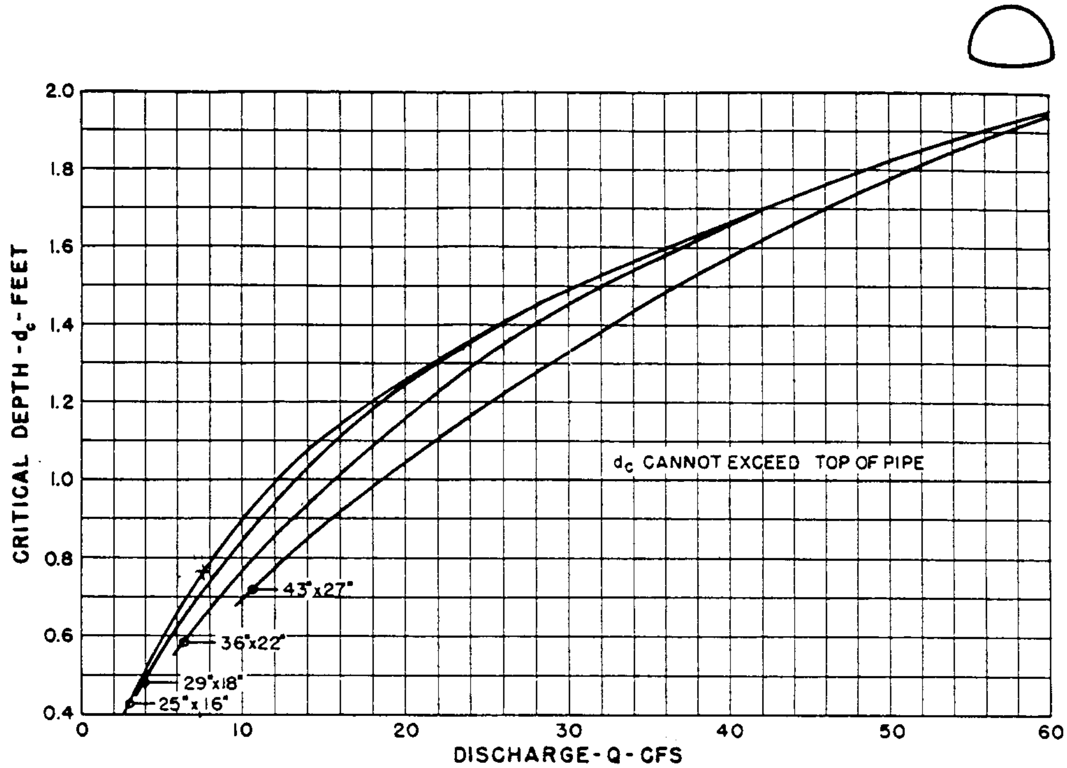


FIGURE 5.37
CRITICAL DEPTH FOR STANDARD C.M. PIPE - ARCH
(USDOT, FHWA, HDS-5, 1985)



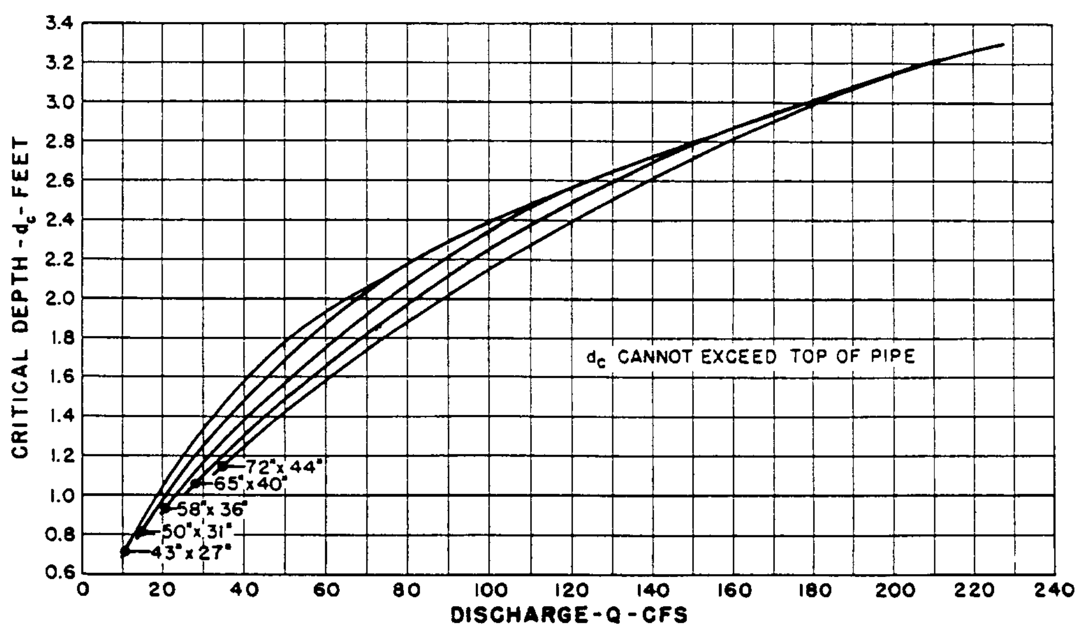
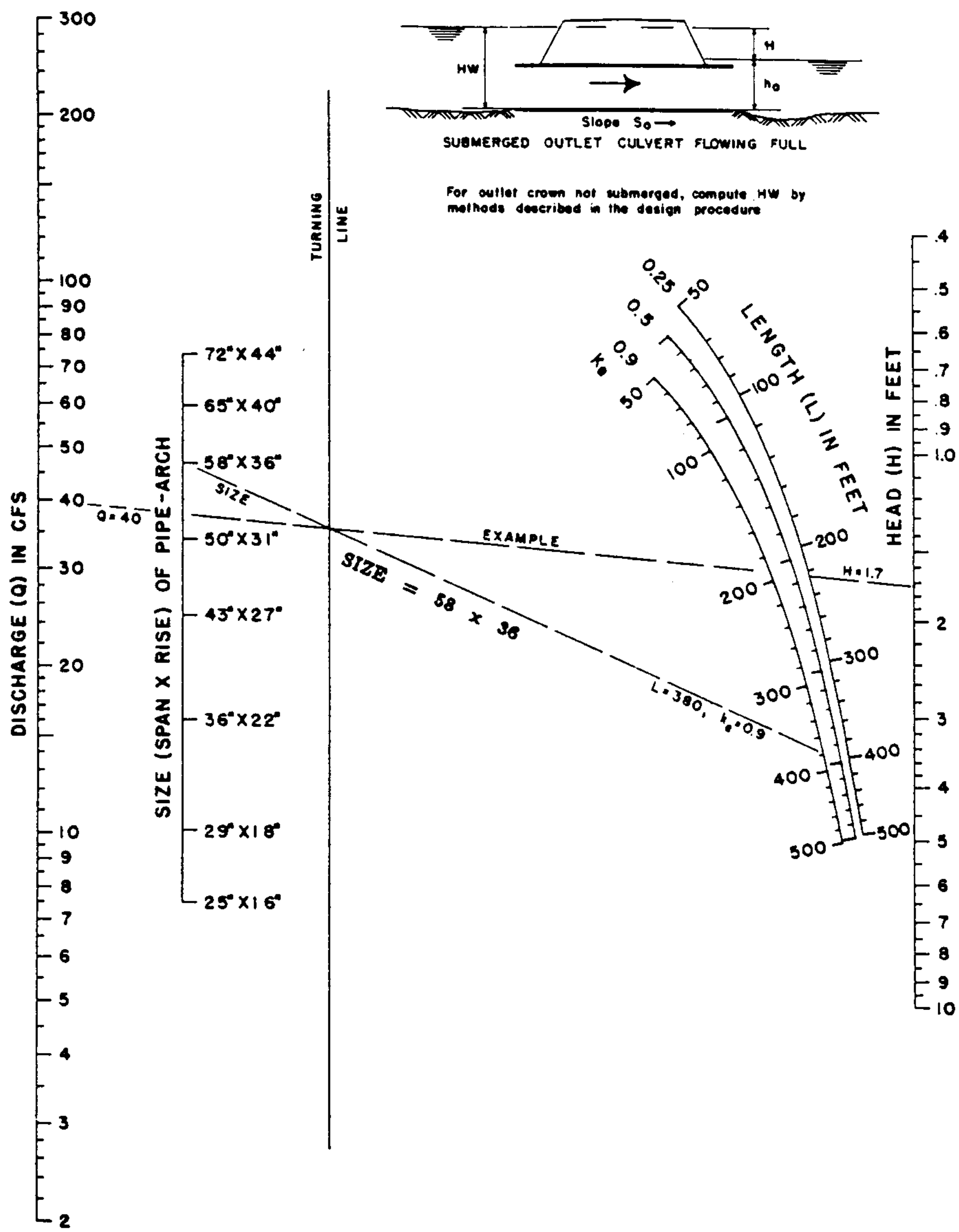


FIGURE 5.38
HEAD FOR STANDARD C.M. PIPE - ARCH CULVERTS FLOWING FULL
***n* = 0.024**
(USDOT, FHWA, HDS-5, 1985)



5.3.5 Design Examples

The following example problems are from *HDS-5* (USDOT, FHWA, 1985) and illustrate the use of the design methods and charts for selected culvert configurations and hydraulic conditions. The problems cover the following situations:

Example 1: Circular pipe culvert, CMP (standard 2-2/3 by 1/2 inch corrugations) with beveled edge or reinforced concrete pipe with groove end. No FALL.

Example 2: Reinforced cast-in-place concrete box culvert with square edges and with bevels. No FALL.

Example 3: Elliptical pipe culvert with groove end and a FALL.

Example 4: Roadway overtopping calculations and performance curve development.

Example 1

A culvert at a new roadway crossing must be designed to pass the 25-year flood. Hydrologic analysis indicates a peak flow rate of 200 cfs. Use the following site information:

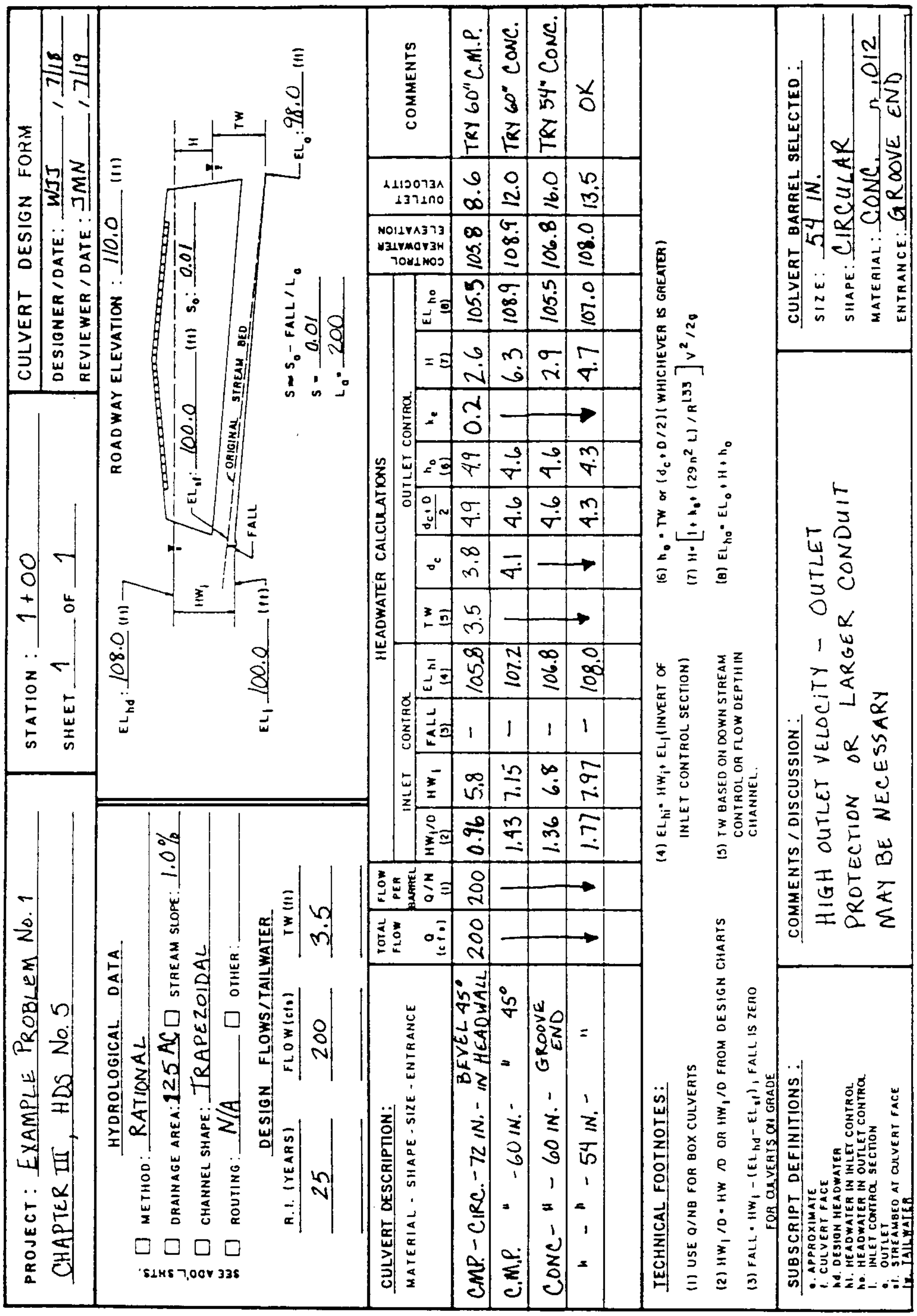
- Elevation of stream bed at Culvert Face: 100 ft
- Natural Stream Bed Slope: 1 percent = 0.01 ft/ft
- Tailwater for 25-Year Flood: 3.5 ft
- Approximate Culvert Length: 200 ft
- Shoulder Elevation: 110 ft

Design a circular pipe culvert for this site. Consider the use of a corrugated metal pipe with standard 2-2/3 by 1/2 inch corrugations and a headwall with beveled edges, and concrete pipe with a groove end, projecting. Base the design headwater on the shoulder elevation with a 2-foot freeboard (elevation 108.0 ft). Set the inlet invert at the natural streambed elevation (no FALL).

Figure 5.39 represents a completed Culvert Design Form for this example. Notice the headwater depth of 8 feet at the inlet. The designer should verify that backwater from the culvert will not present a hazard to upstream facilities and that flow will not be diverted into another watercourse. An easement may be necessary for ponding on private property. Notice the high estimated outlet velocity of 13.5 fps. The designer should provide outlet erosion control in conformance with Section 5.4.3 or Section 8.4, or investigate other culvert options such as a larger pipe size or multiple smaller pipes. When making this decision, the designer should consider the geometry and allowable velocity of the receiving channel to be sure that the selected pipe or pipes are appropriate given the width and depth of the receiving channel. The design should not result in erosion of the bed, banks or overbanks of the downstream system.

Note: Figure 5.20, Figure 5.21, Figure 5.23, Figure 5.24, Figure 5.25 and Table 5.1 were used in this example.

FIGURE 5.39
EXAMPLE 1 CULVERT DESIGN FORM
(USDOT, FHWA, HDS-5, 1985)



PROJECT: EXAMPLE PROBLEM No. 1
CHAPTER III, HDS No. 5

STATION: 1+00
SHEET 1 OF 1

CULVERT DESIGN FORM
DESIGNER / DATE: WJJ / 7/18
REVIEWER / DATE: JMN / 7/19

HYDROLOGICAL DATA

SEE ADD'L SHTS.

☐ METHOD: RATIONAL
☐ DRAINAGE AREA: 125 AC ☐ STREAM SLOPE: 1.0%
☐ CHANNEL SHAPE: TRAPEZOIDAL
☐ ROUTING: N/A ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
25	200	3.5

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	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
CMP - CIRC. - 72 IN. - BEVEL 45° IN HEADWALL	200	200	0.96	5.8	–	105.8	3.5	3.8	4.9	4.9	0.2	2.6	105.5	105.8	8.6	TRY 60" C.M.P.
C.M.P. " - 60 IN. - " 45°			1.43	7.15	–	107.2		4.1	4.6	4.6		6.3	108.9	108.9	12.0	TRY 60" CONC.
CONC. - " - 60 IN. - GROOVE END			1.36	6.8	–	106.8			4.6	4.6		2.9	105.5	106.8	16.0	TRY 54" CONC.
" - " - 54 IN. - "	↓	↓	1.77	7.97	–	108.0	↓	↓	4.3	4.3	↓	4.7	107.0	108.0	13.5	OK

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) FALL = $HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

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

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:
a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:
HIGH OUTLET VELOCITY - OUTLET PROTECTION OR LARGER CONDUIT MAY BE NECESSARY

CULVERT BARREL SELECTED:
SIZE: 54 IN.
SHAPE: CIRCULAR
MATERIAL: CONC. n .012
ENTRANCE: GROOVE END

Example 2

A new culvert at a roadway crossing is required to pass a 50-year flow rate of 300 cfs. Use the following site conditions:

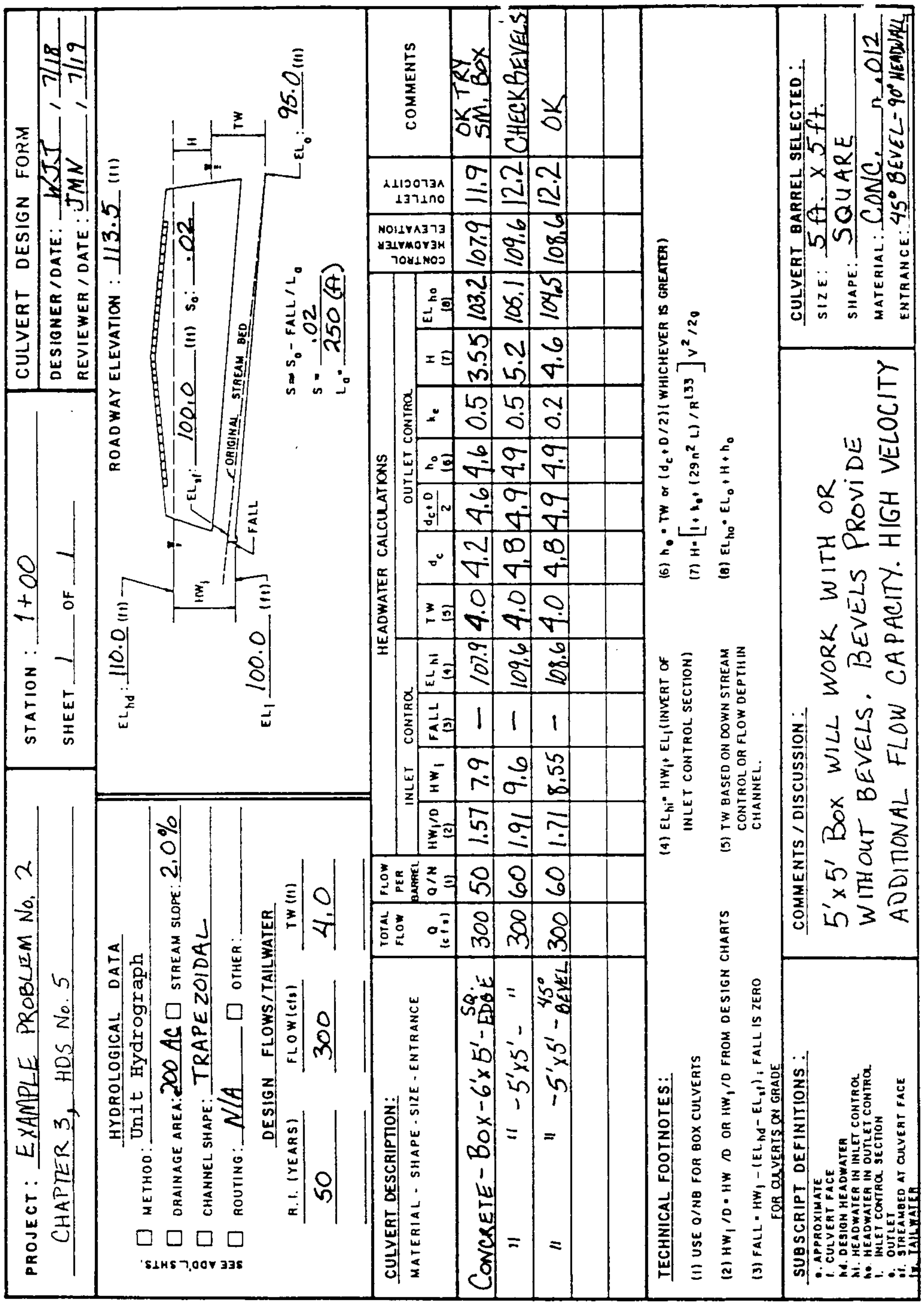
- EL_{hd} : 110 ft based on adjacent structures
- Shoulder Elevation: 113.5 ft
- Elevation of Streambed at Culvert Face (EL_{sf}): 100 ft
- Natural Stream Slope: 2 percent
- Tailwater Depth: 4.0 ft
- Approximate Culvert Length: 250 ft

Design a reinforced concrete box culvert for this installation. Try both square edges and 45 degree beveled edges in a 90^{o} headwall. Do not depress the inlet (no FALL).

Figure 5.40 represents a completed Culvert Design form for Problem No. 2. Notice the headwater depth of 10 feet at the inlet. The designer should verify that backwater from the culvert will not present a hazard to upstream facilities and that flow will not be diverted into another watercourse. An easement may be necessary for ponding on private property. Notice the high estimated outlet velocity of 12.2 fps. The designer should provide outlet erosion control in conformance with Section 5.4.3 or Section 8.4, or investigate other culvert options such as a larger pipe size or multiple smaller pipes. When making this decision, the designer should consider the geometry and allowable velocity of the receiving channel to be sure that the selected pipe or pipes are appropriate given the width and depth of the receiving channel. The design should not result in erosion of the bed, banks or overbanks of the downstream system.

Note: Figure 5.26, Figure 5.28, Figure 5.29, Figure 5.30, and Table 5.1 are used in this solution.

FIGURE 5.40
EXAMPLE 2 CULVERT DESIGN FORM
(USDOT, FHWA, HDS-5, 1985)



PROJECT: EXAMPLE PROBLEM No. 2
CHAPTER 3, HDS No. 5

STATION: 1+00
SHEET 1 OF 1

CULVERT DESIGN FORM
DESIGNER/DATE: WJT / 7/18
REVIEWER/DATE: JMN / 7/19

HYDROLOGICAL DATA
SEE ADD'L SHTS.
☐ METHOD: Unit Hydrograph
☐ DRAINAGE AREA: 200 AC ☐ STREAM SLOPE: 2.0%
☐ CHANNEL SHAPE: TRAPEZOIDAL
☐ ROUTING: N/A ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
50	300	4.0

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	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
CONCRETE - BOX - 6'x5' - SQ. EDGE	300	50	1.57	7.9	—	107.9	4.0	4.2	4.6	4.6	0.5	3.55	103.2	107.9	11.9	OK TRY SM. BOX
" " - 5'x5' - "	300	60	1.91	9.6	—	109.6	4.0	4.8	4.9	4.9	0.5	5.2	105.1	109.6	12.2	CHECK BEVELS
" " - 5'x5' - 45° BEVEL	300	60	1.71	8.55	—	108.6	4.0	4.8	4.9	4.9	0.2	4.6	104.5	108.6	12.2	OK

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) FALL = $HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) h_o = TW or $(d_c + D/2)$ (WHICHEVER IS GREATER)

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

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:
a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:
5'x5' BOX WILL WORK WITH OR WITHOUT BEVELS. BEVELS PROVIDE ADDITIONAL FLOW CAPACITY. HIGH VELOCITY

CULVERT BARREL SELECTED:
SIZE: 5 ft. x 5 ft.
SHAPE: SQUARE
MATERIAL: CONC. n .012
ENTRANCE: 45° BEVEL - 90° HEADWALL

Example 3

Design a culvert to pass a 25-year flow of 180 cfs. Minimum depth of cover for this culvert is 2 feet.

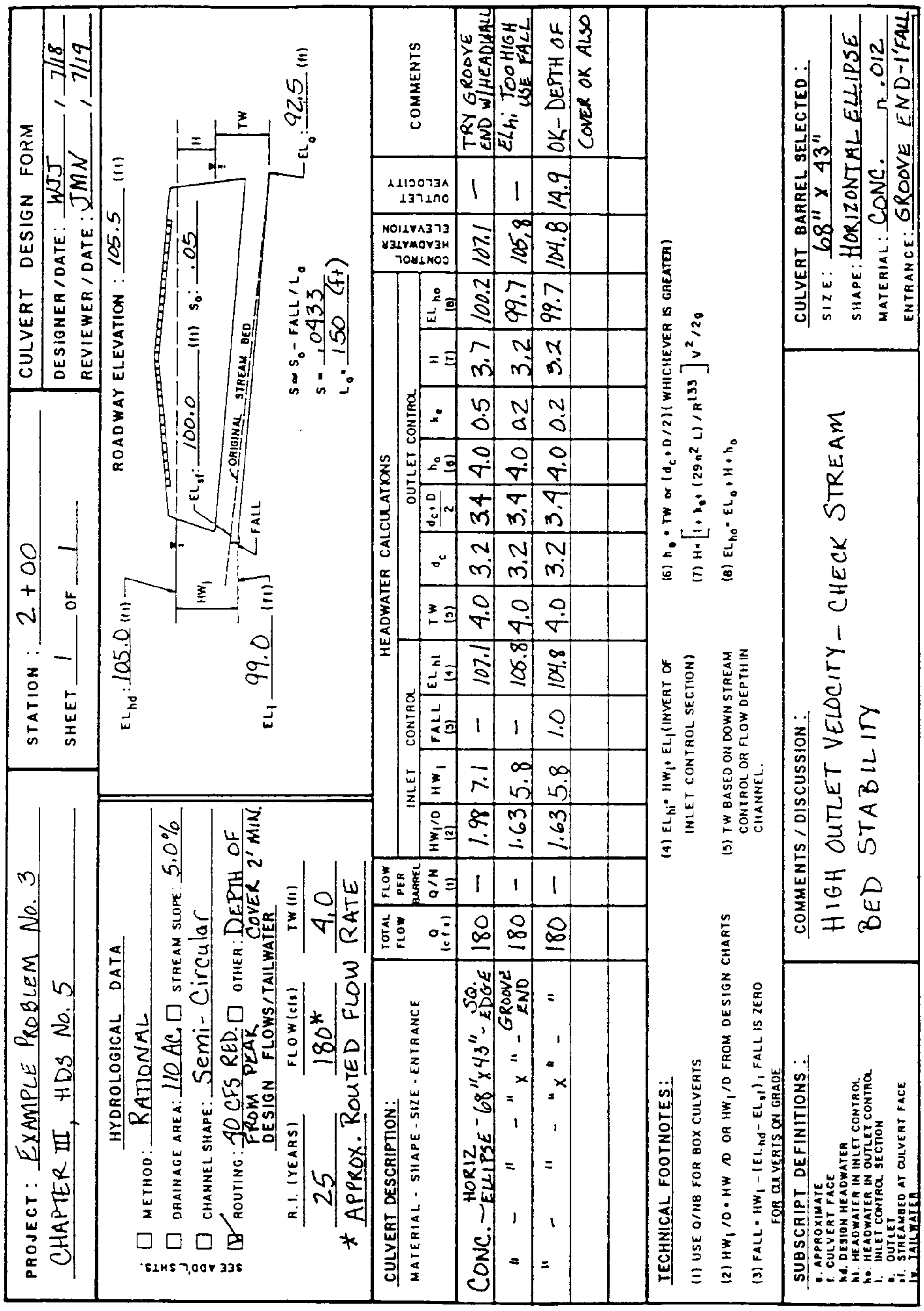
- EL_{hd}: 105 ft based on adjacent structures
- Shoulder Elevation: 105.5 ft
- Elevation of Streambed at Culvert Face (EL_{sf}): 100 ft.
- Original Stream Slope: 5 percent
- Tailwater Depth: 4 ft
- Approximate Culvert Length: 150 ft

Due to the low available cover over the conduit, use a horizontal elliptical concrete pipe. This example allows a small depression (FALL) of about 1 ft at the inlet to demonstrate how FALL is applied. Use of FALL in streams carrying a heavy sediment load, which is the case for most of Maricopa County, is not recommended.

Refer to Figure 5.41 for a completed Culvert Design Form for this problem.

Note: Figure 5.31, Figure 5.33, Figure 5.35, and Table 5.1 are used in this solution.

FIGURE 5.41
EXAMPLE 3 CULVERT DESIGN FORM
(USDOT, FHWA, HDS-5, 1985)



PROJECT: EXAMPLE PROBLEM No. 3
CHAPTER III, HDS No. 5

STATION: 2 + 00
SHEET 1 OF 1

CULVERT DESIGN FORM
DESIGNER / DATE: WJJ / 7/18
REVIEWER / DATE: JMN / 7/19

HYDROLOGICAL DATA
SEE ADD'L SHTS.
☐ METHOD: RATIONAL
☐ DRAINAGE AREA: 110 AC ☐ STREAM SLOPE: 5.0%
☐ CHANNEL SHAPE: Semi-Circular
☒ ROUTING: 40 CFS RED. FROM PEAK ☐ OTHER: DEPTH OF COVER 2' MIN.

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)
25	180*	4.0

* APPROX. ROUTED FLOW RATE

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	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
CONC. - HORIZ. ELLIPSE - 68"x43" - SQ. EDGE	180	—	1.98	7.1	—	107.1	4.0	3.2	3.4	4.0	0.5	3.7	100.2	107.1	—	TRY GROOVE END W/HEADWALL
" - " - " x " - GROOVE END	180	—	1.63	5.8	—	105.8	4.0	3.2	3.4	4.0	0.2	3.2	99.7	105.8	—	EL_{hi} TOO HIGH USE FALL
" - " - " x " - "	180	—	1.63	5.8	1.0	104.8	4.0	3.2	3.4	4.0	0.2	3.2	99.7	104.8	14.9	OK - DEPTH OF COVER OK ALSO

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) FALL = $HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) h_o = TW or $(d_c + D/2)$ (WHICHEVER IS GREATER)

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

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:
a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:
HIGH OUTLET VELOCITY - CHECK STREAM BED STABILITY

CULVERT BARREL SELECTED:
SIZE: 68" x 43"
SHAPE: HORIZONTAL ELLIPSE
MATERIAL: CONC. n .012
ENTRANCE: GROOVE END - 1' FALL

Example 4

Develop a performance curve for the installation in Figure 5.42 below, including roadway overtopping up to 0.5 feet above the roadway. Use the following dimensions:

Tailwater Channel:	
Flow, cfs	***TW*, ft**
50	101.8
100	102.6
150	103.1
200	103.5
250	103.8
300	104.2
350	104.4

Figure 5.21, Figure 5.23 and Figure 5.25 were used in completion of the Culvert Design Form. Figure 5.43 represents a completed Culvert Design Form for this problem. Figure 5.44 provides the performance curve and roadway overtopping computations.

FIGURE 5.42
EXAMPLE 4 ROADWAY OVERTOPPING AND PERFORMANCE CURVE DEVELOPMENT

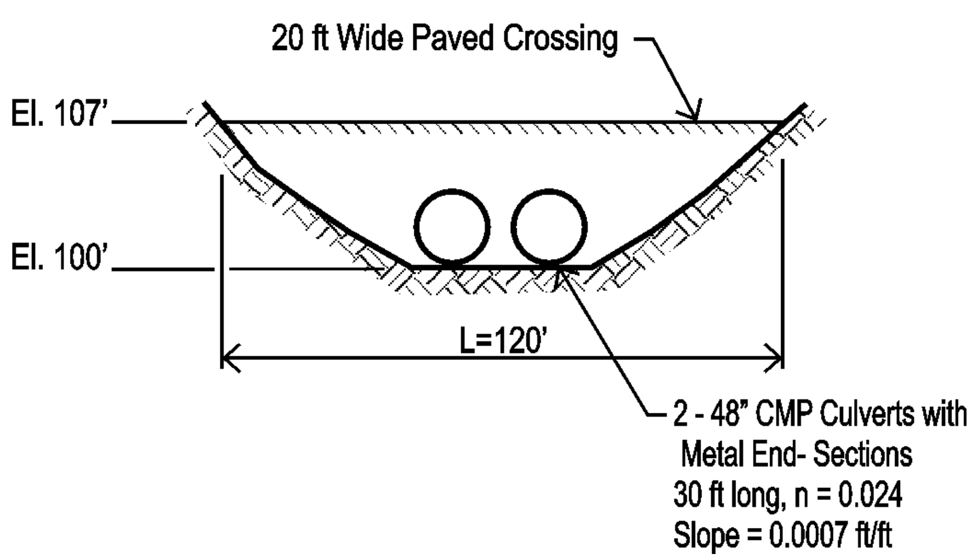


FIGURE 5.43
EXAMPLE 4 CULVERT DESIGN FORM

PROJECT: Example
Roadway Overtopping – Performance Curve

STATION: ______
SHEET 1 OF ______

CULVERT DESIGN FORM
DESIGNER / DATE: EJF / -3-2
REVIEWER / DATE: ______ / ______

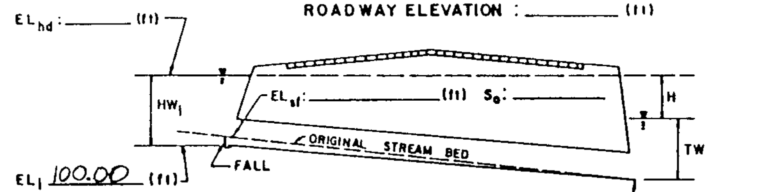
HYDROLOGICAL DATA

SEE ADD'L SHTS.
- ☐ METHOD: ______
- ☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: .0008
- ☐ CHANNEL SHAPE: Trapezoid
- ☐ ROUTING: ______ ☐ OTHER: ______

DESIGN FLOWS/TAILWATER

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

ROADWAY ELEVATION: ______ (ft)



$S = S_o - FALL / L_a$
$S =$ .0007
$L_a =$ 30'

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	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
CMP – Circular – 48" – end sect	50	25	.52	2.08	—	102.08	1.8	1.5	2.75	2.75	0.5	0.22	103.0			
	100	50	.78	3.12	—	103.12	2.6	2.1	3.05	3.05		0.60	103.6			
	150	75	1.03	4.12	—	104.12	3.1	2.6	3.30	3.30		1.35	104.6			
	200	100	1.30	5.20	—	105.20	3.5	3.0	3.50	3.50		2.40	105.9			
	250	125	1.63	6.52	—	106.52	3.8	3.4	3.70	3.80		3.75	107.5			

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

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

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

- a. APPROXIMATE
- f. CULVERT FACE
- hd. DESIGN HEADWATER
- hi. HEADWATER IN INLET CONTROL
- ho. HEADWATER IN OUTLET CONTROL
- i. INLET CONTROL SECTION
- o. OUTLET
- sf. STREAMBED AT CULVERT FACE
- tw. TAILWATER

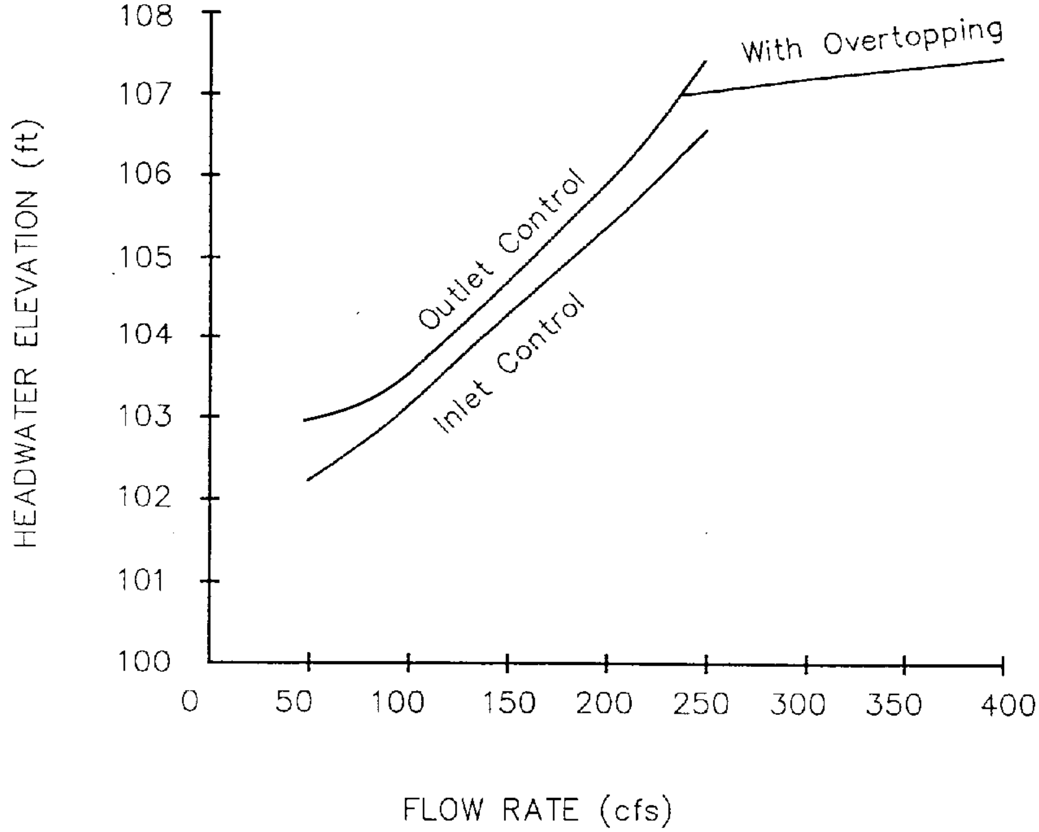
COMMENTS / DISCUSSION:

Used scale (1) of Figure 5.21 for inlet control comps.
used Figure 5.25 for outlet control head.

CULVERT BARREL SELECTED:

SIZE: ______
SHAPE: ______
MATERIAL: ______
ENTRANCE: ______

FIGURE 5.44
EXAMPLE 4 PERFORMANCE CURVE AND ROADWAY OVERTOPPING COMPUTATIONS



$$Q_O = K_t C_r L_s (HW_r)^{1.5}$$

HW_r	C_r	K_t	L_s	Q_o	Q_{pipe}		Q_{total}
0.25	2.98	1	120	44.7cfs	+244	=	289
0.50	3.02	1	120	128.1cfs	+250	=	378

5.4 ENTRANCES AND OUTLETS FOR CULVERTS

This section provides guidelines for design of culvert type inlets and outlets to closed conduit systems. Runoff entering and exiting closed conduits may require transitions into and out of the conduit to minimize entrance losses and protect adjacent property and drainage facilities from possible erosion. Pavement drainage inlets that allow runoff to drop into catch basins are discussed in Chapter 3, Section 3.3 and are not addressed here.

5.4.1 Interaction with Other Systems

Closed conduit inlets and outlets provide transitions from a ponded or channelized condition upstream into the closed conduit and then back to a natural or channelized condition downstream. Additional channel bank protection may be required in the vicinity of the inlet or outlet to complete the transition to the design velocity and flow depth of the receiving channel. The design of inlets and outlets should take into account all conditions in the upstream and downstream direction to the location where the inlet, outlet, and closed conduit have no effect on pre-design flow conditions.

When an open channel or stormwater storage basin drains into a storm drain system, culvert type inlets are frequently used. The storm drain hydraulic grade line must be considered when estimating the inlet capacity for culvert type inlets. The storm drain hydraulic grade line at the inlet, with the appropriate entrance loss added, should be substituted for the outlet control headwater elevation normally used for outlet control computations. To determine the controlling headwater, the computed outlet control headwater elevation should be compared with the inlet control headwater elevation obtained from the standard inlet control nomograph.

5.4.2 Special Criteria for Closed Conduits

Bank Protection

Roadway embankments with culverts passing through them should be protected from potential damage caused by roadway overtopping during a runoff event in excess of the culvert design capacity. When a planned flow over the road has damage potential, such as when the 100-year discharge causes flow over the roadway, the embankment for both upstream and downstream sides may need to be protected by use of paving, grouted riprap, or other means of permanent stabilization.

Entrance Structures and Transitions

Criteria for culvert entrances are contained in Section 5.3.2. The same criteria apply to culvert type entrances for storm drains. Design considerations include aligning the culvert with the natural channel profile, protection against inlet failure due to buoyant forces, and safety considerations for the public.

Culvert performance can be improved by providing a smooth and gradual transition at the entrance. Improved inlet designs have been developed for culverts operating in inlet control and are presented in Section 5.3.2.

Supercritical flow transitions at inlets require special design consideration. For design of supercritical flow contractions, refer to *Hydraulic Design of Energy Dissipators for Culverts and Channels* (USDOT, FHWA, HEC-14, 2006).

Outlet Structures

Standard measures for scour protection at conduit outlets include cutoff walls, wingwalls with aprons, and grouted or ungrouted riprap. These measures should be used as appropriate such that the velocity entering the receiving channel is within the allowable range of velocities for the channel outlet condition. Outlet conditions are classified as follows:

1. *Natural channel outlets* where the existing natural channel is modified only to transition to and from the culvert.

2. *Artificial channel outlets* where the culvert is part of an overall drainage plan and discharges into an improved, artificial channel.

3. *Side channel outlets* where a conduit drains into a larger receiving channel from the side at some angle of confluence.

It is not always desirable to totally restrict the movement of natural channels at the culvert outlet. Limited downstream scour and channel movement may be allowed in some cases. However, for artificial channel and side channel outlets, scour and bed movement should not be permitted. The following criteria shall be used in determining the type of outlet protection required based on the outlet condition.

Natural Channel Outlets

Natural channel outlet protection is based on the ratio of the culvert outlet velocity to the average natural stream velocity.

1. Culverts with outlet velocities less than or equal to 1.3 times the average natural stream velocity for the design discharge should have a cutoff wall as a minimum for protection. Design criteria for cutoff walls are presented below.

2. Where the outlet velocity is greater than 1.3 times the natural stream velocity, but less than 2.5 times, a riprap apron should be provided. Design procedures for riprap aprons are in Section 8.4.2.

3. When outlet velocities exceed 2.5 times the natural stream velocity, an energy dissipator should be provided. Several energy dissipators are described in Chapter 8, Hydraulic Structures.

Artificial Channel and Side Channel Outlets

Artificial channel and side channel outlet protection is based on the ratio of the culvert outlet velocity to the allowable velocity for the channel lining material. High velocity flow from the outlet must be transitioned to reduce the velocity to the allowable. Allowable velocities for several channel lining materials are shown in Chapter 6, Table 6.2 and Table 6.3.

1. Conduits with outlet velocity less than or equal to the allowable require no outlet protection.

2. Conduits with outlet velocity greater than one and less than 2.5 times the allowable velocity should be provided with a riprap, concrete, or other suitable apron to transition the flow to the allowable channel velocity.

3. When outlet velocities exceed 2.5 times the allowable channel velocity, an energy dissipator should be provided. Several energy dissipators are described in Chapter 8, Hydraulic Structures.

Cutoff Walls

A cutoff wall placed at the culvert outlet in a natural wash provides adequate protection of the downstream end of the culvert when the outlet velocity does not exceed 1.3 times the average natural stream velocity for the design discharge. Cut-off walls are appropriate where the development of a scour hole will not undermine nearby structures or result in other harmful effects.

Depth of scour for cohesionless materials (0.2mm<=D_{50}<=2.0mm) downstream of culvert structures may be estimated using Equation (5.8) from *Hydraulic Design of Energy Dissipators for Culverts and Channels* (USDOT, FHWA, HEC-14, 2006).

$$d_s = R_c C_h C_s \left(\frac{2.27}{\sigma^{1/3}}\right)\left(\frac{Q}{\sqrt{g}(R_c^{\ 2.5})}\right)^{0.39}\left(\frac{t}{316}\right)^{0.06} \tag{5.8}$$

where:

d_s = depth of scour hole, ft

R_c = hydraulic radius at the end of the culvert (assuming full flow)

Q = discharge, cfs

g = gravitation constant, 32.2 ft/sec^2

t = time of scour, set at 30 minutes if unknown

σ = $(D_{84}/D_{16})^{0.5}$, material standard deviation

C_h = drop height adjustment coefficient, see Table 5.2

C_s = slope correction coefficient, see Table 5.3

TABLE 5.2
COEFFICIENT C_h FOR OUTLETS ABOVE THE BED

H_d	C_h
0	1.00
1	1.22
2	1.26
4	1.34

where: H_d is the height above the bed in pipe diameters.

TABLE 5.3
COEFFICIENT C_s FOR CULVERT SLOPE

Slope, %	C_s
0	1.00
2	1.03
5	1.08
>7	1.12

The bed-material grain size distribution is determined by performing a sieve analysis (ASTMDA22-63). The values of D_{84} and D_{16} are extracted from the grain size distribution. If σ <1.5, the material is considered to be uniform. If σ >1.5, the material is classified as graded. Typical values for σ are 2.10 for gravel and 1.87 for sand.

If the soil is cohesive in nature, Equation (5.9) should be used to determine the depth of scour. Equation (5.9) is from *Hydraulic Design of Energy Dissipators for Culverts and Channels* (USDOT, FHWA, HEC-14, 2006). Use of Equation (5.9) should be limited to sandy clay soils with a plasticity index in the range of 5 to 16.

$$d_s = y_e C_h C_s \alpha_e \left(\frac{\rho V^2}{\tau_c}\right)^{0.18} \left(\frac{t}{316}\right)^{0.10} \tag{5.9}$$

where:

d_s = depth of scour hole, ft

y_e =equivalent depth $(A/2)^{1/2}$, ft (or culvert diameter for circular pipes)

A = cross sectional area of flow, ft^2

V = mean outlet velocity, ft/s

g = gravitation constant, 32.2 ft/sec^2

t = time of scour, set at 30 minutes if unknown

τ_c = critical tractive shear stress, lb/ft^2

ρ = fluid density of water, 1.94 $slugs/ft^3$

α_e = 37 (0.86 for circular pipe culverts)

C_h = drop height adjustment coefficient, see Table 5.2

C_s = slope correction coefficient, see Table 5.3

$$\tau_c = 0.001(S_v + \alpha_u)\tan(30 + 1.73PI) \quad (5.10)$$

where:

τ_c = critical tractive shear stress, lb/ft^2

S_v = the saturated shear strength, lb/ft^2

α_u = unit conversion constant, 180 lb/ft^2

PI = Plasticity Index from Atterberg limits

The following guidelines, applicable to cutoff walls, are based on the computed depth of scour hole analysis identified above.

1. The depth of the cutoff wall should be equal to or greater than the maximum depth of scour hole.

2. The depth of the cutoff wall should not normally exceed 6 feet. Where a deeper wall is necessary to meet the above guidelines, either another form of protection should be employed or an analysis will be required to substantiate the walls structural stability. Typically, some combination of cutoff wall and erosion protection such as rip-rap is used at culvert outlets.

Topics on scour are presented in Chapter 11, Sedimentation.

Safety

Inlets and outlets to closed conduits may present dangers to the public when access is not controlled. Refer to Chapter 1, Section 1.4 for the safety requirements related to conduit inlets and outlets.

5.4.3 Protection at Culvert Outlets

Riprap aprons placed downstream of culverts provide protection against scour immediately around the culvert as well as providing for the uniform spreading of the flow and decreasing the flow velocity, thus mitigating downstream damages. Use the procedures in Chapter 8, Section 8.4 for designing culvert outlet protection.

5.5 INVERTED SIPHONS

5.5.1 General

Because of the resulting physical conditions, inverted siphons are rarely used in urban drainage and should be avoided where possible. Due to the flat topography and a large number of canals in Maricopa County, however, the designer may have to consider using an inverted siphon.

Inverted siphons are used to convey water by gravity under canals, roads, railroads, other structures, and depressions. An inverted siphon is a closed conduit designed to run full and under pressure. When flowing at design capacity, the structure should operate without excess head.

For canal structures, inverted siphons are economical, easily designed and built, and have proven to be a reliable means of water conveyance. However, because of sediment and debris present in stormwater, maintenance can be a significant negative factor. In addition, canals run more or less continually and can be drained between periods of use, but inverted siphons for stormwater do not operate on a regular cycle. If water is left to stand, significant health hazards could result. Inverted siphons shall be considered only when absolutely necessary, and permitted by the jurisdictional agency.

5.5.2 Design

All pipes should be designed for watertight joints. Velocity in the conduit should be a minimum of 5.0 ft/sec to prevent sedimentation. The cover over the conduit should exceed the minimum cover necessary to meet its loading classification. Inlet and outlet structures are required, and the facility shall meet the requirements for safety described in Chapter 1, Section 1.4. Pipe collars and blow-off structures may be required as determined by the jurisdictional agency. Air vents, after the entrance, should be used unless the agency agrees with eliminating the vents.

At a minimum, the designer should compute losses for the entrance and outlet (including trashracks), pipe friction, and losses at bends and transitions.

5.5.3 Design Procedure

A design procedure with examples is contained in *Design of Small Canal Structures* (USBR, 1974). Taking into consideration conditions that are more specific to urban drainage described before, this publication can be used for most applications in Maricopa County.

5.6 BRIDGES

This section presents a brief overview of the hydraulic analyses for bridge crossings over open channels. A general discussion of scour is also presented. Comprehensive guidelines and criteria for hydraulic analyses of bridge crossings are beyond the scope of this manual. The reader should refer to appropriate texts and technical handbooks for further information on this subject.

Roadways must often cross open channels in urban areas; therefore, sizing the bridge openings is of paramount importance. In general, bridges should be designed to have as little effect as possible upon the flow passing beneath them. If possible, bridges over natural or man-made channels should be designed so that there is no disturbance to the flow whatsoever. Whenever piers are used, they need to be oriented parallel to flow. Impacts upon channels and floodplains created by bridges usually take the form of increased flow velocities through and downstream of the bridges, increased scour and upstream ponding due to backwater effects. These impacts can cause flood damage to the channel, to adjacent property and to the bridge structure itself.

A new or replacement bridge should not be permitted to create a rise in the existing water surface elevation, to cause an increase in lateral extent of the floodplain, or to otherwise worsen existing conditions for discharges up to and including the 100-year discharge, unless appropriate measures are taken to mitigate the effects of such increases.

5.6.1 Hydraulic Analysis

The hydraulic analyses of pre- and post-bridge conditions can be performed using a computerized step-backwater model. The HEC-RAS program developed by the U.S. Army Corps of Engineers (USACE, 2001) is the most common backwater computation software available and is used nationwide. HEC-RAS is the preferred computer software for one-dimensional hydraulic analyses for studies of this type in Maricopa County. The Corps older HEC-2 program may also be used for analyzing bridges, but is not preferred.

Bridge analysis requires meticulous input preparation for proper analysis, and care should be taken to review input data and to examine results thoroughly for reasonableness. Analyses of this type should only be undertaken by an engineer with a solid understanding of hydraulic fundamentals.

If there is a good possibility of debris collecting on the piers, it may be advisable to use a value greater than the physical pier width to account for debris blockage. Some agencies require the pier width to be modeled as twice its width while others require 1 foot added to each side of the pier. Thus, modeling requirements of debris blockage should be reviewed with the jurisdictional agency. For guidance, refer to the Uniform Drainage Policy and Standards Manual for the jurisdiction in question.

5.6.2 Hydraulic Design Considerations

Additional factors to be considered in the design of a bridge crossing include flow regime (i.e., subcritical or supercritical flow), anticipated scour effects, and freeboard.

Freeboard

Freeboard at a bridge is the vertical distance between the design water surface elevation and the low-chord of the bridge. The bridge low-chord is the lowest portion of the bridge deck superstructure. The purpose of freeboard is to provide room for the passage of floating debris, to provide extra area for conveyance in the event that debris build-up on the piers reduces hydraulic capacity of the bridge, and to provide a factor of safety against the occurrence of waves or floods larger than the design flood. Freeboard should be provided as required by jurisdictional standards.

A minimum freeboard of 2 feet for the 100-year event is recommended. The structural design of the bridge should take into account the possibility of debris and/or flows impacting the bridge.

In certain cases, site conditions or other circumstances may limit the amount of freeboard at a particular bridge crossing. An example would be the replacement of a “perched” bridge across a natural watercourse where major flows overtop the roadway approaches. In general, variances to the minimum freeboard requirement will be evaluated on a case by case basis by the jurisdictional agency.

Supercritical Flow

For the special condition of supercritical flow within a lined channel, the bridge structure should not affect the flow at all. That is, there should be no projections, piers, etc. in the channel area. The bridge opening should be clear and permit the flow to pass unimpeded and unchanged in cross section.

Scour

The issue of scour analysis at a bridge is beyond the scope of this chapter. The following discussion touches upon the subject matter to provide the interested designer an indication of the issues. Local pier and abutment scour, contraction scour, and long-term scour must be investigated when designing a bridge. Refer to Chapter 11, Sedimentation for guidance and insight into sedimentation and scour.

General scour from a contraction usually occurs when the normal flow area of a stream is decreased by a bridge. The contraction of the flow by the bridge can be caused by a decrease in flow area of the stream channel by the abutments projecting into the channel and/or the piers taking up a large portion of the flow area. Also, the contraction can be caused by approaches to the bridge that cut off the overland flow that normally goes across the floodplain during high flow. This latter case also can cause clear-water scour at the bridge section because overland flow normally does not transport any significant bed material sediments. This clear-water picks up additional sediment from the bed when it returns to the bridge crossing. In addition, if floodwater returns to the stream channel at an abutment it increases the local scour there. A guide bank at an abutment decreases the risk from scour of that abutment from returning overbank flow. Also, relief bridges in the approaches reduce general scour by decreasing the amount of flow returning to the natural channel, which then decreases the scour problem. See Chapter 11, Sedimentation for scour analysis protocol.

5.7 REFERENCES

5.7.1 Cited in Text

Brater, E. F. and King, H. W., 1976, *Handbook of Hydraulics for the Solution of Hydraulic Engineering Problems, Sixth Edition*.

Debo, Thomas N., and Reese, Andrew J., 1995, *Municipal Storm Water Management*, p. 320.

Chow, V. T., 1959, *Open Channel Hydraulics*.

Hulsing, Harry, 1968, *Techniques of Water-Resources Investigations of the United States Geological Survey, Chapter A5, Measurement of Peak Discharge at Dams by Indirect Method, Book 3 Applications of Hydraulics*.

United States Army Corps of Engineers Hydrologic Engineering Center (USACE), 1990, *HEC-2 Water Surface Profiles, User's Manual*. [HEC WEB Site]

——, 2001a, *HEC-RAS River Analysis System, Hydraulic Reference Manual*.

——, 2001b, HEC-RAS River Analysis System, User's Manual.

United States Department of the Interior, Bureau of Reclamation (USBR), 1974, *Design of Small Canal Structures*, Denver, Colorado.

United States Department of Transportation (USDOT), Federal Highway Administration, 1978, *Hydraulics of Bridge Waterways, Hydraulic Design Series No. 1*. [USDOT Hydraulics WEB Site]

——, 1985, *Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5*.

——, 1999, *HY8 Culvert Analysis Version 6.1 Computer Program*.

——, 2006, *Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14*.

5.7.2 References Relevant to Chapter

American Society of Civil Engineers and the Water Pollution Control Federation (ASCE), 1969, *Design and Construction of Sanitary and Storm Sewers*.

American Iron and Steel Institute, 1990, *Modern Sewer Design*.

Richardson, E.V., Simons, D.B., Lagasse, P.F., 2001, *River Engineering for Highway Encroachments, Highways in the River Environment, Hydraulic Design Series No. 6*, U.S. Department of Transportation, Federal Highway Administration. [USDOT Hydraulics WEB Site]

United States Department of Transportation (USDOT), Federal Highway Administration, 1986, *Bridge Waterways Analysis Model Research Report*.

——, 1997, *Bridge Scour and Stream Instability Countermeasures, HEC-23*.

U.S. Environmental Protection Agency, 1976, *Erosion and Sedimentation Control, Surface Mining in the Eastern U.S., EPA-625/3-76-006*.

Page intentionally left blank

6 OPEN CHANNELS

TABLE OF CONTENTS

6.1 SYMBOLS

The following symbols will be used in equations throughout Chapter 6.

α	=	Velocity head coefficient
β	=	Momentum coefficient, or channel bend angle, degrees
γ_s	=	Specific weight of stone, lb/ft^3
γ_w	=	Specific weight of water, lb/ft^3
φ	=	Bank angle, degrees
θ	=	The channel slope angle, degrees
Δ_y	=	Change in water surface elevation, ft
$\Sigma L_o/L_t$	=	Ratio of the summation of the distances between rows of buildings, L_o, to the total length of the reach along a profile parallel to flow, L_T, ft/ft
Φ	=	Angle of repose, degrees, bank angle
A	=	Cross sectional area of flow, sq ft
A_T	=	Total area, sq ft
A_{lf}	=	Area of low flow channel, sq ft
A_{mc}	=	Area of main channel, sq ft
b	=	Channel bottom width, ft
C	=	Overall correction factor when using a different stability factor or specific gravity
C_{sf}	=	Correction factor for stability factor
C_{sg}	=	Correction factor for specific gravity
C_{VI}	=	Volume increase coefficient, percent
d	=	Depth of flow, or hydraulic depth, ft
d_{ave}	=	Average depth of flow in the main channel, ft
D	=	Diameter, ft
DAR	=	Durability absorption ratio
D_i	=	The average diameter of a rock particle for which "i" percent of gradation is finer by weight, mm
E	=	Specific energy, ft
EL	=	Elevation, ft
E_T	=	Total energy, ft
F	=	Specific force, ft^3
F'	=	Force from friction, bends and other factors, ft^3
FB	=	Freeboard, ft
F_r	=	Froude number
g	=	Acceleration due to gravity, 32.2 ft/sec^2
G	=	Gradation coefficient
h_f	=	Head loss, ft

h'_f	=	Head loss due to external forces, ft
H	=	Toe thickness, ft
H_V	=	Vertical launch distance, ft
K_1	=	Bank angle correction factor
KE	=	Kinetic energy, ft
l	=	Length along channel, ft
L	=	Characteristic length, ft, or required toe length, ft
L_o	=	Sum of individual length between buildings measured parallel to flow, ft
L_T	=	Total length of the floodplain, including buildings, ft
m	=	Mass, lb
M	=	Momentum, ft-lb
n	=	Manning's roughness coefficient
n_o	=	Roughness coefficient for the area between the buildings in the floodplain
n_u	=	Adjusted urban roughness coefficient
P	=	Wetted perimeter, ft
P_T	=	Perimeter of composite section, ft
P_{lf}	=	Perimeter of low flow channel, ft
P_{mc}	=	Perimeter of main channel, ft
P_h	=	Hydrostatic pressure, ft
q	=	unit discharge, cfs/ft
Q	=	Discharge, cfs
R	=	Hydraulic radius, ft
Re	=	Reynolds Number
r_c	=	Radius of channel center-line curvature, ft
SF	=	Stability factor
S_o	=	Channel bottom slope, ft/ft
S_f	=	Friction slope, ft/ft
S_s	=	Specific gravity of the rock riprap
T	=	Channel width along the top of the water surface, ft, or riprap layer thickness, ft
V *or* $\bar{v}$	=	Average velocity of a section, ft/sec
v	=	Vectoral velocity, ft/sec
ν	=	Kinematic viscosity of water, ft^2/sec
V_a	=	Average velocity in the main channel, ft/sec
W	=	Weight of water, lb
w	=	Unit weight of water, lb/ft^3
W_i	=	Weight of stone where i is the percent of stones weighing less than the given weight, lb
W_o	=	Sum of clear width between buildings, measured perpendicular to flow, ft

W_T	=	Total width of the floodplain including buildings, ft
W'	=	Volume of water (W/w), ft^3
y	=	Pressure head or depth of flow, ft
y_c	=	Critical depth of flow, ft
y_n	=	Normal depth of flow, ft
z	=	Elevation of channel invert (elevation head), ft, or distance from water surface to the centroid of the section, ft
Z_T	=	Total scour depth, ft

6.2 INTRODUCTION

6.2.1 Open Channel Defined

An *open channel* is a conveyance system in which water flows with a free surface at the water atmosphere interface. The channel may be either a natural watercourse or an artificial, "engineered" conveyance. *Natural streams* typically consist of a main flow channel and adjacent floodplains. *Artificial channels* are used for a wide variety of applications varying in scale from modest roadside ditches to large conveyance facilities that can be up to several hundred feet wide. Design guides are provided for the analysis of both natural and engineered channels.

6.2.2 Scope of Chapter

This chapter is intended to provide a concise review of the fundamentals of open channel hydraulics and to provide design guidelines for use by engineers in the design of public infrastructure projects. More detailed explanations and further information are available from the technical resources listed at the end of this chapter. Readers are strongly encouraged to review the reference list and consider adding some of those publications to their design library.

The Open Channel chapter contains four general sections:

- Section 6.3 - Open channel hydraulics fundamentals which are applicable to both engineered and natural channels, augmented with illustrative computational examples;
- Section 6.4 – General considerations for open channel drainage planning, such as route and layout factors; hydraulic analysis considerations and limitations which are generally applicable to both engineered and natural channels, but some, such as grade control, will be specific to engineered channels;
- Section 6.5 – Design factors for open channels, such as determination of freeboard and toe down requirements;
- Section 6.6 - Design guidelines are recommended for various types of open channels and for several alternate channel materials, including concrete lined channels, shotcrete, soil cement, cement stabilized alluvium, riprap, and gabions.

6.2.3 Application

The theories and concepts presented in this chapter are applicable to both natural and engineered channels.

6.2.4 Limitations

This chapter assumes that all channel boundaries are rigid, i.e., the channel cross section remains unaffected by erosion and the channel gradient remains constant for all flows. In this respect, this chapter is limited to channels where erosion, transportation, and deposition of sediment are not critical design considerations. For channels requiring consideration of non-rigid boundaries and/or sedimentation, see Chapter 11.

Recommendations in this chapter address only channels designed to sustain subcritical or mildly supercritical flow regimes. Supercritical flows with Froude numbers greater than 1.13 require design procedures outside the scope of this chapter. If a designer determines that flows in the supercritical regime are unavoidable because of unique physical conditions, they should consult the technical staff of the jurisdiction involved for appropriate guidance. Section 6.3.2 contains discussion of the calculation of the Froude number and the determination of flow regime.

The design guidelines in Section 6.5 of this chapter for channel side slopes, lining materials, and allowable velocities have been put forth to protect the health and welfare of the public while minimizing societal costs. Designers are strongly encouraged to stay within these guidelines, unless alternative analytic procedures, guidelines, etc. can be substantiated.

6.3 BASIC OPEN CHANNEL HYDRAULICS

6.3.1 Flow Classification

Open channel flow is classified into many types and described in various ways based upon how the flow varies spatially and temporally. A steady flow is one in which all conditions at any point in a stream remain constant with respect to time (Daugherty and Franzini, 1977). Steady flow is often more simply defined as a constant flow rate producing a constant depth of flow at a given point in a channel for the time period under consideration. Conversely, the flow is unsteady if the flow conditions such as depth change with time. Thus, time is the criterion in the determination of steady and unsteady flow. In most open channel design problems, only steady flow conditions are considered.

Space is the criterion in the determination of uniform and varied flow. A truly uniform flow is one in which the velocity is the same in both magnitude and direction at a given instant at every point in the fluid (Daugherty and Franzini, 1977). Open channel flow is often considered uniform if the flow depth is the same at every point along the channel. Flow is nonuniform where it is spatially varied or discontinuous; that is, discharge varies or other flow conditions change along the course of flow. Uniform flow may be steady or unsteady, depending on whether or not the flow conditions change with time. Uniform flow is also called normal flow and the flow depth under uniform flow conditions is referred to as normal depth. Refer to Section 6.3.5 for more detailed information in regard to the computation of normal depth.

Flow is varied if the flow conditions, such as depth, change along the length of the channel. If the depth varies at points along the channel, it will do so either rapidly or gradually, depending upon the channel geometry and flow constraints. The flow is rapidly varied if the depth changes abruptly over a relatively short distance. Examples of rapidly varied flow include local phenomena, such as hydraulic jumps and hydraulic drops. Under steady flow conditions, if the depth of flow along the length of the channel gradually increases or decreases it is gradually varied. This is the usual condition in open channel flow. Gradually varied flow occurs under either subcritical or supercritical flow regimes. Water surface profile computations are required to estimate the depth of flow for varied flow conditions at any given location as described in Section 6.3.6.

6.3.2 Flow Regimes

Froude Number

The state of open channel flow is governed by the effects of viscosity and gravity relative to the inertial forces of the flow. The effect of gravity on the state of flow is represented by a ratio of inertial forces to gravity forces. This ratio is given by the Froude number, defined as:

$$F_r = \frac{V}{\sqrt{gd}} \tag{6.1}$$

where V is the mean velocity (ft/sec), g is the acceleration of gravity (ft/sec^2), and d is the hydraulic depth (ft) which is the cross sectional area of the water, A (sq ft), divided by the width of the free surface, T (ft).

When F_r is equal to 1, the flow is in the critical state. This flow condition is unstable and flow depths at or near critical depth should be avoided. If F_r is less than 1, the flow is subcritical and gravity forces dominate. When F_r is greater than 1, the flow is supercritical and inertial forces predominate.

Specific Energy

Specific energy in a channel section is defined as the energy per pound of water at any section of a channel measured with respect to the channel bottom and may be expressed as:

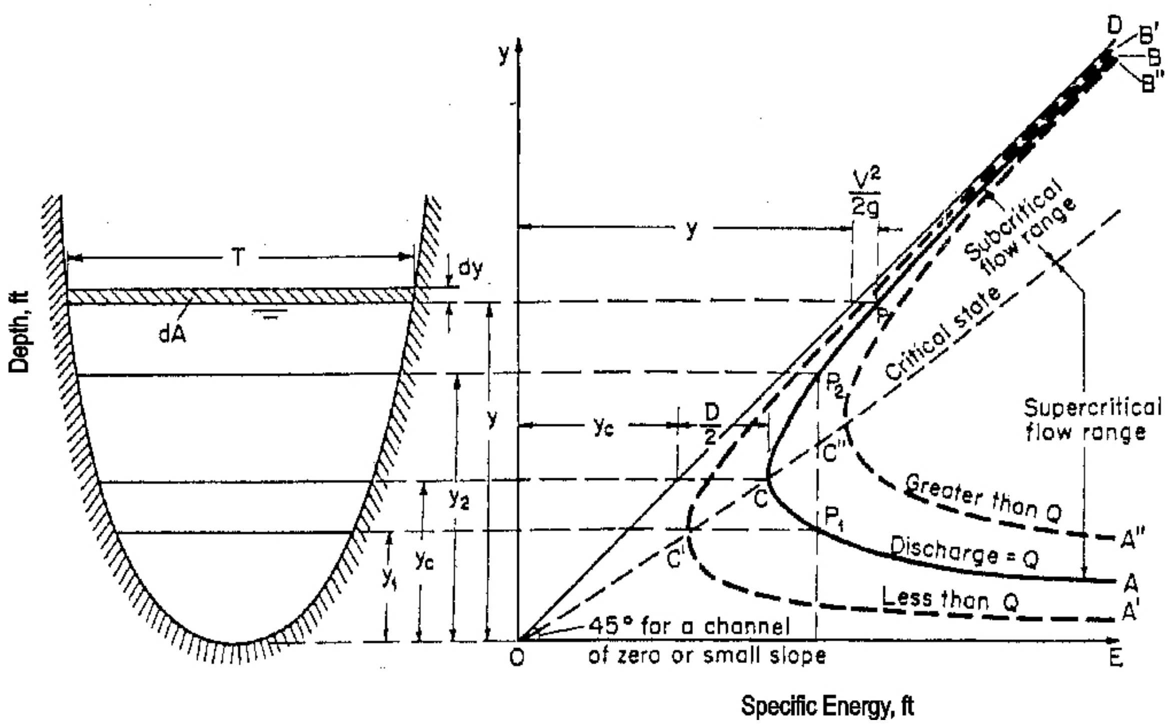
$$E = y + \frac{V^2}{2g} = y + \frac{Q^2}{2gA^2} \tag{6.2}$$

When the depth of flow is plotted against the specific energy for a given channel section and discharge, a specific energy curve is obtained (Figure 6.1). The specific energy curve has two limbs, AC and BC. The limb AC approaches the horizontal axis asymptotically toward the right. The limb BC approaches the line OD as it extends upwards and to the right. The line OD has an angle of inclination equal to 45°. At any point P on this curve, the ordinate represents the depth of flow, and the abscissa represents the specific energy that is equal to the sum of the pressure head, y, and the velocity head, $V^2/2g$. The curve shows that, for a given specific energy, there are two possible depths, the low stage, y_1, and the high stage, y_2. The low stage is called the alternate depth of the high stage and vice versa.

At point C, the specific energy is a minimum and the stage is at critical depth. When the depth of flow is greater than the critical depth, the velocity of flow is less than the critical velocity and the flow is subcritical. When the depth of flow is less than the critical depth, the flow is supercritical. Inspection of the energy curve in the vicinity of critical depth reveals that a small change in the energy will result in a relatively large change in the depth of flow. For this reason, it is strongly recommended that flow depths producing Froude numbers between 0.87 and 1.13 be avoided.

The Froude Number limit for all types of channel linings is $F_r \leq 0.86$. For concrete and shotcrete lined channels, the additional range of $1.13 \leq F_r \leq 2.0$ is allowed. F_r should not fall between 0.86 and 1.13 in order to maintain stable flow conditions. Due to safety concerns resulting from excessively high velocities and intractable hydraulic forces, the recommended upper limit of F_r is 2.0 except at certain structures such as drop structures.

FIGURE 6.1
SPECIFIC ENERGY CURVE
(MODIFIED FROM: Chow, 1959)



Critical Flow

Critical depth in an open channel has the following characteristics:

- For a given flow rate, the specific energy is at a minimum.
- The discharge is at a maximum for a given specific energy.
- The velocity head is one half of the flow depth.
- The Froude Number is 1.0.

By substituting $V^2 = Q^2/A^2$ into Equation (6.1) and rearranging, we can obtain a general expression for critical depth that is applicable to any channel cross section:

$$\frac{Q^2}{g} = \frac{A^3}{T} \tag{6.3}$$

EXAMPLE 6.1: What is the critical depth of flow for 400 cfs flowing in a rectangular channel 10.0 feet wide?

$$\frac{400^2}{32.2} = \frac{(10y_c)^3}{10}$$

$$y_c = 3.68\,ft$$

Subcritical Flow

Flows producing Froude numbers less than 1.0 are subcritical and have the following general characteristics relative to critical depth:

- Slower velocities.
- Greater depths.
- Lower hydraulic losses.
- Less erosive power.
- Less sediment carrying capacity.
- Behavior easily described by relatively simple mathematical equations.
- Surface waves propagate upstream.

Supercritical Flow

Flows with Froude numbers greater than 1.0 are supercritical and have the following general characteristics relative to critical depth:

- Higher velocities.
- Shallower depths.
- Higher hydraulic losses.
- More erosive power.
- More sediment carrying capacity.
- With few exceptions, behavior can't be easily predicted mathematically.
- Surface waves propagate downstream only.

6.3.3 Equations of Flow

Continuity

For any flow, the discharge, Q, at a channel section is expressed by:

$$Q = AV \tag{6.4}$$

Where V is the mean velocity (ft/sec) and A is the cross sectional area of the flow measured normal to the direction of flow (sq ft). Under steady flow conditions, the discharge is constant and:

$$Q = A_1V_1 = A_2V_2 \tag{6.5}$$

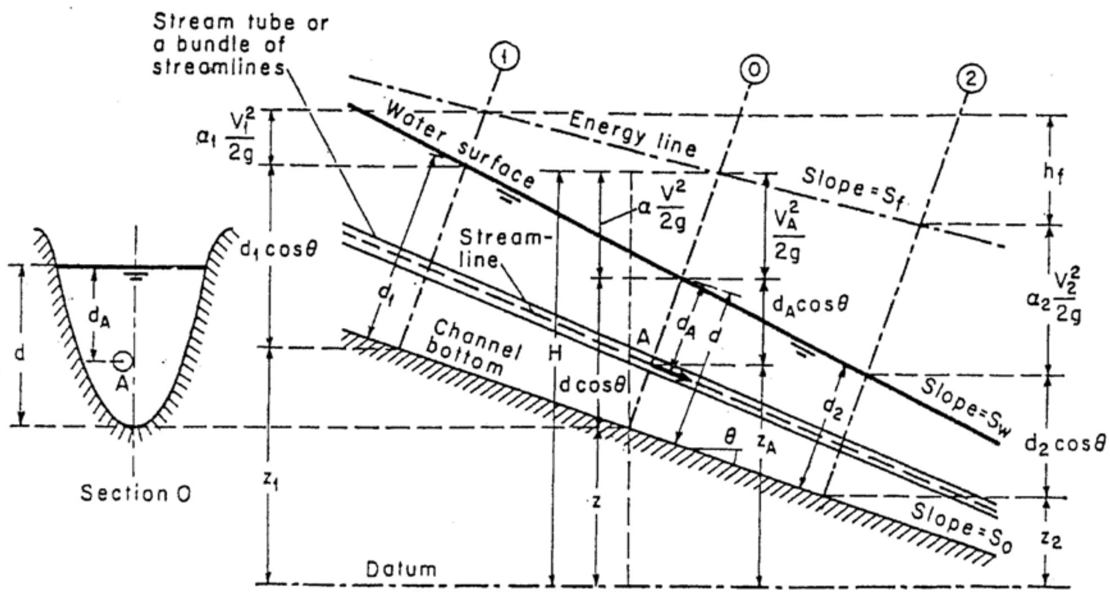
The subscripts denote different channel sections. Equation (6.5) is known as the Continuity Equation and is applicable to the flow conditions addressed in this chapter.

Obviously, Equation (6.5) is invalid for unsteady flow conditions in which discharge increases nor decreases along the course of flow. Examples of unsteady flow are flood waves, bores, roadside gutters, side-channel spillways, wash water troughs in filters and, effluent channels around sewage treatment tanks. Precise treatment of unsteady flow is mathematically complicated and beyond the scope of this chapter.

Energy

The First Law of Thermodynamics states that energy can neither be created nor destroyed; it can only be transformed. Thus, in the case of an open channel carrying a steady flow, the total energy at any two points must be equal. At a given cross section, the total energy at any point is the sum of kinetic and potential energy at that point as illustrated in Figure 6.2.

FIGURE 6.2
ENERGY IN GRADUALLY VARIED OPEN CHANNEL FLOW
(Chow, 1959)



The following relationship is readily deduced from Figure 6.2:

$$E_T = \alpha_1 \frac{V_1^2}{2g} + y_1 + z_1 = \alpha_2 \frac{V_2^2}{2g} + y_2 + z_2 + h_f \qquad (6.6)$$

where:

$$y_1 = d_1 \cos\theta$$

The velocity head coefficient, α, is a correction to account for the non uniformity of the velocity in the channel. Experimental data indicates this value varies between 1.03 and 1.36 for fairly straight, prismatic channels. The value is generally higher for small channels, and lower for larger streams of considerable depth. For channels of regular cross section and fairly straight alignment, the effect of non-uniform velocity distribution on the computed velocity head is small, especially when compared to other uncertainties involved in the computation. Therefore, α is often assumed to be 1.0. Additionally, experience indicates that using the average velocity often gives satisfactory accuracy for usual open channel flow conditions. However, in some cases it may be desirable to use the computed value of the energy coefficient α. Kinetic energy (KE) is estimated by Equation (6.7).

$$KE = \alpha\frac{\bar{v}^2}{2g} \qquad \alpha \geq 1.0 \tag{6.7}$$

where:

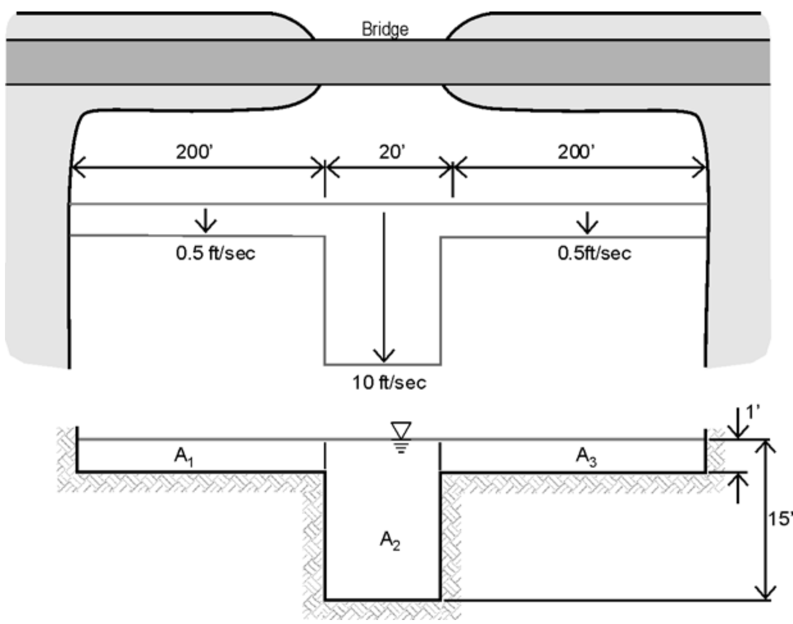
$$\alpha = \frac{\int v^3 dA}{\bar{v}^3 A} \tag{6.8}$$

Computational form:

$$\alpha = \frac{\sum v^3 \Delta A}{\bar{v}^3 A} \tag{6.9}$$

Example 6.2: The flow in a river downstream of a bridge constriction is as shown. Calculate the energy coefficient.

$$Q_T = (.5)(1)(200) + (10)(15)(20) + (.5)(1)(200) = 3,200\ cfs$$



$$A = A_1 + A_2 + A_3 = 200 + 300 + 200 = 700\ ft^2$$

$$\bar{v} = \frac{Q}{A} = \frac{3,200}{700} = 4.57\ ft/sec$$

$$\alpha = \frac{\sum v^3 \Delta A}{\bar{v}^3 A} = \frac{(.5)^3(200) + (10)^3(300) + (.5)^3(200)}{(4.57)^3(700)}$$

$$\alpha = 4.49$$

If the datum is the invert of the channel at Section 2, $z_2 = 0$ and $z_1 = S_o l$, where l is the channel length between Sections 1 and 2. The energy lost due to friction is represented as $h_f = S_f l$. Making these substitutions, Equation (6.6) reduces to the following:

$$E = \alpha\frac{V_1^2}{2g} + y_1 + S_o l = \alpha\frac{V_2^2}{2g} + y_2 + S_f l \tag{6.10}$$

Equation (6.10) is the basis for calculating water surface profiles, which will be discussed in more detail in Section 6.3.6.

Momentum

The momentum of a flow passing a channel section per unit time is expressed by $\beta wQV/g$.

where:

w	=	is the unit weight of water, and
β	=	is the momentum coefficient.

According to Newton's Second Law Of Motion, the change of momentum per unit time in a body of water in a flowing channel is equal to the resultant of all the external forces that are acting on the body. Assuming a channel of small slope, the momentum of a volume of water between section 1 and 2 can be expressed as follows (Chow, 1959):

$$z_1 + y_1 + \beta_1\frac{V_1^2}{2g} = z_2 + y_2 + \beta_2\frac{V_2^2}{2g} + h'_f \tag{6.11}$$

Where β_1 and β_2 are momentum correction coefficients at the two sections. In the energy equation, h_f measures the internal energy dissipated in the whole mass of water in the reach, whereas h'_f in the momentum equation measures the losses due to external forces exerted on the water by the boundaries of the channel. Assuming the small differences between a and β in uniform flow, the rate with which surface forces are doing work is equal to the rate of energy dissipation. In that case, a distinction does not exist between h_f and h'_f except in definition.

The momentum (M) is estimated by mv where m is mass and v is vectoral velocity. For "nonuniform" velocity distributions, this should be corrected.

$$M = \beta mv \qquad \beta \geq 1.0 \tag{6.12}$$

where: $$\beta = \frac{\int v^2 dA}{\bar{v}^2 A} \tag{6.13}$$

Computational form:

$$\beta = \frac{\Sigma v^2 \Delta A}{\bar{v}^2 A} \tag{6.14}$$

Example 6.3: For the conditions presented in Example 6.2, calculate the momentum coefficient, β

$$\beta = \frac{\sum v^2 \Delta A}{\bar{v}^2 A} = \frac{(.5)^2(200) + (10)^2(300) + (.5)^2(200)}{(4.57)^2(700)}$$

$$\beta = 2.06$$

The similarity between the energy and momentum principles may be confusing. A clear understanding of the basic differences is important, despite the fact that in many instances the two principles produce practically identical results. The inherent distinction between the two lies in the fact that energy is a scalar quantity whereas, momentum is a vector quantity. Also, the energy equation contains a term for internal losses (energy), whereas, the momentum equation contains a term for external resistance (force).

Chow (1959) presents the development of the specific energy and specific force curves for a given channel and discharge (Figure 6.3). For a short horizontal reach of prismatic channel, the external force of the friction and the weight effect of water can be ignored. Thus, the momentum equation can be written as:

$$Q\left(\frac{w}{g}\right)(V_2 - V_1) = P_{h_1} - P_{h_2} \tag{6.15}$$

where P_h is hydrostatic pressure,

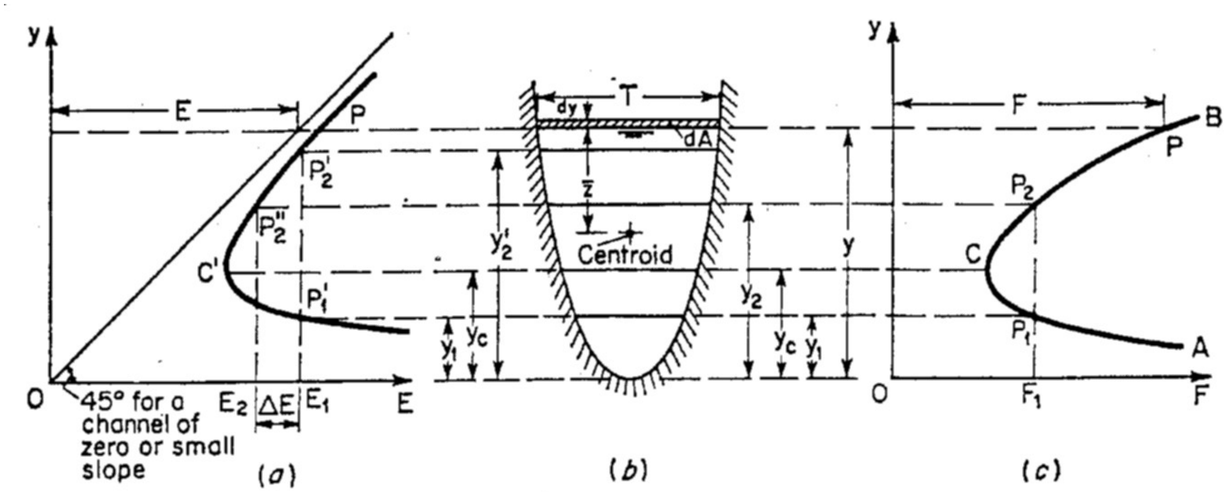
$$P_{h_1} = wz_1A_1 \tag{6.16}$$

$$P_{h_2} = wz_2A_2 \tag{6.17}$$

with z as the distance to the centroids of the respective water areas below the surface of flow. With constant steady flow and $V = Q/A$,

$$\frac{Q^2}{gA_1} + z_1 A_1 = \frac{Q^2}{gA_2} + z_2 A_2 \tag{6.18}$$

FIGURE 6.3
SPECIFIC-FORCE CURVES SUPPLEMENTED WITH SPECIFIC-ENERGY CURVES
(a) Specific-Energy Curve; (b) Channel Section; (c) Specific-Force Curve.
(Chow, 1959)



Equation (6.18) is the most basic form of the momentum equation. For a channel forming angle θ with the horizontal, a weight of water W between the points 1 and 2 of the equation, and inserting β to account for non-parallel flow, the equation becomes:

$$\frac{\beta_1 Q^2}{gA_1} + z_1 A_1 = \frac{\beta_2 Q^2}{gA_2} + z_2 A_2 + W' \sin\theta + F' \tag{6.19}$$

where W' = weight (W) in pounds divided by w (62.4 lb/ft^3), and where F' is the force in ft^3 resulting from friction, bends, and all other factors. The sum of external forces (F) is $F'w$.

Specific Force

The two sides of Equation (6.18) are analogous and may be expressed by the general function:

$$F = Q^2 / gA + zA \tag{6.20}$$

Both terms in the function are essentially force per unit weight of water, and their sum may be called the **specific force**. Since $F_1 = F_2$, the specific forces of Sections 1 and 2 are equal, provided that the external forces and the weight effect of water in the reach between the two sections can be ignored. On a plot of depth against specific force for a given channel section and discharge, two possible depths are evident for a given-value of the specific force. These depths constitute the initial and sequent depths of a hydraulic jump. At the point where the two depths become one, specific force is at a minimum and the depth is equal to critical depth. The two basic equations for hydraulic analysis are energy and momentum. The simplest forms of the equations are developed for restricted cases which establish boundary conditions so that complex differential equations are avoided. These equations, when correctly applied, can provide good solutions to many problems; however, the hydrologist or engineer must know the limits of the basic foundation of the equations. The assumptions for the equations as presented are as follows:

1. The flow is steady.
2. Water is incompressible.
3. The continuity equation is valid.
4. The flow is essentially parallel.

Generally, the energy principle offers a simpler and clearer explanation than does the momentum principle. However, the momentum principle has many advantages in problems involving hydraulic jumps, hydraulic structures, and channel junctions.

6.3.4 Resistance to Flow

Roughness coefficients (Manning's n-values) vary considerably according to depth of flow, and type and quality of the surface material. Estimates of n-values should include consideration that roughness may vary with flood stage, depending on such factors as the width-depth ratio of the watercourse; presence of vegetation in the main channel; the types of materials making up the channel bed; and the degree of meandering. Guidance for selection of Manning's roughness coefficients for natural channels and floodplains, and unlined constructed channels, is provided in Chapter 7. Additional information concerning Manning's roughness coefficients can be found in Phillips and Ingersoll (1998), Thomsen and Hjalmarson (1991), Davidian (1984), Aldridge and Garrett (1973) and Barnes (1967).

Typical values of roughness coefficients for lined channels are given in Table 7.6. For each material and/or construction method listed, three possible values of n are given. These values should be interpreted as follows:

- minimum = new construction;
- normal = good maintenance; and
- maximum = deteriorated and/or poor maintenance.

The hydraulic design of a channel should be based upon the maximum n-value anticipated during the life of the structure. The maximum n-value for a particular channel material as listed in

Table 7.6, is representative of this design-life condition. Channel design based on the maximum n-value results in a conservative estimation of flow depth. Likewise, use of the minimum n-value results in estimation of the maximum velocity of flow in the channel. The minimum n-values as listed in Table 7.6 represent newly constructed conditions. Maximum expected channel velocity should be a consideration in the analysis of supercritical flow, hydraulic jumps, and forces on structures, among others.

It is recommended that both maximum and minimum n-values be applied in the design of channels to check for sufficient hydraulic capacity and stability of channel linings, respectively. The scour estimation should be based on the minimum n-values.

6.3.5 Uniform Flow

Manning's Equation

The most commonly used equations for analysis of open channel flow express mean velocity of flow as a function of the roughness of the channel, the hydraulic radius, and the slope of the energy gradient. They are empirical equations in which the values of constants and exponents have been derived from experimental data. Manning's equation is one of the most widely accepted and commonly used of the open channel equations:

$$V = \frac{1.486}{n} R^{2/3} S_f^{1/2} \tag{6.21}$$

Substituting Equation (6.4) and rearranging yields the familiar form of Manning's equation:

$$Q = \frac{1.486}{n} A R^{2/3} S_f^{1/2} \tag{6.22}$$

The Manning's roughness coefficient (n-value) is a measure of the frictional resistance exerted by a channel on the flow. The n-value can also reflect other energy losses such as those resulting from unsteady flow, extreme turbulence, and transport of suspended material and debris that are difficult or impossible to isolate and quantify. The reader is referred to Chapter 7 and to Barnes (1967) and Thomsen and Hjalmarson (1991) for discussion of the estimation of n-values for constructed, natural and composite channels.

The most common error in the application of Manning's equation is to substitute the bed slope of the channel, S_o, for the slope of the energy gradient, S_f. This substitution is correct only when the two gradients are parallel, as in the case of uniform flow. For a given condition of n, Q, and S_o, uniform flow is maintained only at normal depth. Normal depth rarely occurs in nature, and it is primarily a theoretical concept that simplifies the computation and analysis of uniform flow. Table 6.1 lists the algebraic expressions for computing the hydraulic geometry for typical channel sections.

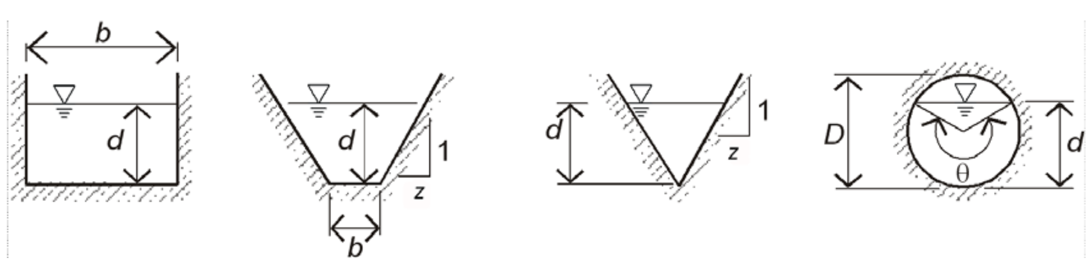
TABLE 6.1
ELEMENTS OF CHANNEL SECTIONS
(1)

Channel Section	Area	Wetted Perimeter	Hydraulic Radius	Top Width
Rectangle	bd	$b + 2d$	$\frac{bd}{b + 2d}$	b
Trapezoid	$bd + zd^2$	$b + 2d\sqrt{z^2 + 1}$	$\frac{bd + zd^2}{b + 2d\sqrt{z^2 + 1}}$	$b + 2zd$
Triangle	zd^2	$2d\sqrt{z^2 + 1}$	$\frac{zd}{2\sqrt{z^2 + 1}}$	$2zd$
Circular < 1/2 full (2)	$\frac{D^2}{8}\left(\frac{\pi\theta}{180} - \sin\theta\right)$	$\frac{\pi D\theta}{360}$		$D\sin\theta$ or $2\sqrt{d(D-d)}$
Circular > 1/2 full (3)	$\frac{D^2}{8}\left(2\pi\frac{\pi\theta}{180} + \sin\theta\right)$	$\frac{\pi D(360 - \theta)}{360}$	$\left(\frac{45D}{\pi(360 - \theta)}\right) * \left(2\pi - \frac{\pi\theta}{180} + \sin\theta\right)$	$D\sin\theta$ or $2\sqrt{d(D-d)}$

(1) After USDA Soil Conservation Service ES-33 (NRCS), 1956.

(2) $\theta = 4\sin^{-1}\sqrt{d/D}$ Insert θ in degrees

(3) $\theta = 4\cos^{-1}\sqrt{d/D}$ Insert θ in degrees



Rectangle | Trapezoid | Triangle | Circular

Composite Channels

The cross section of a natural or artificial watercourse or a street right-of-way may be composed of several distinct subsections, with each subsection having different hydraulic characteristics, such as hydraulic roughness and average flow depth. For example, a natural alluvial channel may have a primary, sand-bed channel which is bounded on both sides by densely-vegetated, overbank floodplains, or an urban flooded street section may be bounded on both sides by land-scaped front yards having shallower flow depths and slower flow velocities.

In composite channels like these, the discharge is computed for each subsection having distinct and different hydraulic characteristics, and the total computed discharge is set equal to the sum of the individual discharges. Similarly, the mean velocity for the entire flow cross section is assumed to be equal to the total discharge divided by the total water area. Open Channel Hydraulics (Chow, 1959), provides an example of computing flow in channels having composite roughness.

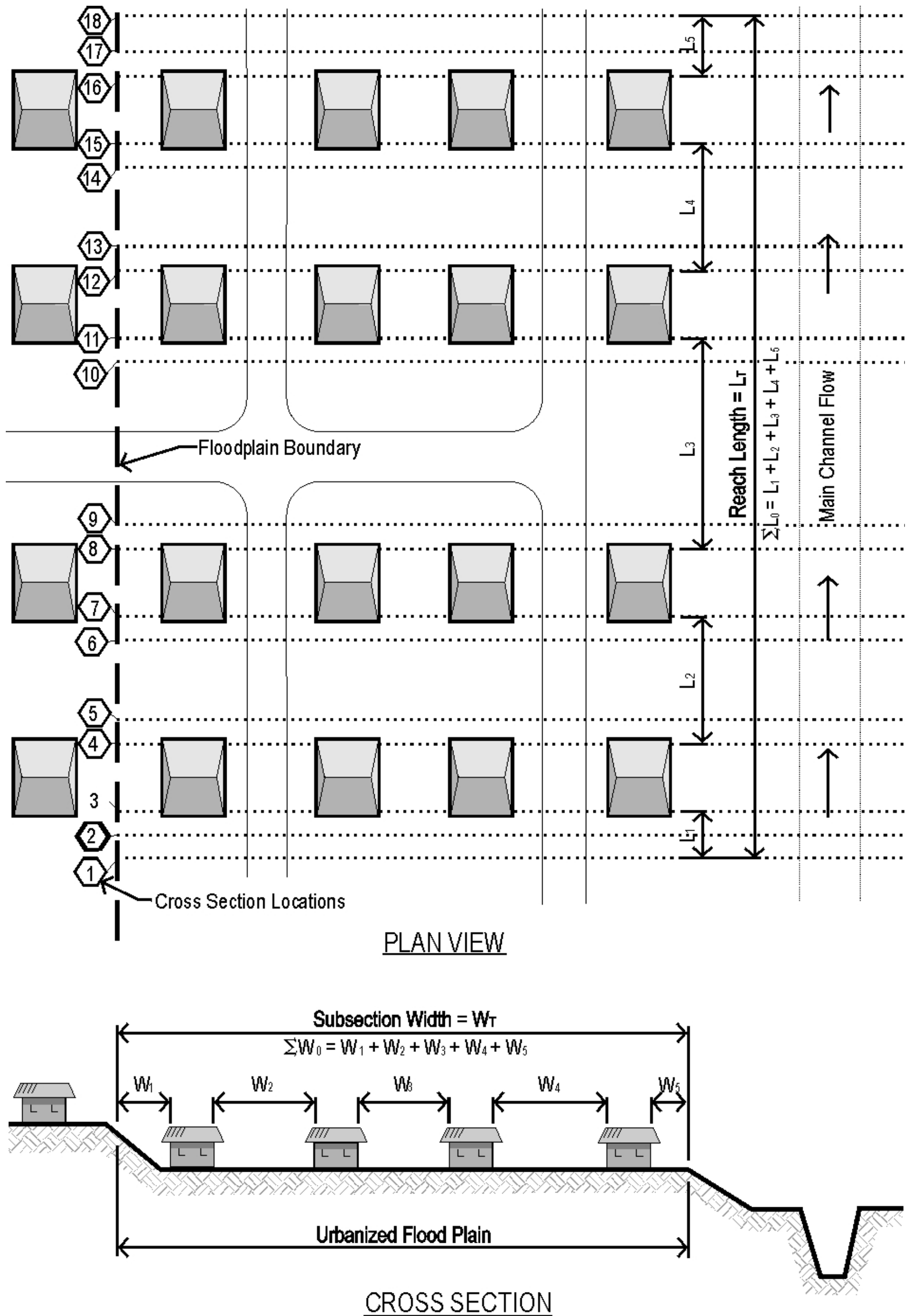
In the urban setting, it is not unusual for buildings and other structures to occupy a significant portion of any given hydraulic cross section. Under these circumstances, it is often difficult to estimate both the effective width of the cross section and the Manning's roughness coefficient for the overbank areas. Given this situation, the engineer should eliminate the portion of the cross section occupied by the building.

Where only an estimate of the computed water surface elevation is needed, a second option may be selected. An adjusted urban roughness coefficient, n_u, may be computed and applied to the total cross sectional area (Hejl, 1977). See Figure 6.4.

$$n_u = n_o\left(1.5\left(\frac{W_T}{\Sigma W_o}\right) + \left(1 - \frac{W_T}{\Sigma W_o}\right)\frac{\Sigma L_o}{L_T} - 0.5\right) \tag{6.23}$$

where all coefficients are as defined in Section 6.1.

FIGURE 6.4
DIAGRAM OF IDEALIZED URBAN FLOODPLAIN
(Hejl, 1977, JOURNAL OF RESEARCH, U.S. GEOLOGICAL SURVEY)



Examples:

The following examples illustrate the concept of normal depth and the selection of the roughness coefficient.

EXAMPLE 6.4: What is the hydraulic capacity of a shotcrete lined channel with a 20 foot bottom width, 2:1 side slopes, an invert gradient of 0.0016 ft./ft., and a uniform flow depth of 4.0 feet? Selecting the appropriate expressions from Table 6.1 for the cross section area and the hydraulic radius:

$$A = bd + zd^2 = (20)(4) + (2)(16) = 112\text{ft}^2$$

$$R = \frac{bd + zd^2}{b + 2d\sqrt{(z^2 + 1)}} = \frac{(20)(4) + (2)(4^2)}{20 + (2)(4)\sqrt{(2^2 + 1)}} = \frac{112}{37.89} = 2.96ft$$

Select the appropriate Manning's roughness coefficient from Table 7.6. Substituting these values in Equation (6.22):

$$Q = \frac{1.49}{n} AR^{2/3} S_f^{1/2}$$

$$Q = \frac{1.49(112.0)(2.96)^{2/3}(0.0016)^{1/2}}{0.022} = 625.5cfs$$

EXAMPLE 6.5: What is the normal depth of flow in a shotcrete lined channel with a 20-foot bottom width, 2:1 side slopes, an invert gradient of 0.0016 ft/ft and, a steady flow rate of 625 cfs?

Rearranging Equation (6.22):

$$AR^{2/3} = \frac{Qn}{1.49S^{1/2}} = \frac{(625)(0.022)}{(1.49)(0.0016^{1/2})} = 230.70$$

$$= (20d + 2d^2) \times \left[\frac{(20d + 2d^2)}{20 + (2d)\sqrt{(2^2 + 1)}}\right]^{2/3}$$

By trial and error solution d = 4.0 ft

6.3.6 Gradually Varied Flow

Classification of Water Surface Profiles

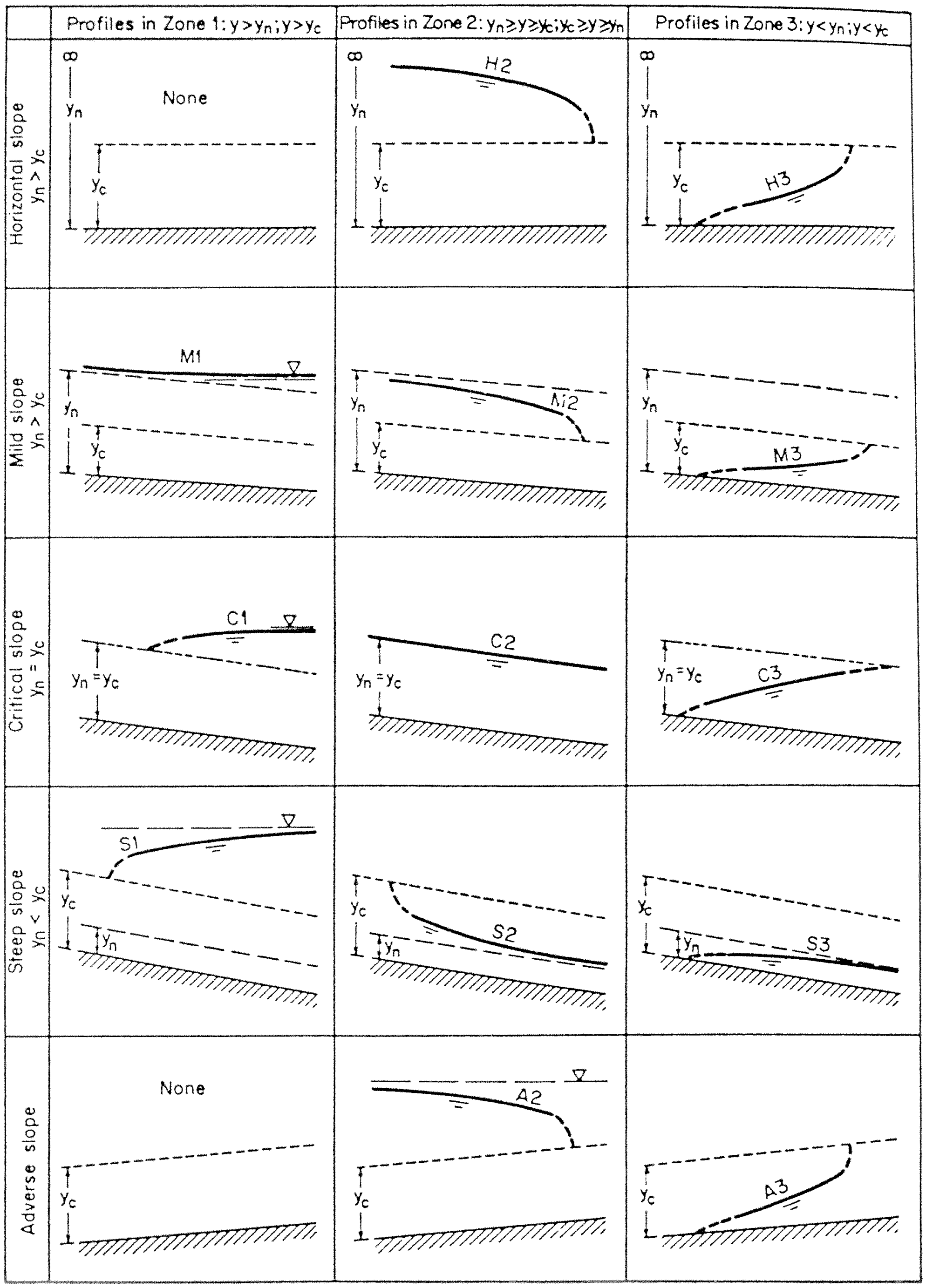
Chow (1959) describes the classification of these flow profiles into fifteen different types according to the nature of the channel slope and the zone in which the flow surface for a given discharge lies. These water surface profile types are designated according to an alphanumeric protocol, as follows:

- The **letter** is descriptive of the slope, i.e., **H** for horizontal, **M** for mild, **C** for critical, **S** for steep (supercritical), and **A** for adverse slope; and
- The **numeral** represents the zone number, where:
    - **Zone 1** – water surface above both normal and critical depths.
    - **Zone 2** – water surface between normal and critical depths.
    - **Zone 3** – water surface below both normal and critical depths.

These types are designated as H1, H2, H3; M1, M2, M3; C1, C2, C3; S1, S2, S3; and A1, A2, A3 as shown in Figure 6.5.

Flow profile analysis enables the designer to predict the general shape of the flow profile for a given channel layout. This step is a significant part of the open channel design process and it should not be omitted. Flow profile analysis will serve to identify control sections and to provide a work plan for more detailed design calculations.

FIGURE 6.5
CLASSIFICATION OF FLOW PORTION OF GRADUALLY VARIED FLOW
(Chow, 1959)



Calculation of Water Surface Profiles

Section 6.3.5 presents methods for calculation of normal depth which assume uniform flow. However, sudden changes in discharge, bed slope, and cross sectional area and/or form will produce additional energy losses which are not accounted for in Manning's equation. This may be particularly true in cases of sudden contractions and expansions of the channel cross section.

In those instances where an upstream or downstream hydraulic control section exists, the Standard Step Method should be used for evaluating water surface profiles. The procedure used for Standard Step calculations is presented in several of the technical references listed at the end of this chapter. The designer can perform the Standard Step calculations either manually using standard forms, or digitally using readily available and well-documented computer programs such as HEC-2 (USACE, 1990) or HEC-RAS (USACE, 2001a & b). These programs were developed by the U.S. Army Corps of Engineers and are available through the Corps web site at: http://www.hec.usace.army.mil.

One advantage of the Standard Step Method is the ability to converge an actual water surface profile for the study reach without needing to know the precise starting water surface elevation. If the computation is started at an assumed elevation that is incorrect for the given discharge, the resulting flow profile will approach the correct water surface elevation with each succeeding cross section evaluated within a study reach. If no accurate elevation is known within or near the reach under consideration, an arbitrary elevation may be assumed at a cross section far enough away from the “starting” cross section in the study reach to compensate for any initial error.

The step computations should be carried upstream if the flow is subcritical, and downstream if the flow is supercritical. Otherwise, step computations carried in the wrong direction will result in a profile that diverges from the actual water surface profile.

For natural streams flowing under supercritical conditions, the critical depth profile should be used as the water surface profile. Using the critical depth will produce higher, and thus more conservative, water surface elevations for design purposes. For FEMA floodplain delineation, a subcritical flow regime is normally used in HEC-RAS modeling to obtain more conservative water surface elevations. Velocities computed for the supercritical profile will be higher and more conservative and, therefore, should be used to evaluate scour potential and other velocity critical design features such as superelevation and freeboard.

The reader is referred to the technical references listed at the end of the chapter for more information regarding application of the standard step method and/or use of computer models such as HEC-2 and HEC-RAS for computation of water surface profiles. Specific references most instructive in this subject include Chow (1959) and USACE (1990, 2001a, 2001b), among others.

6.3.7 Control Sections

A quantitatively definitive relationship between the stage and discharge of flow in an open channel exists at a control section. The control section regulates the hydraulic properties of flow in such a way as to restrict the transmission of the effects of changes in flow condition either in the upstream or downstream direction depending on the flow regime in the channel. These sections are ideal beginning points for calculation of water surface profiles. A control is in any section where depth of flow is known, such as critical depth, depth upstream of a culvert, depth of flow over a weir and depth of flow under a gate.

6.4 GENERAL CONSIDERATIONS FOR OPEN CHANNEL DESIGN

6.4.1 Route Considerations

The design of a safe and economical drainage system should be one of the first steps in the land development process. Drainage system requirements may determine the character of the development, and often dictate the layout of streets and lots. Attention to drainage requirements during the first phases of planning will result in better land use decisions and lower maintenance costs.

A drainage system that is well planned and designed incorporates several features. The proposed drainage system should be aligned with any existing and proposed structures, such as bridges and culverts, and be designed in such a manner that subcritical flow is maintained throughout (except at designed drop structures). The design should incorporate uniform channel properties, such as gradient and cross sectional geometry, as much as possible. Sharp and closely spaced curves should be avoided. Uncontrolled local runoff should not be allowed to enter the channel; rather, it should be collected and discharged into the channel through a structure specifically designed for that purpose. In all cases, the issue of wet and dry weather safety should be a paramount consideration in route and right-of-way determinations.

6.4.2 Layout

Unless special exception is made by the governing agency, all artificial channels must begin and end where, historically, runoff has flowed.

The alignment of new drainage channels should follow existing washes, swales, and depressions whenever possible. The water must be collected and discharged at the same point and in the same manner as prior to the construction of the new channel. This means that the design of the new drainage features must account for runoff entering the property in the same location and manner as it historically flowed, and collect the water and transition it into the new channel for conveyance through the project site. At the downstream end of the channel, the drainage design must provide a transition from the on-site channel to return the runoff to its historic location prior to leaving the property. This requirement applies to the hydraulic geometry and velocity of the water, and the elevation of the water surface.

6.4.3 Grade Control

Regardless of the size of watershed, a key design element, including conceptual layout, is establishing whether or not grade control exists below the design section. General degradation and aggradation is beyond the scope of this manual; however, references are provided in Section 6.7.

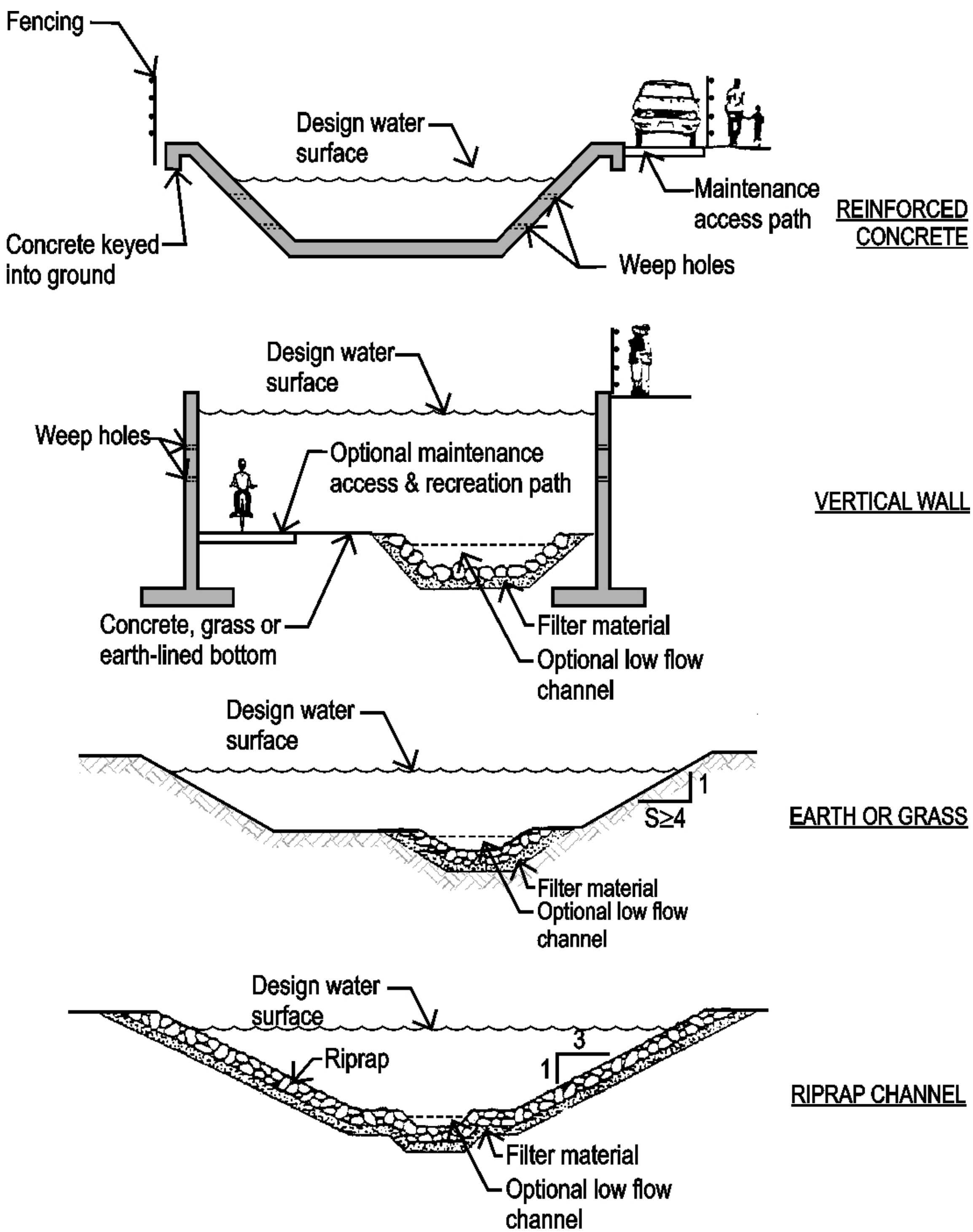
Grade control is a critical factor in the long-term behavior of non-rigid channels. By definition, grade control is any natural or man-made structure within a channel that limits or prevents vertical movement of the channel bed, either degradation or aggradation. Examples include rock outcroppings, culverts under embankments, drop structures, and bridges; however, not all drop structures, culverts, or bridges can be considered as grade control structures.

Grade control and channel slope are interrelated. In the design of grade control structures, the stability of the study reach must be assessed in context of the equilibrium of the entire system. The benefits of establishing grade control within a specific channel reach are minimal when the adjacent channel reach is either in a degradational or aggradational mode. When designing artificial channels, the designer needs to assess the stability of the reach immediately downstream from the segment under design. If there is evidence of ongoing downstream degradation, a grade control structure may be required. The grade control structure downstream side should extend to the total scour depth, which includes local scour due to grade control structure, long-term scour, general scour, and other scour components (see Chapter 11 for total scour estimation method). For each alternative investigated, the longitudinal spacing of grade control structures and the design slope of the channel should result in a stable channel.

6.4.4 Channel Linings

Artificial channel linings vary with the shape of the section and with the velocity of the water. Typical channel linings include concrete, soil cement, rock, earth (natural), and grass. These linings can be used alone or in combination with other linings. Typical linings and sections are shown in Figure 6.6.

FIGURE 6.6
TYPICAL CHANNEL SECTIONS



The type of stabilization that may be best suited for a particular purpose will depend upon a variety of factors, including hydraulic conditions, economic factors, soil conditions, material availability, aesthetics, maintenance and compatibility with existing improvements. The order of preference for subcritical flow conditions is natural channels with periodic grade-control structures, channels with vegetal linings, compound channels, channels lined with riprap, or its variations, channels lined with soil cement, and concrete-lined channels. Where supercritical flow conditions occur, only acceptable structurally sound channel linings such as concrete and shotcrete are recommended.

Earth Lined Channels

This category includes both bare earth and naturally vegetated channels in Maricopa County. Subsequent to construction, some revegetation will naturally occur, or landscaping practices may be used to establish growth of indigenous plant materials. For Maricopa County, this growth will be desert-like, with few grasses and a sparse spacing of other plants.

Earth lined channels are to be designed for subcritical flow regimes. Normally, these channels are relatively small and do not require low flow channels. If earth lining is used for larger channels, an armored low flow channel is required to control meandering and sediment deposition during low flow events. The low flow design should be checked for the effect that less frequent storms may have on sediment or scour, in terms of maintenance and aesthetic implications.

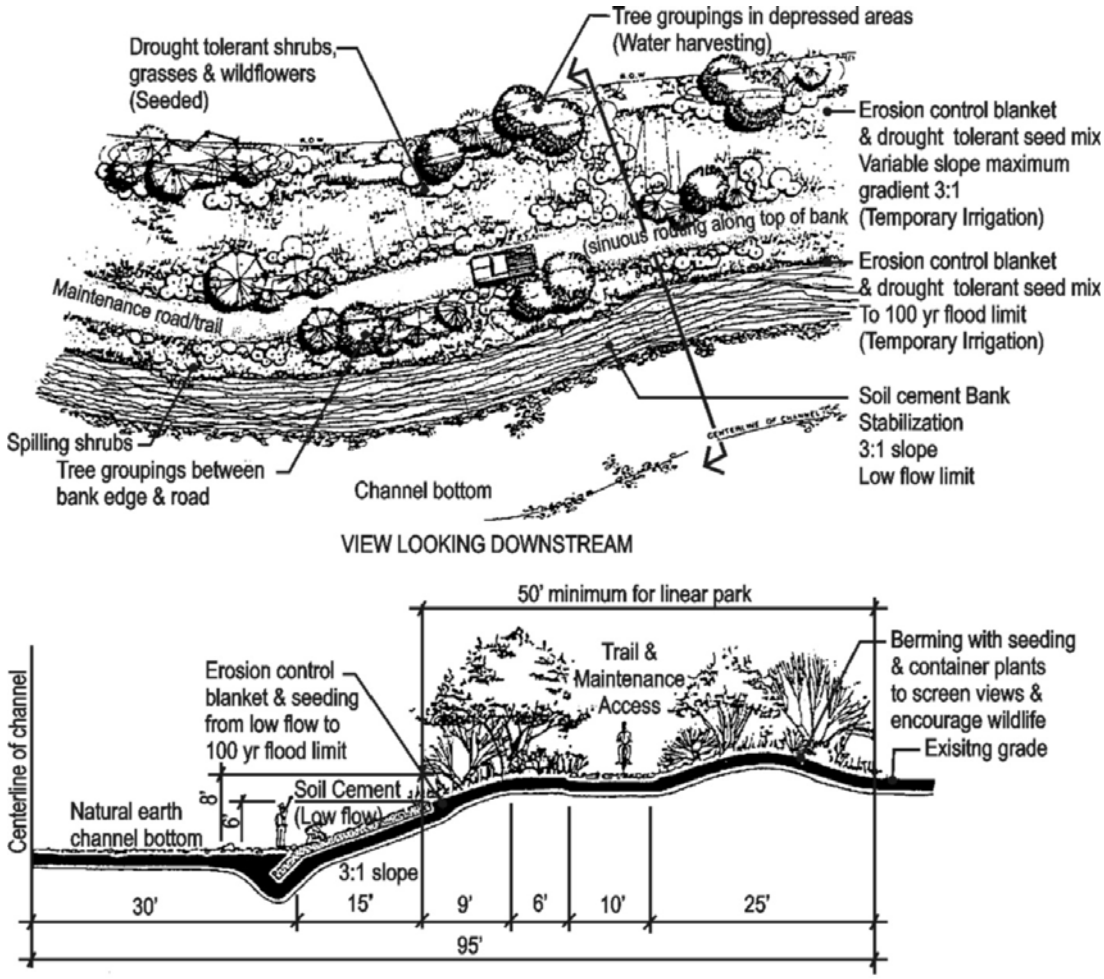
Grass Lined Channels

In a desert environment such as Maricopa County, there is not enough natural rainfall to maintain a grass lined channel without irrigation. Therefore, only those channels where an irrigation system is provided and maintenance can be performed are candidates for grass lining.

Compound Channels With Multi-Use Opportunities

A channel with a compound, or contoured cross section typically contains a smaller, interior channel that isolates frequent low-flows from upper portions of the channel. The upper portions of the channel which are only inundated during the less frequent storm events (typically, 100-year event), may then be utilized for landscaping and recreation opportunities (such as trails and bike paths). See Figure 6.7. Bank protection can extend from the channel bottom to the top of the low-flow channel; or it can extend the full height of the channel sides to the top of the high-flow portion of the channel, depending on the hydraulic characteristics of the channel.

FIGURE 6.7
COMPOUND CHANNEL
(Simons, Li, and Associates, 1989)



Rock Lined Channels

Rock lined channel lining includes both common riprap (graded rock) and gabion basket linings. Both types require a gravel filter layer and/or filter fabric between the rock layer and the natural ground. Excluding applications for hydraulic structures, gabion riprap is normally used when rock of sufficient size for common riprap is unavailable, poorly shaped, and/or overly expensive for a project. Gabion basket should not be used when the bed load has large cobbles that will damage the wires. Normally, rock linings are used for channels where right-of-way is limited (considering maximum side slope requirements) and subcritical flow can be maintained. These linings are also used immediately upstream and downstream of hydraulic structures. Refer to Section 6.6.3 and Section 6.6.6.

Soil Cement

Soil cement linings are composed of a thick layer (4-foot minimum) of unreinforced soil cement and are used successfully in many locations in Maricopa County. Soil cement is subject to weathering and abrasion. Soil cement can withstand relatively high velocities for short periods of time and, therefore, is most appropriate for channels with limited right-of-way or as a bank lining near bridges and culverts where local velocities tend to be high. Refer to Section 6.6.2.

Concrete Lined Channels

Concrete lined channels may be constructed of reinforced concrete or shotcrete. They are used primarily where right-of-way is limited and may be designed for either subcritical or supercritical flow. Concrete lined channels generally have steep side slopes because of the limited right-of-way. Inherently, these channels present public safety problems both in wet and dry weather.

The anticipated structural loads and the clearance requirements of the reinforcing steel will dictate the thickness of the concrete lining. Weep holes and subdrains are required to prevent uplift pressures from hydrostatic force in saturated conditions. Reinforced tie-ins are required at the top of the lining. These concepts are illustrated in Figure 6.6. Designers are cautioned against copying these details directly without first evaluating the design conditions for their specific project.

Concrete and shotcrete lined channels are discouraged in residential and recreational areas. If concrete channels are needed in these areas, the designer should contact the technical staff of the appropriate jurisdiction. Refer to Section 6.6.1.

6.4.5 Low Flow Channels

Some of the sections shown in Figure 6.6 have an optional low flow channel. Low flow channels are provided to minimize lateral meandering and sedimentation during low flow events. They also permit the incorporation of recreational amenities by preventing these facilities from being flooded during high frequency, low discharge flow events in compound channels.

Many large drainage basins have small base flows resulting from irrigation returns, treatment plant effluent, or urban cooling water. In addition, the most frequent runoff events are considerably smaller in magnitude than the storm for which the channel was designed. In the long term, these high frequency, low magnitude flows will deposit considerable amounts of sediment in the channel. Sediment deposition can cause redirection of flow into the channel banks resulting in erosion and/or a meandering low flow channel in the channel bottom. Earth and grass lined channels are particularly susceptible to this problem. When concrete low flow channels are used, riprap may be installed along both outer edges of the concrete low flow channels to prevent erosion. The riprap is especially needed at bends. It is recommended that low flow channels be provided whenever the following condition exists:

$$\frac{b}{Vy} \geq 1.40 \qquad (6.24)$$

where:

b = channel bottom width, ft,

V = average velocity, fps, and

y = depth of flow, ft.

6.4.6 Safety

Deep channels with steep side-slopes and high flow velocities can be a hazard to the health, safety, and welfare of the general public. Therefore, the design engineer must always consider the safety aspects of any design. The reader is referred to Chapter 1, Section 1.4 of this manual.

6.4.7 Maintenance

The design engineer must also consider maintenance issues associated with any design. At a minimum, a 16-ft maintenance access lane with access ramps is recommended to be provided on one side of a channel for publicly maintained channels. Refer to each jurisdiction's *Policies and Standards Manual* for specific criteria. To minimize maintenance; paths, walkways, play areas, and irrigation systems should be located in less frequently inundated levels of channels. Bottom widths of channels should be designed in consideration of maintenance requirements for the channel lining, and will be no narrower than 8 feet unless otherwise approved by the jurisdictional entity.

6.4.8 Confluence Junction

The design criteria for confluence junctions between a main channel and side channel should be based on USACE (1994). One of the key design criteria is that the angle of junction intersection should not be greater than 12 degrees. Other design criteria and procedure can be found in USACE (1994) and Section 8.6 in Chapter 8.

6.5 DESIGN FACTORS FOR OPEN CHANNELS

6.5.1 General

Good design practice requires that several issues be addressed. Unless exempted by the governing agency, water surface profiles must be computed for all channels during final design and clearly shown on a copy of the final drawings. Computation of the water surface profile should use standard step backwater methods (see Section 6.3.6). These computations must account for all losses due to changes in velocity, drops, bridge openings, and other factors.

Computations should begin at a known point and extend in an upstream direction for subcritical flow regimes, and in a downstream direction for supercritical regimes. Concrete lined channels with supercritical flow regimes should be analyzed as described in Section 6.3.6. The energy gradient must be shown on all preliminary drawings to help check for errors; however, it is optional for final drawings. Open channel flow in urban drainage is usually non-uniform due to bridge openings, channel curves, and hydraulic structures, therefore backwater computations must be used for all final channel design work.

6.5.2 Minimum Velocity

Very low velocities encourage sedimentation and undesirable plant growth, which decreases channel carrying capacity and promotes nuisance ponding. Channels must be designed with respect to sedimentation issues elaborated in Chapter 11. In general, there are two design philosophies for open channel design. One is to design a channel such that the velocity is low and no scour will occur. This can be achieved by lowering the channel bed slope through constructing drop structures. However, this design may cause a sedimentation problem. Therefore, sediment basins and regular sediment cleaning may be required. Another design philosophy is to design a "steeper" channel such that the sediment is moved through the channel. Because of higher velocities, erosion protection will be required for the channel banks and other structures. It may be noted that when an equilibrium slope is used as the channel slope, the long-term scour component should not be included into total scour estimation (see Chapter 11 for scour estimation). Culverts and storm drains should be designed such that sediments do not settle.

6.5.3 Maximum Velocity

For earthen or grass lined channels, maximum permissible velocities should be governed by Table 6.2 and Table 6.3, respectively. If the natural channel slope would cause excessive velocity, employ drop structures, checks, riprap (USDOT, FHWA, 1989), or other suitable velocity control design features. The maximum permissible velocities for concrete channels and other revetments can be found in Maricopa County, 2007.

TABLE 6.2
MAXIMUM PERMISSIBLE VELOCITIES FOR ROADSIDE DRAINAGE CHANNELS
WITH ERODIBLE LININGS
(USDOT, FHWA, 1961 AND 1988)

Soils Type of Lining (Earth, No Vegetation)	**Permissible Velocity (1)(2) (ft/sec)**
Fine Sand (noncolloidal)	2.5
Sandy Loam (noncolloidal)	2.5
Silt Loam (noncolloidal)	3.0
Ordinary Firm Loam	3.5
Fine Gravel	5.0
Stiff Clay (very colloidal)	5.0
Graded, Loam to Cobbles (noncolloidal)	5.0
Graded, Silt to Cobbles (noncolloidal)	5.5
Alluvial Silts (noncolloidal)	3.5
Alluvial Silts (colloidal)	5.0
Coarse Gravel (noncolloidal)	6.0
Cobbles and Shingles	5.5
Shales and Hard Pans	6.0

(1) For sinuous channels multiply permissible velocity by:

0.95 for slightly sinuous;

0.90 for moderately sinuous; and

0.80 for highly sinuous

(2) Higher velocities may be allowed for design of unlined channels, for the 100-year design event in particular, based on sediment balance considerations defined using the guidelines in Chapter 11. However, sufficient setback allowance should be provided for expected bank erosion during the 100-year event, or a series of annualized events over a 60-year period. Higher velocities may also be acceptable for 100-year peak flow design with approved engineering justification based on a tractive force analysis (USDOT, FHWA HEC-11, 1989).

TABLE 6.3
MAXIMUM PERMISSIBLE VELOCITIES FOR GRASS-LINED ROADSIDE CHANNELS
UNIFORM STAND OF GRASS COVER AND WELL MAINTAINED
(ADAPTED FROM USDOT, FHWA 1961 AND 1988)(1)(2)(3)

Cover	Permissible Velocity (ft/sec)
Bermuda Grass	6.0
Desert Salt Grass Vine Mesquite	5.0
Lehman Lovegrass Big Galleta Purple Threeawn Sand Dropseed	3.5

(1) Use velocities over 5 ft/sec only where good covers and proper maintenance can be obtained.
(2) Grass is accepted only if an irrigation system is provided.
(3) Grass lined channels not recommended for slopes greater than 5%.

6.5.4 Freeboard

Freeboard is the distance between the calculated water surface and the top of the channel lining or bank. The minimum freeboard is calculated as follows:

$$FB = 0.25\left(y + \frac{V^2}{2g}\right) \tag{6.25}$$

In subcritical channels, the minimum required freeboard is the larger of 1 foot or that calculated using Equation (6.25). In supercritical channels, the required freeboard is the larger of 2 feet or the results of Equation (6.25). In all instances, the freeboard required is additive to any increases in water surface due to superelevation or channel curvature. Freeboard for levees must meet FEMA freeboard requirements (3, 3.5 or 4 feet minimum depending on location relative to end of levee, and to other structures). Refer to 44 CFR Section 65.10: Mapping of Areas Protected by Levee Systems (USGPO, 2000).

For sand-bed channels, when the Froude Number is equal to or larger than 0.7, the freeboard shall be the larger value of $0.027V^2$ or $0.25(y+V^2/(2g))$ where V is the channel velocity and y is the flow depth. The reason is as follows. When the Froude Number reaches 0.7 in sand-bed channels, an antidune bed form may develop. Under the antidune bed form condition, the water surface wave is in phase with the sand wave on the channel bed, i.e., the peaks and troughs of the sand wave and water wave on the surface will occur simultaneously. The amplitude of the water surface wave may exceed that of the sand wave by a factor of 1.5 to 2 (Chien and Wan, 1998).

The amplitude for the antidune sand wave can be estimated by $0.027V^2$ where V is the channel velocity. Therefore, the water wave amplitude can be estimated by $0.054V^2$ if a factor of 2 is used. Water wave amplitude is the vertical distance between the wave peak and wave trough. Assuming the average water surface elevation is at the middle of the amplitude, the wave height above the water surface elevation is then $0.027V^2$.

Roll waves also known as slug flow are intermittent surges on steep slopes that will occur when the Froude Number is greater than 2.0 and the channel invert slope is greater than 12/Re where Re is the Reynolds Number (Chow, 1959). The Reynolds Number (Re) is defined as VL/ν where V is velocity fps, L is characteristic length ft, and ν is the kinematic viscosity. L can be assumed as flow depth for a wide open channel. When this occurs, it is important to estimate the wave height as part of freeboard design. Detailed discussions and design procedures and examples on roll wave height can be found in LACFCD (1982) and Brock (1967).

6.5.5 Channel Curvature

The minimum radius of a curved channel, measured to the channel centerline, carrying subcritical flows is recommended to be three times greater than the width of the water surface. That is:

$$r_c \geq 3T \tag{6.26}$$

If the channel is carrying supercritical flows, the recommended minimum radius is:

$$r_c = \frac{4V^2T}{gy} \tag{6.27}$$

6.5.6 Superelevation

Curves in a channel cause the maximum flow velocity to shift toward the outside of the bend. Along the outside of the curve, the depth of flow is at a maximum. The consequent rise in the water surface is referred to as superelevation. Under subcritical conditions, the following equation is recommended to estimate the magnitude of the superelevation:

$$y = \frac{0.5V^2T}{gr_c} \tag{6.28}$$

Readers are cautioned to avoid curves in channels with supercritical flows. The shift in the velocity distribution may cause cross-waves to form, which will persist downstream and could severely limit the hydraulic capacity of the channel. Advanced design criteria or physical model studies beyond the scope of this chapter may be required.

6.6 DESIGN GUIDELINES FOR OPEN CHANNELS

6.6.1 Concrete Lined Channels

Reinforced concrete and shotcrete are alternative lining materials for channels with limited right of way and/or high velocity flow. The most common problems of concrete lined channels are due to bedding and liner failures. Typical failures are: 1) liner cracking due to settlement of the subgrade; 2) liner cracking due to the removal of bed and bank material by seepage force; and 3) liner cracking and floating due to hydrostatic back pressure from high groundwater.

Lack of maintenance can result in vegetation growth through the concrete lining and sediment deposition in the channel that will increase the flow resistance. This reduction in channel capacity can cause overflow at design discharges and, consequently, permit the erosion of overbank material and failure of concrete lining.

Concrete lined channels are usually designed for high velocity flow conditions. Froude Numbers for supercritical flow shall be greater than 1.13 and less than 2.0. Unstable flow conditions occur when the Froude number falls between 0.86 and 1.13 and must be avoided.

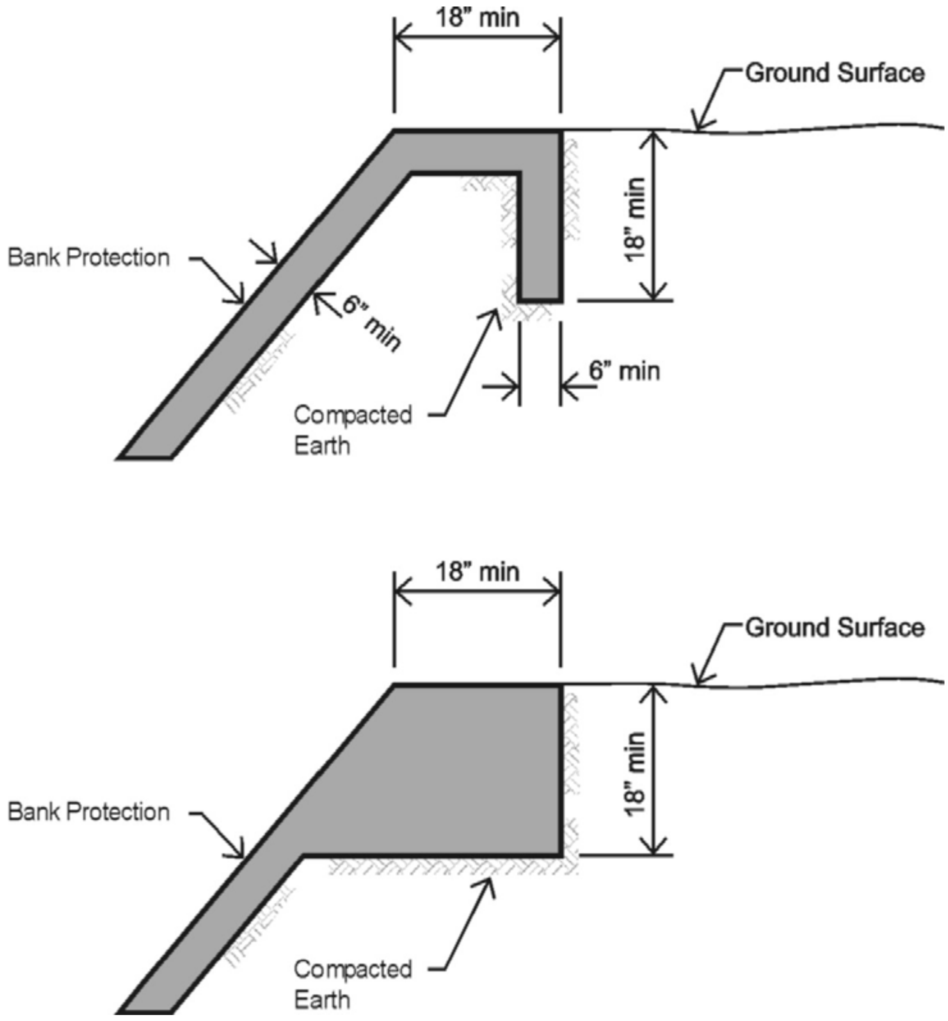
Supercritical flow in an open channel in an urbanized area creates certain hazards that the designer must take into consideration. From a practical standpoint it is generally unwise to have any curvature in a supercritical channel. Careful attention must be taken to prevent or control excessive oscillatory waves that may extend the entire length of the channel from only minor obstructions upstream. Imperfections at joints may rapidly cause a deterioration of the joints, in which case a complete failure of the channel can readily occur. High velocity flow can enter cracks or joints and create uplift forces by the conversion of velocity head to pressure head causing damage to the channel lining. It is evident that when designing a lined channel with supercritical flow, the designer must use utmost care and consider all relevant factors.

All concrete lined channels must have continuous reinforcement extending both longitudinally and laterally. For channels carrying supercritical flow, there shall be no reduction in cross sectional area at bridges or culverts, or any obstructions in the flow path.

Bridges or other structures crossing the channel must be anchored satisfactorily to withstand the full dynamic load that might be imposed upon the structure in the event of major debris blockage. Tributary storm drain pipelines must not protrude into the channel flow area.

Generally, if side slopes steeper than 2:1 are used, then safety and structural requirements become a primary concern. To determine the thickness of the lining refer to ADOT (1989). Design of the lining should also include consideration of anticipated vehicular loading from maintenance equipment. Joints in the lining should be designed in accordance with standard structural analysis procedures with consideration of the size of the channel, thickness of the lining and anticipated construction techniques. The concrete lining must be keyed into the adjacent over-banks as shown in Figure 6.8.

FIGURE 6.8
TYPICAL BANK-PROTECTION KEY-INS
(NOT TO SCALE; Simons, Li and Associates, 1989A)



The roughness coefficient for a concrete lining can vary from 0.011 for a troweled finish to 0.020 for a very rough or unfinished surface. Refer to Table 7.6. For shotcrete, roughness coefficients can vary from 0.016 to 0.025. The accumulation of sediment and debris must be taken into account when determining the roughness coefficient.

Long-term stability of concrete lined channels depends in part on proper bedding. Undisturbed soils often are satisfactory for a foundation for lining without further treatment. Expansive clays are usually an extreme hazard to concrete lining and should be avoided. A filter underneath the lining is recommended to protect fine material from creeping along the lining. A well-graded gravel filter should be placed over the channel bed prior to lining the channel with concrete.

Since concrete-lined channels are often used at locations where excessive seepage exists or smaller channel cross sections are required, transitions will be required both upstream and downstream of the concrete lined channel. Such transitions are intended to prevent undermining of the lining and to reduce turbulence. Transitions should be lined with concrete or other scour resistant material to reduce scour potential.

Cutoff walls should be incorporated with transitions at both the upstream and downstream end of the concrete lined channel to reduce seepage forces and prevent lining failure due to scour, undermining, and piping. The depth of cutoff walls should extend below the expected scour depth. Determination of expected total scour depth requires analyses as discussed in Chapter 11.

The probability of damaging the concrete lining due to hydrostatic back pressure and subgrade erosion can be greatly reduced by providing underdrains. There are two types of artificial drainage installations. One type consists of 4- or 6-inch diameter perforated pipelines placed in gravel-filled trenches along one or both toes of the inside slopes. These longitudinal drains are either connected to transverse cross drains which discharge the water below the channel or to pump pits, or extend through the lining and connect to outlet boxes on the floor of the channel. The outlet boxes are equipped with one-way flap valves that prevent backflow and relieve any external pressure that is greater than the water pressure on the upper surface of the channel bottom. The second type consists of a permeable gravel blanket of selected material or sand and gravel pockets, drained into the channel at frequent intervals (10 to 20 feet) by flap valves in the channel invert. Figure 6.9 shows a drawing of a flap valve for use without tile pipe and in a fine gravel and sand subgrade. Both the tile and pipe system and the unconnected flap valve type must be encased in a filter that will prevent piping of subgrade material into the pipe or through the valve. For detailed information on underdrains refer to *Lining for Irrigation Canals* (USBR, undated).

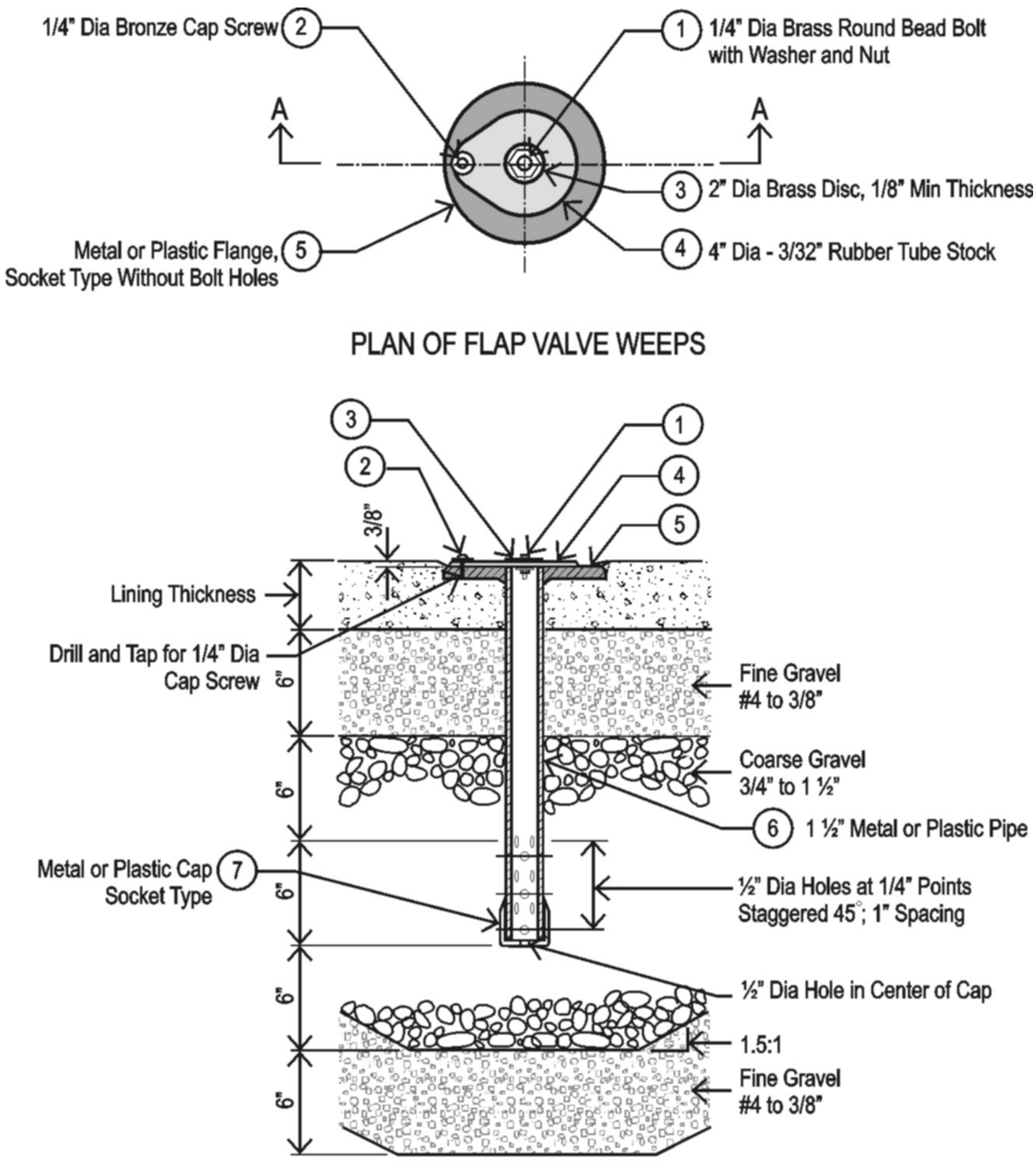
Where a lesser degree of seepage control is warranted, weep holes spaced at appropriate intervals may be used. When embankment stability may be compromised or when groundwater levels may be raised by back drainage from the lined channel, weep holes may be equipped with flap valves or other measures that allow seepage relief but prevent backflow or introduction of surface water behind the lining.

The shotcrete process has become an important and widely used technique. Shotcrete is mortar or concrete pneumatically projected at high velocities onto a surface. In the past, the term ‘gunite’ was commonly used to designate dry-mix mortar shotcrete. The term is currently outdated and ‘shotcrete’ has become the trade name for all pneumatically applied dry-mix or wet-mix concrete or mortar.

ACI 506R (2005) discusses the properties, applications, materials, reinforcement, equipment, shotcrete crews, proportioning, batching, placement, and quality control of the shotcrete process. As a channel lining, shotcrete is an acceptable method of applying concrete with a general improvement in density, bonding, and decreased permeability. The same design considerations discussed for concrete channels apply in the design of shotcrete channels. Shotcrete linings are to be designed to the same thickness and reinforcement as required for concrete linings. Given the limitations of construction, the minimum slope for concrete and shotcrete channels is 0.0015 ft/ft.

.

FIGURE 6.9
FLAP VALVE INSTALLATION FOR A CHANNEL UNDERDRAIN
(Simons, Li and Associates, 1981)



6.6.2 Soil Cement Lined Channels

Soil cement has been shown to be an effective and economical method for slope protection and channel lining in the Maricopa County area.

Materials

A wide variety of soils can be used to make durable soil cement. For maximum economy and most efficient construction, it is recommended that:

1. The soil contains no material retained on a 3-inch (75 mm) sieve;
2. Between 40 percent and 80 percent pass the No. 4 (4.75 mm) sieve; and
3. Between 2 percent and 10 percent pass the No. 200 (0.074 mm) sieve.
4. The Plasticity Index (PI) of the fines should not exceed 10.

If the onsite material does not meet these guidelines, the addition of import material may be necessary. Standard laboratory tests are available to determine the required proportions of cement and moisture to produce durable soil cement. The design of most soil cement for water control projects is based on the cement content indicated by ASTM testing procedures and increased by a suitable factor to account for direct exposure, erosion or abrasion forces.

The Portland cement should comply with one of the following specifications: ASTM C150, CSA A5, or AASHTO M85 for Portland cement of the type specified; or ASTM C595 or AASHTO M240 for Portland blast-furnace slag or Portland pozzolan cement, excluding slag cements Types S and SA.

It is important that testing to establish required cement content be done with the specific cement type, soil, and water that will be used in the project.

Typically, soil cement linings are constructed by the central-plant method, where selected onsite soil materials, or soils borrowed from nearby areas, are mixed with Portland cement and water and transported to the site for placement and compaction.

Design of Soil Cement Linings

Figure 6.10 shows a composite channel consisting of an earth bottom with soil cement stabilization along the banks. On side slopes, the soil cement is often constructed by placing and compacting the material in horizontal layers stair-stepped up the slope. The rounded step facing results from ordinary placement and compaction methods. Generally, an 8 to 9 foot minimum working width is required for placement and compaction of the soil cement layers by standard highway construction equipment. A width of 9-feet is preferred for maintenance and safety reasons. Figure 6.11 shows the relationship between slope of facing, thickness of compacted horizontal layer, horizontal layer width and minimum facing thickness measured normal to slope. For a horizontal working width of 9 feet, a side slope of 2:1 and 6-inch thick layers, the resulting minimum thickness of facing would be about 4 feet, measured normal to the slope. The sideslope can vary from 1:1 to 3:1 depending on the soil type and natural angle of repose. Side slopes steeper than 2:1 are not recommended, due to safety issues, but may be allowed when right-of-way is a problem. Soil cement may be placed on slopes 3:1 or flatter at a minimum thickness of eight to twelve inches, depending upon the mixing technique. This would be done without the stair-step layer approach, where a lesser level of protection is permissible.

An important consideration in the design of the soil cement facing is to provide that all extremities of the facing are tied into non-erodible sections or abutments. The upstream and downstream ends of the facing should terminate smoothly into the natural channel banks. A buried cutoff wall normal to the slope or other measures may be necessary to prevent undermining of the soil cement facing by flood flows.

The top of the lining should be keyed into the ground to protect against erosion of the backside of the soil cement layer by lateral inflows, as shown in Figure 6.8. As with any impervious channel lining system, seepage and related uplift forces should be considered and, if required, appropriate counter-measures provided, such as weep holes or subdrains. Tributary storm drain pipelines can normally be accommodated by placing and compacting the soil cement by hand, using small power tools, or by using a lean mix concrete. For earthen channels with soil cement side slope protection, the lining should be designed to extend to the anticipated depth of total scour below thalweg. Further design information may be found in ACI 230.1(1990), *State Of The Art Report On Soil Cement*. Additional information on design and construction is available from the Portland Cement Association, Skokie, IL (http://www.cement.org/).

FIGURE 6.10
SOIL CEMENT PLACEMENT DETAIL
(NOT TO SCALE)

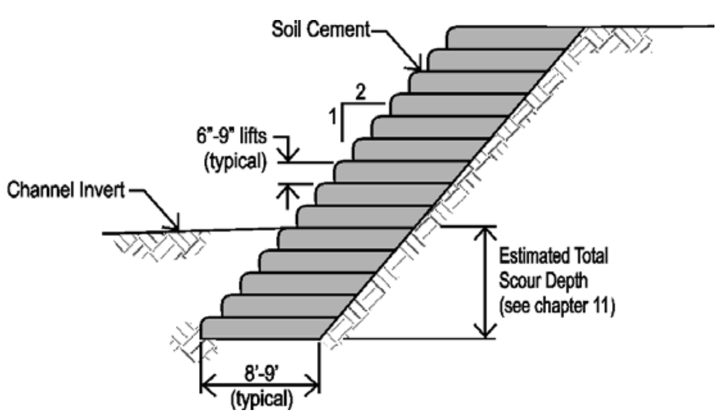
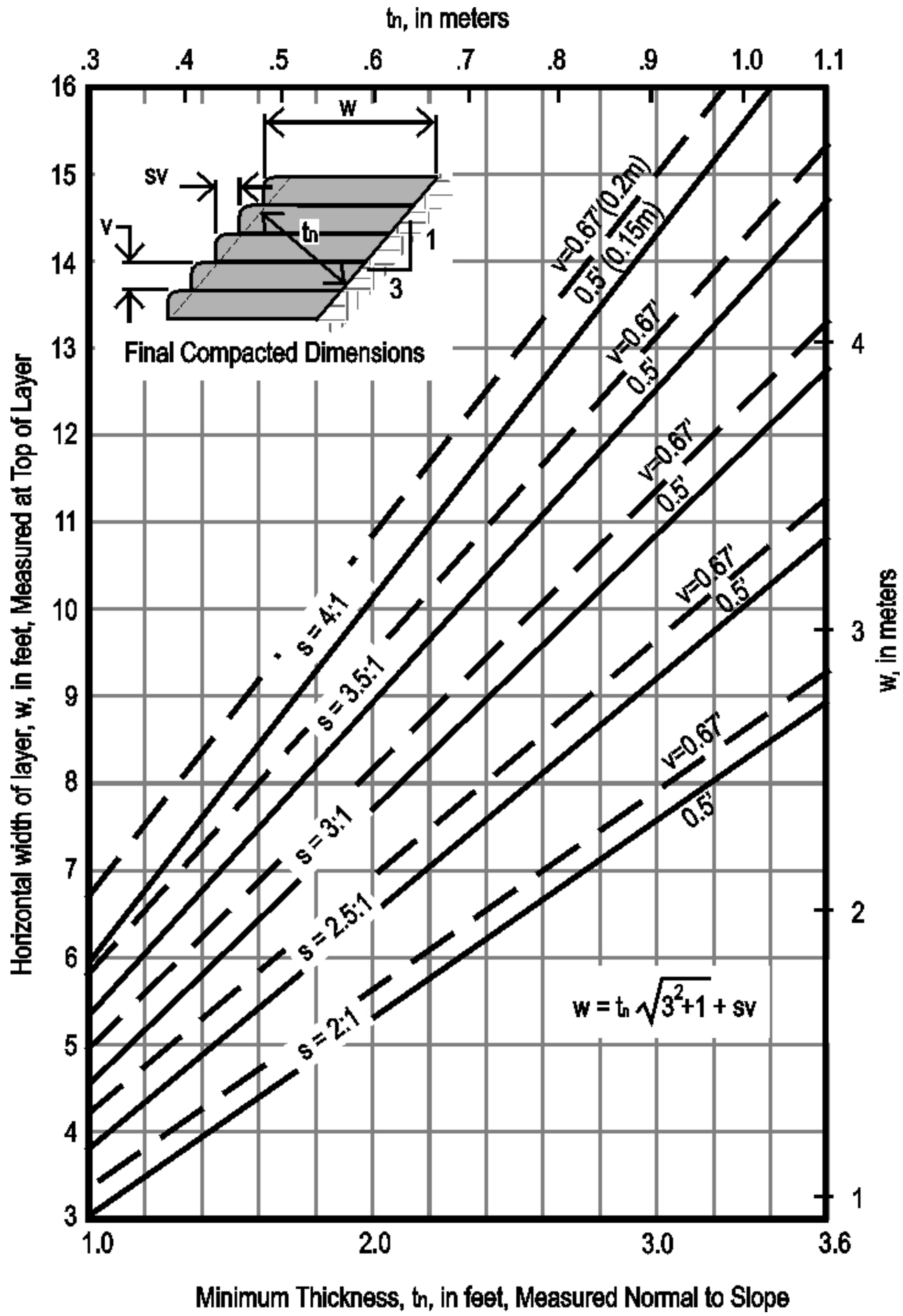


FIGURE 6.11
RELATIONSHIPS FOR SOIL CEMENT LINING
SLOPE, FACING THICKNESS, LAYER THICKNESS, AND HORIZONTAL LAYER WIDTH
(Portland Cement Association, 1987)



6.6.3 Riprap Lined Channels

Common riprap can be an effective lining material if properly designed and constructed. The choice of riprap usually depends on the availability of graded rock with suitable material properties and at a cost that is competitive with alternative lining systems.

Riprap design involves the evaluation of five performance areas. These areas include the evaluation of:

- riprap quality;
- riprap layer characteristics;
- hydraulic requirements;
- site conditions; and
- river conditions.

In Arizona, site requirements and river conditions are important factors in the protection of bridge structures and flood control channels.

Riprap Quality

Riprap quality determination refers to the physical characteristics of the rock particles that make up the bank protection. Qualities determined to be most important include density, durability, and shape. Requirements for each of these properties are summarized in this section.

Specific Gravity - The design stone size for a channel depends on the particle weight, which is a function of the specific gravity of the rock material. All stones composing the riprap should have a specific gravity equal to or exceeding 2.5, following the standard test ASTM C127 (2007). It may be noted that the minimum specific gravity required by MAG (2012) is 2.5.

Durability - Durability addresses the in-place performance of the individual rock particles, and also the transportation of riprap to the construction site. In-place deterioration of rock particles can occur due to cycles of freezing and thawing, or can occur during transportation to the site. The rock particles must have sufficient strength to withstand abrasive action without reducing the gradation below specified limits. Qualitatively, a stone that is hard, dense, and resistant to weathering and water action should be used. Rocks derived from igneous and metamorphic sources provide the most durable riprap.

Laboratory tests should be conducted to document the quality of the rock. Specified tests that should be used to determine durability include: the durability index test and absorption test (see ASTM C127). Based on these tests, the durability absorption ratio (DAR) is computed as follows:

$$DAR = \frac{Durability\ Index}{Percent\ Absorption + 1} \tag{6.29}$$

The following specifications are used to accept or reject material:

1. DAR greater than 23, material is accepted;
2. DAR less than 10, material is rejected;
3. DAR 10 through 23:
   a. Durability index 52 or greater, material is accepted; and,
   b. Durability index 51 or less, material is rejected.

Shape - There are two basic shape criteria. First, the stones should be angular. Angular stones with relatively flat faces will form a mass having an angle of internal friction greater than rounded stones, and therefore will be less susceptible to slope failures. Second, not more than 25 percent of the stones should have a length more than 2.5 times the breadth. The shape of the riprap stone should be cubical, rather than elongated. Cubical stones nest together, and are more resistant to movement. The length is the longest axis through the stone, and the breadth is the shortest axis perpendicular to the length. Angularity is a qualitative parameter which is assessed by visual inspection. No standard tests are used to evaluate this specification. If the engineer is faced with a supply of rounded river rock without a crusher to create angular rock, stone size should be increased 25% and side slopes decreased (USACE, 1994). However, no rounded riprap may be used for sloped drop structures or rock chutes.

Test Methods - The MAG (2012) and ASTM (2007) test methods and requirements should be followed.

Riprap Layer Characteristics

The major characteristics of the riprap layer include: characteristic size; gradation; thickness; and filter-blanket requirements.

Characteristic Size - The characteristic size in a riprap gradation is the d_{50}. This size represents the average diameter of a rock particle for which 50 percent of the gradation is finer, by weight.

Gradation - To form an interlocked mass of stones, a range of stone sizes must be specified. The object is to obtain a dense, uniform mass of durable, angular stones with no apparent voids or pockets. The recommended maximum stone size is 2 times the d_{50} and the recommended minimum size is one-third of the d_{50}.

The gradation coefficient, G, should equal 1.5.

$$G = 0.5(d_{84}/d_{50} + d_{50}/d_{16}) \tag{6.30}$$

Table 6.4 provides design gradations for riprap. As a practical matter, the designer should check with local quarries and suppliers regarding the classes and quality of riprap available near the site.

TABLE 6.4
RIPRAP GRADATION LIMITS
(USDOT, 1989)

Stone Size Range (ft.)	Stone Weight Range (lb)	Percent of Gradation Smaller Than
1.5 d_{50} to 1.7 d_{50}	3.0 W_{50} to 5.0 W_{50}	100
1.2 d_{50} to 1.4 d_{50}	2.0 W_{50} to 2.75 W_{50}	85
1.0 d_{50} to 1.15 d_{50}	1.0 W_{50} to 1.5 W_{50}	50
0.4 d_{50} to 0.6 d_{50}	0.1 W_{50} to 0.2 W_{50}	15

Thickness - The riprap-layer thickness shall be the greater of 1.0 times the d_{100} value, or 1.5 times the d_{50} value. But the thickness need not exceed twice the d_{100} value. The thickness is measured perpendicular to the slope upon which the riprap is placed.

Filter Blanket Requirements - The purpose of granular filter blankets underlying riprap is two-fold. First, they protect the underlying soil from washing out; and, second, they provide a base on which the riprap will rest. The need for a filter blanket is a function of particle-size ratios between the riprap and the underlying soil which comprise the channel bank. The inequalities that must be satisfied are as follows:

$$\frac{(d_{15})_{filter}}{(d_{85})_{base}} < 5 < \frac{(d_{15})_{filter}}{(d_{15})_{base}} < 40 \tag{6.31}$$

$$\frac{(d_{50})_{filter}}{(d_{50})_{base}} < 40 \tag{6.32}$$

In these relationships, “filter” refers to the overlying material and “base” refers to the underlying material. The relationships must hold between the filter blanket and base material and between the riprap and filter blanket (USDOT, 1988 and 1989).

If the inequalities are satisfied by the riprap itself, then no filter blanket is required. If the difference between the base material and the riprap gradations are very large, then multiple filter layers may be necessary. To simplify the use of a gravel filter layer, Table 6.5 outlines recommended standard gradations.

The Type-I and Type-II bedding specifications shown in Table 6.5 were developed using the criteria given in Equation (6.31) and Equation (6.32), considering that very fine grained, silty, non-cohesive soils can be protected with the same bedding gradation developed for a mean grain size of 0.045 mm. The Type-I bedding in Table 6.5 is designed to be the lower layer in a two-layer filter for protecting fine grained soils. When the channel is excavated in coarse sand and gravel (i.e., 50 percent or more by weight retained on the No. 40 sieve), only the Type-II filter is required. Otherwise, two bedding layers (Type-I topped by Type-II) are required. For the required bedding thickness, see Table 6.6.

TABLE 6.5
GRADATION FOR GRAVEL BEDDING
(Simons, Li and Associates, 1989)

Standard Sieve Size	Type I (1)	Type II (1)
3 inches	-	90 to 100
1-1/2 inches	-	-
3/4 inch	-	20 to 90
3/8 inch	100	-
#4 (4.76 mm)	95 to 100	0 to 20
#16 (1.18 mm)	45 to 80	-
#50 (0.30 mm)	10 to 30	-
#100 (0.149 mm)	2 to 10	-
#200 (0.074 mm)	0 to 2	0 to 3

(1) Percent passing by weight

TABLE 6.6
THICKNESS REQUIREMENTS FOR GRAVEL BEDDING
(Simons, Li and Associates, 1989)

Riprap Size Classification (in)	Minimum Bedding Thickness (in)		
	Fine Grain Native Soils		Coarse Grain Native Soils
	Type I	Type II	Type III
6, 8	4	4	6
12	4	4	6
18	4	6	8
24	4	6	8
30	4	8	10
36	4	8	10

Filter Fabric Requirements - The design criteria for filter fabric are a function of the permeability of the fabric and the effective opening size. The permeability of the fabric must exceed the permeability of the underlying soil, and the apparent opening size (AOS) must be small enough to retain the soil.

The criteria for apparent opening size are as follows:

1. For soil with less than 50 percent of the particles, by weight, passing a No. 200 sieve, the AOS should be less than 0.6 mm (a No. 30 sieve).

2. For soil with more than 50 percent of the particles, by weight, passing a No. 200 sieve, the AOS should be less than 0.3 mm (a No. 50 sieve).

The detailed specifications can be found in Section 796 of MAG (2012). Filter fabric is not a complete substitute for granular bedding. Filter fabric provides filtering action only perpendicular to the fabric and has only a single equivalent pore opening between the channel bed and the riprap. Filter fabric has a relatively smooth surface which provides less resistance to stone movement. Tears in the fabric greatly reduce its effectiveness so that direct dumping of riprap on the filter fabric is not allowed and due care must be exercised during construction. The site conditions and specific application and installation procedures must be carefully considered in evaluating filter fabric as a replacement for granular bedding material. Filter fabric can provide an adequate bedding for channel linings along uniform mild sloping channels where leaching forces are primarily perpendicular to the fabric.

Numerous failures have occurred because of the improper installation of filter fabric. Therefore, when using filter fabric it is critical that the manufacture's guidelines for installing it be followed.

Hydraulic Design Requirements

General - Channel linings constructed of placed, graded riprap or gabions to control channel erosion have been found to be cost effective where channel reaches are relatively short and where a nearby source of quality rock is available.

Situations where riprap or gabion basket linings may be appropriate are:

1. Major flows are found to produce channel velocities in excess of allowable non-eroding values;

2. Channel side slopes at 3:1 for riprap and 2:1 for gabion mattresses; and

3. Where rapid changes in channel geometry occur, such as channel bends and transitions.

This section presents design requirements for common riprap, while Section 6.6.6 contains additional design considerations specifically related to gabions. Both sections are valid only for subcritical flow conditions where the Froude Number is 0.86 or less.

Loose Angular Riprap Sizing (d_{50})

In Simons and Senturk (1992) and ASCE (2006), the Isbash equation for low turbulent flow has a term which accounts for bank slope effects. However, in USACE (1994), the Isbash equation does not account for bank slope effects, but has coefficients to account for both low and high turbulent flows. By combining these equations, the Flood Control District of Maricopa County (FCDMC) has developed a modified Isbash equation which accounts for both the bank slope effects and the flow regime (whether low or high turbulent flows).

In USACE (1994), the Isbash equation is based on an average channel velocity. However, the channel velocity for a cross section is not uniform. The maximum velocity is higher than the average velocity. The maximum velocity usually occurs in the middle of a cross section. In alluvial channels, the main channel may laterally migrate within the floodplain. Therefore, using maximum velocity is more reasonable. To account for the maximum velocity for a particular cross section that may occur anywhere, the FCDMC uses the maximum velocity, V. The maximum velocity can be approximated by $1.33V_a$ (Subramanya, 1997). The FCDMC-recommended modified Isbash equation has the form:

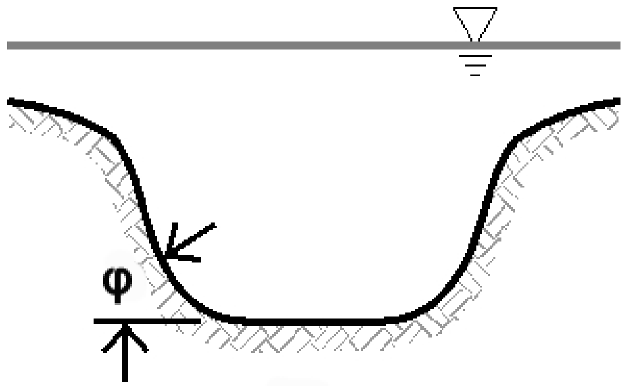
$$d_{50} = \frac{V^2}{2gC^2\cos\varphi}\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right) \tag{6.33}$$

where:

- V = maximum velocity $V = 1.33V_a$, (Subramanya, 1997), (ft/s),
- V_a = average velocity (ft/s),
- C = coefficient (use 1.2 for low turbulence areas or 0.86 for high turbulence areas),
- g = gravitational acceleration (ft/s^2),
- γ_s = specific weigh of stone (lb/ft^3),
- γ_w = specific weight of water (lb/ft^3),
- φ = bank angle (degrees), see Figure 6.12 and
- d_{50} = median rock size, also defined as the diameter where 50% is finer by weight (ft).

This general equation can be simplified under various conditions: (1) channel banks on straight reaches, (2) channel banks on curve reaches, (3) channel bed on straight reaches, (4) channel bed on curve reaches, (5) downstream of grade control/drop structures, downstream of stilling basins, spur disk/guide bank/abutments, sloped drop structures and rock chutes. Simplified equations are presented on the following pages.

FIGURE 6.12
DEFINITION FOR BANK ANGLE



Channel Banks on Straight Reach

The FCDMC-recommended Isbash equation can be simplified with C = 1.2 for bank protection on a straight reach or a mildly curved reach (a reach with a bend angle, β, ≤ 30°). The loose riprap d_{50} for bank protection in a straight channel reach or a mildly curved reach with a bend angle (β) ≤ 30° can be calculated with the following equation. The equation has the form:

$$d_{50} = \frac{0.0191 V_a^2}{\cos\varphi}\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right) \tag{6.34}$$

where:

d_{50}	=	the median diameter (ft),
V_a	=	average velocity (ft/s),
γ_s	=	specific weigh of stone (lb/ft^3),
γ_w	=	specific weight of water (lb/ft^3),
φ	=	bank angle (degrees), see Figure 6.12 and
β	=	channel bend angle (degrees).

Channel Banks on Curved Reach

Since the Isbash equation does not account for the increased erosion in a bend, the FCDMC recommends the use of the high turbulent C coefficient of 0.86 when there is a bend. The loose riprap d_{50} for the outer bank of a curved reach with a bend angle, β, greater than 30 degrees is:

$$d_{50} = \frac{0.0372 V_a^2}{\cos\varphi}\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right) \tag{6.35}$$

where:

d_{50} = the median diameter (ft),

V_a = average velocity (ft/s),

γ_s = specific weigh of stone (lb/ft^3),

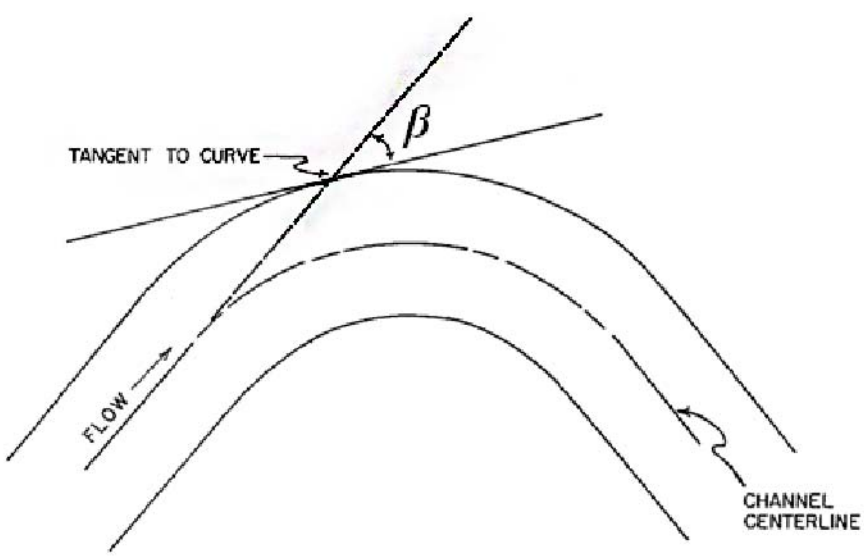
γ_w = specific weight of water (lb/ft^3),

φ = bank angle (degrees), see Figure 6.12, and

β = bend angle, see Figure 6.13, (degrees).

The inner bend riprap sizing can be based on the straight reach equation.

FIGURE 6.13
DEFINITION FOR CHANNEL BEND ANGLE
(FIGURE ADAPTED FROM Simons, Li and Associates, 1989B)



Channel Bed on Straight Reach

The loose riprap d_{50} for a straight channel reach or a mildly curved reach (bend angle, $\beta \leq 30^{o}$) is:

$$d_{50} = 0.0191 V_a^2 \left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right) \tag{6.36}$$

where:

d_{50} = the median diameter (ft),

V_a = average velocity (ft/s),

γ_s = specific weigh of stone (lb/ft^3), and

γ_w = specific weight of water (lb/ft^3).

This equation is also a simplified Isbash equation with C = 1.2 and 0.0 degrees of bank angle.

Channel Bed on Curved Reach

The loose riprap d_{50} for channel bed protection near the outer bank of a curved reach with more than a 30° bend angle is:

$$d_{50} = 0.0372 V_a^2 \left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right) \tag{6.37}$$

where:

d_{50} = the median diameter (ft),

V_a = average velocity (ft/s),

γ_s = specific weigh of stone (lb/ft^3), and

γ_w = specific weight of water (lb/ft^3).

This equation is also a simplified Isbash equation with C = 0.86 and 0.0 degrees of bank angle.

Downstream of Grade Control/Drop Structure

The loose riprap d_{50} for channel bed protection downstream of a grade control or a drop structure is:

$$d_{50} = 0.0372 V_a^2 \left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right) \tag{6.38}$$

where:

d_{50} = is the median diameter (ft),

V_a = average velocity (ft/s),

γ_s = specific weigh of stone (lb/ft^3), and

γ_w = specific weight of water (lb/ft^3).

This equation is also a simplified Isbash equation with C = 0.86 and 0.0 degrees of bank angle.

Downstream of Stilling Basin

The loose riprap d_{50} for riprap downstream of a stilling basin can be from Figure 165 in Peterka (1978) or can be found by the following equivalent equation (Berry, 1948):

$$d_{50} = 0.0126 V_a^2 \tag{6.39}$$

where:

d_{50} = is the median diameter (ft), and

V_a = average velocity (ft/s).

Spur Dike/Guide Bank/Abutment

The loose riprap d_{50} for spur dike, abutment, and guide bank (Simons, Li and Associates, 1989a) is:

$$d_{50} = 0.01 V_a^{2.44} \tag{6.40}$$

where:

d_{50} = the median diameter (ft), and

V_a = average velocity (ft/s).

Sloped Drop Structure/Rock Chute

The loose angular riprap d_{50} equations for the sloped drop structure or rock chute at different slope ranges have been developed by Robinson, et al. (1998). As indicated by Robinson, et al. (1998), an appropriate safety factor should be applied when using these equations. With a safety factor of 1.5, the loose angular riprap median size equations are simplified from Robinson, et al. (1998) as:

$$d_{50} = 2.12q^{0.529}S_0^{0.794} \qquad 0.02<S_0<0.10 \tag{6.41}$$

$$d_{50} = 0.69q^{0.529}S_0^{0.307} \qquad 0.1\le S_0\le 0.4 \tag{6.42}$$

where:

d_{50} = the median diameter (ft),

q = unit discharge (cfs/ft), (discharge divided by the width where the width is defined as the wetted area divided by the flow depth and the flow depth can be the Manning's equation-based normal depth or the maximum flow depth from HEC-RAS), and

S_0 = channel bed slope (ft/ft).

It should be noted that these two equations are for loose riprap on the slope of the structure. For downstream of the sloped drop structure, the equation above for Downstream of Grade Control/ Drop Structure should be used. The thickness for the riprap layer on the slope should be at least $2d_{50}$. A granular filter should be used beneath the riprap layer. The design for rock chutes and downstream energy dissipators can be found in Lorenz et al. (2000).

Concrete Rubble

Concrete rubble or broken concrete is a very economical riprap material. However, the successful use of such material requires good quality control on shape, specific weight, gradation, and durability (USDOT, 1989). Due to the difficulties of achieving good quality of these aspects, extra caution must be exercised. Careful inspection of the material must be performed to ensure the quality of the material meets the specified requirements. In addition, aesthetics should be considered. Agency approval is required before the use of concrete rubble material. The following specifications should be followed for concrete rubble design. The specifications have been developed by FCDMC based on Florida Department of Transportation (2010), Missouri Department of Natural Resources (2009), and Montana Department of Environmental Quality (2011).

- The specific gravity shall be at least 2.3 or the specific weight shall be at least 143.5 lb per cubic foot. The specific gravity shall be shown on the construction plan.
- Materials shall be free of grease, oils, paint, chemicals, and other pollutants.
- All protruding foreign material such as rebar must be cut off.

- The longest dimension shall not be more than three times the shortest dimension. The length shall not be greater than twice its width.
- Materials, when trucked or imported to the site, must be sorted at the site.
- Materials shall be stockpiled before placement to allow for inspection. Visual inspection shall ensure that materials are free of cracks, soft seams or other structural defects.
- Materials shall be hard, durable, rough and angular in shape. Additional tests may be required subject to the inspector's judgment. Examples of such tests include the Los Angeles Abrasion Test (AASHTO T96), Durability and Soundness Test (AASHTO T104), Absorption Test (AASHTO T85), Drop Test (USACE, 1990, EM 1110-2-2302). The requirements for these tests are a maximum loss of 45% for the Los Angeles Abrasion Test, a maximum loss of 12% for the Durability and Soundness Test, and a maximum of 5% for the Absorption Test. The Drop Test requirements are: no new cracks developed, or no existing crack widened more than an additional 0.1 inch, or final largest dimension greater than or equal to 90% of the original largest dimension of a dropped piece.
- The maximum weight of any piece shall not exceed 500 lbs.
- Materials shall be reasonably well graded. The gradation shall be based on general riprap gradation documented in Table 6.4. All large slabs shall be broken up to conform to the gradation requirement.
- Either a granular or fabric filter blanket is required for loose rubble concrete riprap. The specification for filter blankets can be found in the Riprap Layer Characteristics Section, Filter Blanket Requirements.
- The thickness of rubble concrete riprap for river bank protection shall be based on the Riprap Layer Characteristics Section, Thickness.
- The maximum allowable bank slope is 3:1 (H:V).
- All material shall be placed in a manner such that the large and small sizes are evenly distributed and placed so as to fill the voids between the larger pieces without sharp exposed edges.
- All material shall be placed in a manner such that each piece is touching the adjacent piece in a configuration creating the highest possible density while producing a reasonably solid mass within the limits shown in the plans.
- The largest material must be keyed into the toe and also used in the base of the riprap.

6.6.4 Bank Toe Protection

Toe protection failures result when the foundation of the bank protection measure is undermined by scour at the toe resulting from local scour and/or general channel bed degradation. Proper design of protection from toe scour involves two parameters. First, an estimate of the total scour depth must be made. Second, a means of protection must be provided for the total scour. The first parameter, total scour depth estimation, requires specialized analysis techniques by a qualified engineer. The procedure for estimating the total scour depth can be found in Chapter 11. Mitigation measures for providing protection for the total scour are presented in this section.

The two methods of providing toe protection in erodible channels are:

1. To extend protection to the total scour depth; and
2. To provide protection that adjusts to the scour as it occurs.

The first method is the preferred technique because the protection is initially placed to a known depth and the designer does not have to depend on uncertainties associated with the method that adjusts to the scour. This method requires extension of the bank protection into the excavated channel bed and is primarily used for placement in dry conditions because of the expense and uncertainties of deep excavation that can frequently encounter groundwater.

The second method is also called the launchable riprap toe protection method. The main advantage of the second method is the elimination of relatively deep excavation and related water control. The most frequently used material for providing adjustable toe protection is riprap placed at the toe of the bank in a weighted riprap configuration. The riprap moves downslope, as scour occurs, to form a protective cover. Figure 6.14 shows the desirable configuration for a weighted riprap toe. Studies by Linder (1976) and the U.S. Army Corps of Engineers (1981 and 1994) on riprap toe protection arrived at the following conclusions:

1. Volume of rock in the weighted riprap toe is probably the most significant factor in determining the success of the weighted riprap toe.
2. Toe shape has a definite influence on performance. Thin toes do not release rock fast enough, which results in poor slope coverage. Thick toes release rock at a greater rate than is needed. The thickness of the recommended toe ranges from two to three times the thickness of the riprap bank protection. The recommended toe shape is shown in Figure 6.14.
3. Complex toe designs that are difficult to construct are not necessary.
4. Downslope rock movement occurred without significant movement in the downstream direction.
5. Results from modeling and the subsequent prototypes show that the recommended

weighted toe designs launch at a slope slightly steeper than 2:1.

6. In theory, toe volume in the physical model is equal to the volume needed to extend the bank protection to the total scour depth at a 2:1 slope. However, because of the loss of rock during the launching process as scour takes place, the required rock volume for the toe protection should be more than the volume of extending the bank protection to the total scour depth (USACE, 1994). If the original volume is Vol, then the increased volume should be $(1+C_{VI}/100)*Vol$ where C_{VI} is volume increase coefficient in percent. Table 6.7 lists volume increase coefficient values for both dry placement and underwater placement at two different vertical launch distances (H_V). The vertical launch distance is defined as the vertical distance between the bottom of the toe protection and total scour depth below the thalweg. Figure 6.15 illustrates the vertical launch distance. The required toe length L can be computed by

$$L = 1.5H + \left(1 + \frac{C_{VI}}{100}\right) T\sqrt{5}\frac{H_v}{H} \tag{6.43}$$

where H is the toe thickness $2T < H \leq 3T$, C_{VI} is the volume increase in percent from Table 6.8, T is the riprap layer thickness $T = 1.5d_{50}$, and H_V is the vertical launch distance = $EL_{TOP} - H - EL_{TG} + Z_T$ where EL_{TOP} is the top elevation for toe protection, EL_{TG} is the thalweg elevation, and Z_T is the total scour depth. The total scour depth estimation procedure can be found in Chapter 11.

The launchable toe protection method has been widely used on sand bed streams for applications such as windrow revetments (riprap placed at top of bank), trench-fill revetments (riprap placed at low water level), and weighted riprap toes (riprap placed at intersection of channel bottom and side slope). However, this method for gravel bed streams is not as widely accepted as in sand bed streams (USACE, 1994). For gravel-bed or cobble-bed streams, extra caution should be exercised because more rocks may be lost during the launching process due to the impinging force caused by the moving cobbles as part of stream bed load. For gravel-bed streams with large cobbles, the rock size (d_{50}) or rock volume at the toe protection section may be increased by 25%. A filter must be installed beneath the rock at the bank slope and the toe protection section as shown in Figure 6.14. A granular filter must be installed beneath the toe protection section because a fabric filter may affect the launching process.

TABLE 6.7
VOLUME INCREASE COEFFICIENT IN PERCENT FOR SAND-BED CHANNELS
(ADAPTED FROM USACE, 1994)

Vertical Launch Distance (H_V)	Volume Increase in Percent (C_{VI})	
	Dry Placement	**Underwater Placement**
≤15 ft	25	50
>15 ft	50	75

TABLE 6.8
VOLUME INCREASE COEFFICIENT IN % FOR GRAVEL-BED CHANNELS
WITH LARGE COBBLES (ADAPTED FROM USACE (1994))

Vertical Launch Distance (H_V)	Volume Increase in Percent (C_{VI})	
	Dry Placement	**Underwater Placement**
≤15 ft	50	75
>15 ft	75	100

Weighted riprap toes have been used successfully for many years. However, success has not been universal. A common factor among the failures appears to be the presence of impinged flow on the bank. Therefore, the guidelines herein apply chiefly to flow conditions parallel to the bank. Where impinged flow is likely, then analyses must be made to determine an appropriate additional level of protection for this type of hydraulic condition.

FIGURE 6.14
TOE PROTECTION CHANNEL LINING
(Wright-McLaughlin, 1969)

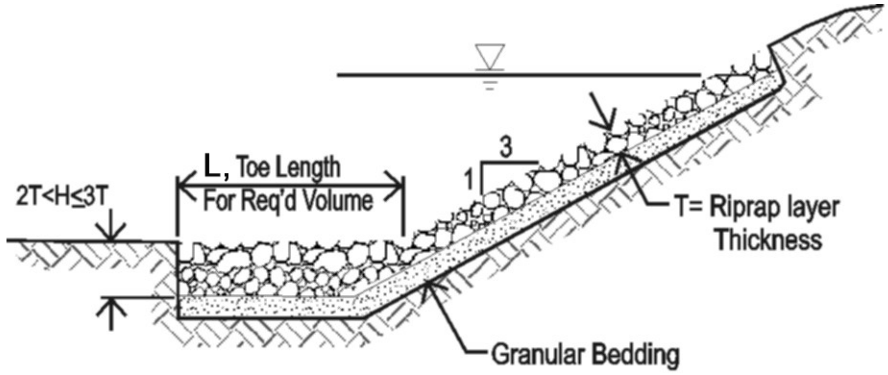
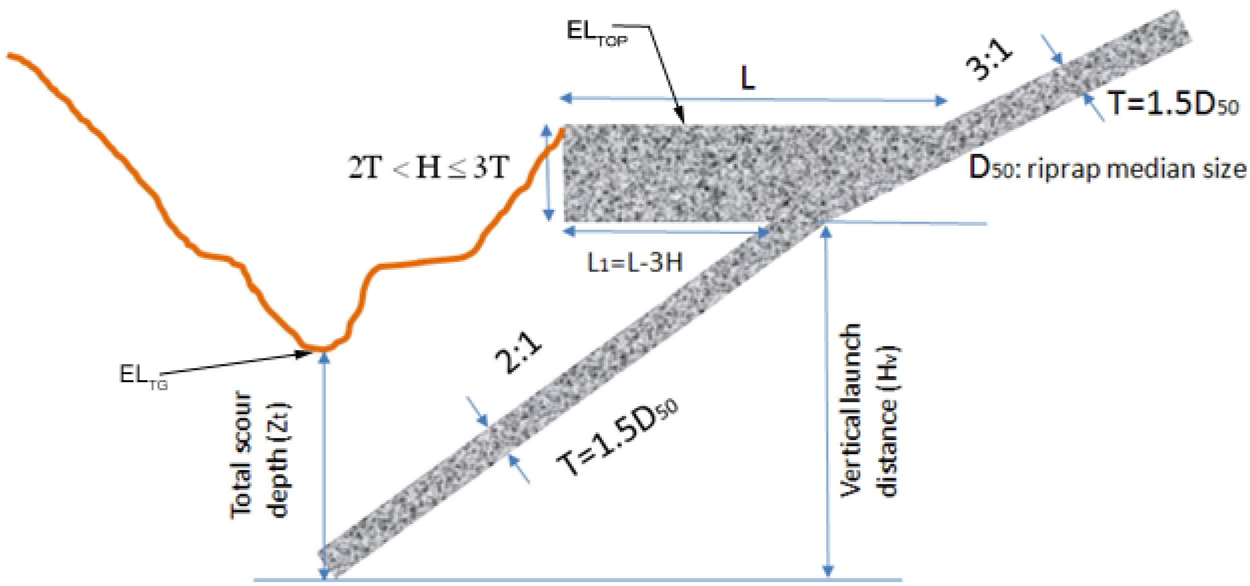


FIGURE 6.15
TOE PROTECTION CHANNEL LINING DETAILS



6.6.5 Grouted Riprap Lined Channels

General - This section is developed based on USACE (1992) and HEC-11 (USDOT, 1989). Grouted riprap may be an economical alternative to the conventional loose riprap approach where the required stone size cannot be economically procured (USACE, 1992). In areas where transportation costs are a significant portion of the construction cost of the riprap treatment, it may be less expensive to use grouted riprap. Typical applications include protection of bed and bank slopes in spillway entrance channels, zones of turbulence adjacent to energy dissipaters, drainage ditch linings, culvert and storm sewer outfalls, and open channels. Grouted riprap is a structural lining comprised of a blanket of rock that is interlocked and bound together by means of concrete grout injected into the void spaces to form a monolithic revetment. Grouted rock provides a stable lining similar to concrete with the added advantage of a higher roughness factor due to the rock surfaces projecting above the grout layer. However, it is a rigid revetment that does not conform to changes in bank geometry due to settlement, and is susceptible to failure from undermining and the subsequent loss of the supporting bank material.

Several limitations of grouted riprap are summarized in USACE (1992).

1. Grouted riprap must be used only on properly designed slopes. The additional expense of grouting riprap cannot be justified without providing proper slope stability. Furthermore, grouted riprap placed on a poorly designed slope can have the detrimental effect of masking progressive slope failure until it has advanced far enough to cause failure of the riprap treatment.

2. It must be recognized that grouted riprap will crack, cracking will be irregular, and cracks will likely extend within the grout matrix and around the periphery of larger stones. Cracking may cause enhanced weathering, including aggressive chemical reactions, but should not significantly diminish the effectiveness of the treatment. If the sub-base is properly designed and constructed to provide adequate drainage without loss of sub-base materials through cracks. Grouted riprap should not be used in areas where frost heave or ice in the sub-base can be expected to cause uplift failure.

3. River-side slopes of levees should not be protected with grouted riprap. At first, it may appear that a reduction in construction cost might be realized if grouted riprap could be provided for levee protection. However, levees undergo significant settlement that cannot be accommodated by the rigid nature of grouted riprap.

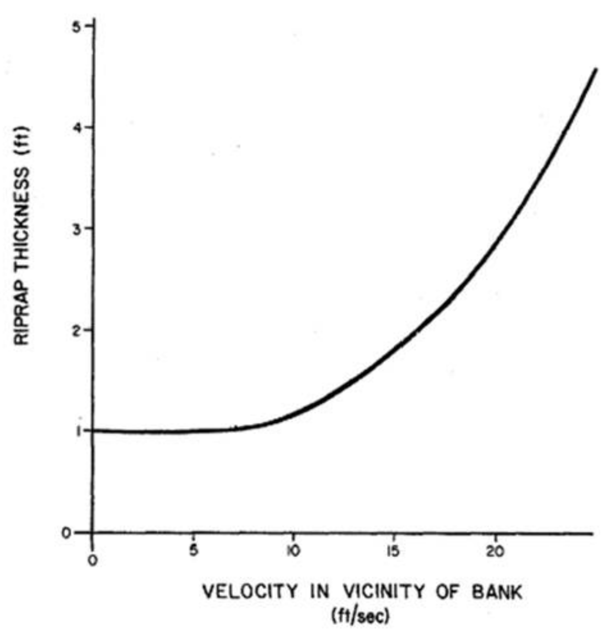
4. Applying grout to salvage a failing conventional riprap treatment without proper design to address the cause of the failure should not be undertaken. This practice most often does not provide a successful repair and results in a waste of resources. Examples are slope failures resulting from upslope surface runoff, piping-related internal erosion, down-slope

riprap failure resulting from toe scour, and failures of frequently overtopped drainways and drainage ditches.

Materials - Riprap quality should conform to the property requirements described in Section 6.6.3. Figure 6.16 gives the required blanket thickness of grouted rock for a given design channel velocity. The median rock size d_{50} shall not exceed 0.67 times the blanket thickness and the largest rock used should not exceed the blanket thickness. d_{50} and d_{100} can be 0.67 and 1.0 times the blanket thickness, respectively. It is required that rocks smaller than the d_{50} size be removed. The other gradation limits should conform to those described in Section 6.6.3.

The grout mix should be specified to provide the strength and durability required to meet the specific application. The minimum 28-day compressive strength shall be 2,000 psi and the slump shall be within a range of 4 to 7 inches. The stone aggregate should conform to the gradation requirements of Size Number 8 coarse aggregate (3/8 inch to No. 8) as specified in ASTM C-33. A maximum of 30 percent of the cementitious material may be fly ash (ASTM C-618, Type C or F). Fiber reinforcement is recommended to be added to the grout to provide additional control of shrinkage and cracking.

FIGURE 6.16
REQUIRED BLANKET THICKNESS OF GROUTED ROCK
(USDOT, 1989)



Design Considerations for Grouted Riprap Channels

- Riprap smaller than the d_{50} shall be removed.

- The grout shall extend the full thickness of the rock blanket.

- Finished grout should leave face stones exposed for one-fourth to one-third their depth.

- Bank slopes for grouted riprap shall not exceed 1.5:1 (H:V)

- The grouted riprap protection for open channels must extend below the channel thalweg to the depth of total scour. The total scour depth estimation method can be found in Chapter 11.

- Bank should be prepared by first clearing all trees and debris from the bank, and grading the bank surface to the desired slope. The bank surface should be tamped or lightly compacted. Care must be taken during bank compaction to maintain a soil permeability similar to that of the natural, undisturbed bank material. The foundation for the grouted riprap revetment should have a bearing capacity sufficient to support either the dry weight of the revetment alone or the submerged weight of the revetment plus the weight of the water in the wedge above the revetment for design conditions, whichever is greater.

- Filters are required under all grouted riprap revetments to provide a zone of high permeability to carry off seepage water and prevent damage to the overlying structure from uplift pressures. A 6-in (15.4-cm) granular filter is required beneath the grouted riprap to provide an adequate drainage zone. The filter can consist of well-graded granular material, or uniformly-graded granular material with an underlying filter fabric. The filter should be designed to provide a high degree of permeability while preventing base material particles from penetrating the filter.

- Weep holes should be provided in the revetment to relieve hydrostatic pressure build-up behind the grout surface. Weeps should extend through the grout surface to the interface with the gravel underdrain layer. Weeps should consist of 3-in (7.6-cm) diameter pipes having a maximum horizontal spacing of 6-ft (1.8 m) and a maximum vertical spacing of 10 feet (3.0 m). The buried end of the weep should be covered with wire screening of an appropriate gage or a fabric filter that will prevent passage of the gravel underlayer.

- The edges of grouted rock revetments (the head, toe, and flanks) require special treatment to prevent undermining. The revetment toe should extend to a depth below anticipated channel bed total scour depths or to bedrock. Grouted riprap should extend from below the anticipated channel bed total scour depth to the design high water level plus additional height for freeboard. The total scour depth procedures can be found in Chapter 11. The toe should be designed as illustrated in Figure 6.17(a). After excavating to the desired depth, the riprap slope protection should be extended to the bottom of the trench and grouted. The remainder of the excavated area in the toe trench should be filled with ungrouted riprap. The ungrouted riprap provides extra protection against undermining at the bank toe. The riprap sizing for the ungrouted riprap should be based on the loose angular riprap sizing equation, Equation (6.33).

- To prevent outflanking of the revetment, the upstream and downstream edge of the grouted riprap should be designed in accordance with Figure 6.17(b) and Figure 6.17(c). Figure 6.18 shows three typical design sections for bank and channel revetments. Section A-A is a mid-section. Sections B-B and C-C are flank sections documenting the upstream and downstream edge details respectively.

FIGURE 6.17
GROUTED RIPRAP SECTIONS: (a) SECTION A-A; (b) SECTION B-B; AND (c) SECTION C-C
(REFER TO Figure 6.18 FOR SECTION LOCATIONS).

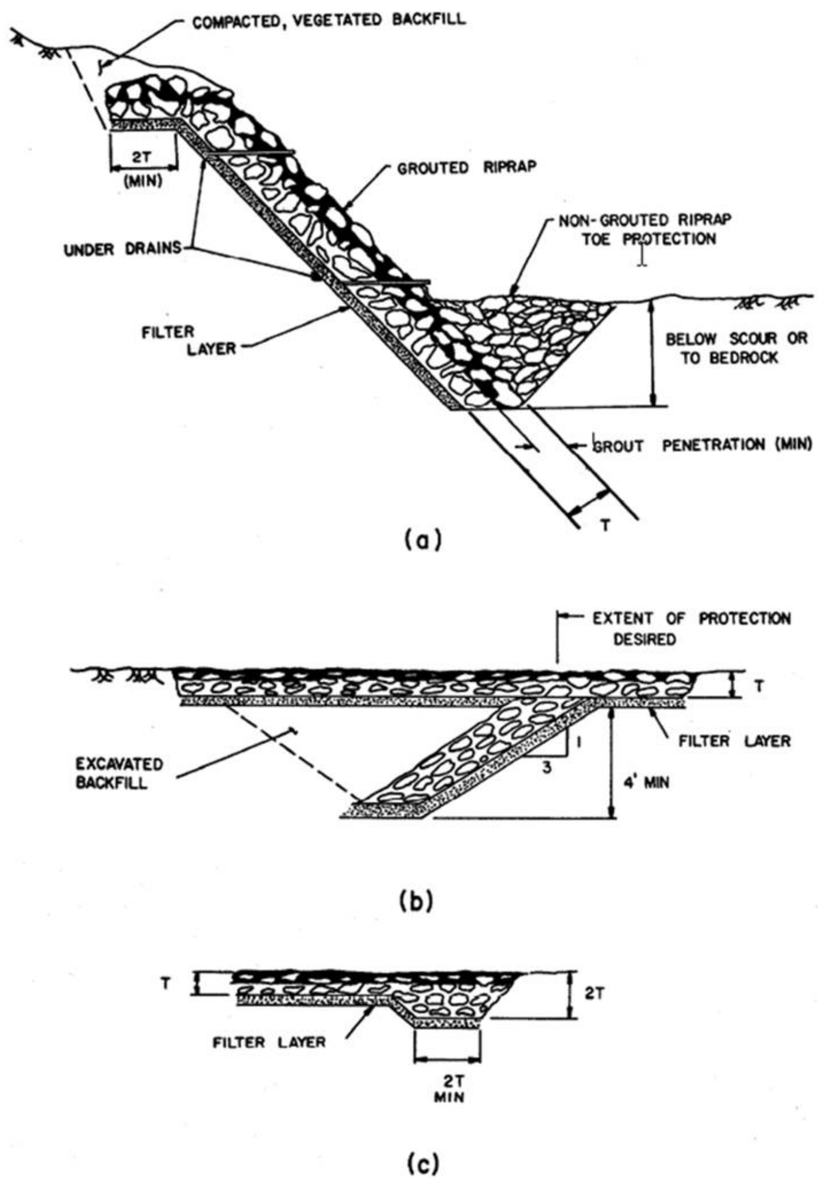
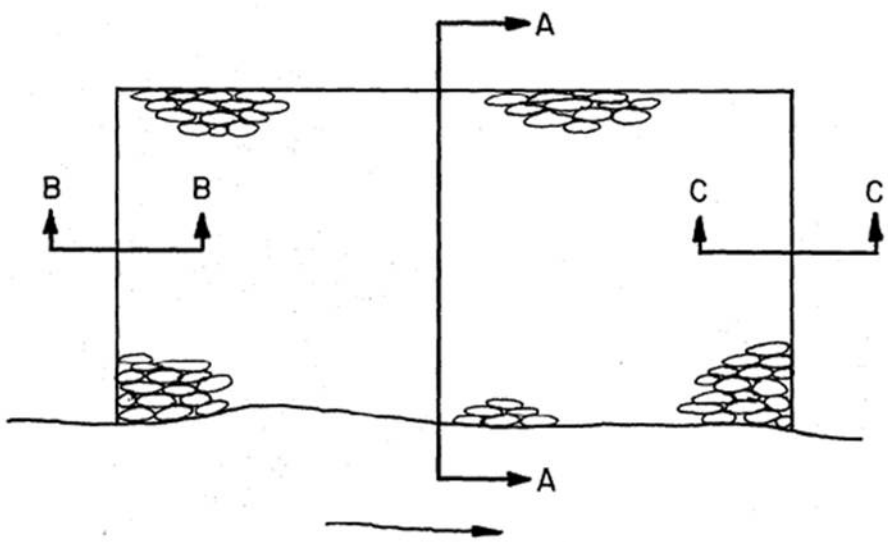


FIGURE 6.18
GROUTED RIPRAP SCHEMATIC



6.6.6 Gabion Lined Channels

Gabions refers to rocks that are confined by a wire basket so that they act as a single unit. The wire mesh enclosed rock units are also known as gabion baskets or gabion mattresses. One of the major advantages of wire enclosed rock is that it provides an alternative in situations where available rock sizes are too small for common riprap. Another advantage is the versatility that results from the regular geometric shapes of wire enclosed rock. The rectangular blocks and mats can be fashioned into almost any shape that can be formed with concrete. The durability of wire enclosed rock is generally limited by the service life of the galvanized binding wire. Under normal conditions here in the arid southwest the service life is considered to be about 25 years, based on the experience of the FCDMC's Operations and Maintenance Division. The service life has the potential of being much shorter, in the range of 5 to 15 years for a variety of reasons including prolonged exposure to water and improper design (derived from Racin and Hoover, 2001). Water carrying silt, sand or gravel can reduce the service life of the wire. Also, water that rolls or otherwise moves cobbles and large stones breaks the wire with a hammer and anvil action and considerably shortens the life of the wire. Gabions should not be used for rivers with large cobbles and stones as part of bed load. The wire has been found to be susceptible to corrosion by various chemical agents and is particularly affected by high sulfate soils. If corrosive agents are known to be in the water or soil, a plastic coated wire should be specified. The designer should verify site specific conditions and coordinate with a qualified manufacturer to properly specify gabion wire. See ASTM A-974 and ASTM A-975.

Gabions are not maintenance free and must be periodically inspected to determine whether the wire is sound. If breaks are found while they are still relatively small, they may be patched by weaving new strands of wire into the wire cage. Wire enclosed rock installations have been found to attract vandalism. Flat mattress surfaces seem to be particularly susceptible to having wires cut and stones removed. It is recommended that, where possible, mattress surfaces be buried, where they are less prone to vandalism. Wire enclosed rock installations should be inspected at least once a year under the best circumstances and may require inspection every three months in vandalism prone areas in conjunction with a regular maintenance program. They should also be inspected after high flow events. Under high flow velocity conditions, mattresses on sloping surfaces must be securely anchored to the surface of the soil as discussed previously.

Materials

Rock and Wire Enclosure Requirements - Rock filler for the wire baskets should meet the rock property requirements for common riprap. Rock sizes and basket characteristics should meet ASTM A-974 and ASTM A-975. The minimum rock size d_0 should be equal to the size of the gabion mesh opening. The maximum rock size d_{100} should be less than the gabion thickness.

Bedding Requirements - Long term stability of gabion (and common riprap) erosion protection is strongly influenced by proper bedding conditions. A large percentage of all riprap failures are directly attributable to bedding failures, which is particularly disturbing in light of the fact that over half of all riprap installations experience some degree of failure within 10 years of construction. Refer to Section 6.6.3 for gravel bedding or filter design. Nonwoven, 8 ounce filter fabric has been found acceptable in many applications. The design engineer should check with the manufacturer for its given application.

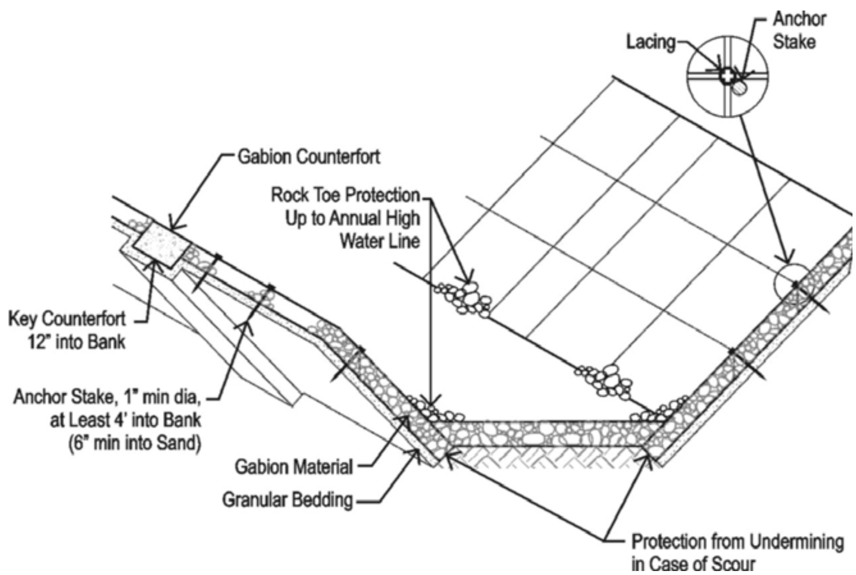
Design Considerations for Gabion Lined Channels

The geometric properties of gabions permit placement in areas where common riprap is either difficult or impractical to place. Proper design and construction is important to successful operation and lifetime performance. Twisted wire mesh has been found to be more tolerant to settlement than welded wire mesh (See ASTM A-975).

Slope Mattress Lining - Figure 6.19 shows a typical configuration for a gabion slope mattress channel lining. The long side of the gabion basket should be aligned parallel with the channel for applications on banks steeper than 2:1. Channel linings should be tied to the channel banks with gabion counterforts (thickened gabion sections that extend into the channel bank) at the upstream edge of the lining. Counterfort spacing shall be per manufacturer's recommendations.

Mattresses and flat gabions on channel side slopes need to be tied to the banks. The ties should be metal stakes no less than 4 feet in length (sandy soils warrant longer lengths). These should be located at the inside corners of basket diaphragms along an upslope (highest) basket wall, so that the metal stakes are an integral part of the basket. The exact spacing of the stakes depends upon the configuration of the baskets, however the following is the suggested minimum spacing: stake every 6 feet along and down the slope for 2:1 slopes or steeper. Channel linings should be tied to the channel banks with gabion counterforts (thickened gabion sections that extend into the channel bank) at the upstream edge of the lining. For most applications, mattresses should be a minimum of 9 inches thick.

FIGURE 6.19
SLOPE MATTRESS LINING
(Wright-McLaughlin Engineers, 1969)



6.6.7 Design Documentation Requirements for Major Watercourses

The following guidelines should be used for all watercourses subject to submittal for FCDMC and FEMA review. These are primarily for watercourses with flows in excess of 2,000 cfs.

<u>Open Channel Hydraulics</u>

HEC-RAS or HEC-2 shall be used to perform water surface profile calculations. Alternative methods require approval. A hard copy and floppy disk/CD-ROM with input and output files shall be submitted for FCDMC review. The HEC input and output files shall be prepared in a format suitable for submittal to FEMA, using *Requirements For Flood Study Technical Documentation*, ADWR 1997.

The starting water surface elevations for profile computations for mainstreams and tributaries should be based on FEMA requirements (FEMA, 2003). In general, the starting water surface elevations chosen for profile computations should be based on normal depth (or slope-area), unless known water surface elevations are available from other sources. When using normal depth on the main stream, the model should be started several cross sections downstream of the beginning of the study reach. For starting conditions on tributaries, normal depth should be used unless a coincident peak situation is assumed, or the tributary flow depths are higher than the corresponding main stream events. The assumption of coincident peaks may be appropriate if a) the ratio of the drainage areas lies between 0.6 and 1.4, b) the times of peak flows are similar for the two combining watersheds, and c) the likelihood of both watersheds being covered by the storm being modeled are high. If gage records are available for the basin, guidance for coincidence of peak flows should be taken from them.

The Consultant shall estimate blockage due to debris at bridge piers based on field conditions. As a minimum, use the greater of 2 times the diameter of the pier or 1 foot on each side of the pier.

Freeboard for levees shall, as a minimum, comply with FEMA freeboard criteria: 3 feet of freeboard at the 100-year peak stage plus 1 foot additional at bridges (FEMA, 2003). Refer to the local jurisdiction *Policies and Standards Manual* for possible more stringent conditions.

Locations of cross sections used in the water surface profile calculations shall be provided on a scaled map and also in a tabular format. The cross section labels on the maps shall reflect cross sections in the models (ADWR, 1997).

Channel Stabilization Design

Channel stability based on permissible velocity shall only be used for preliminary design purposes. The tractive shear stress approach shall be used to confirm unlined channel stability.

Provide calculations to show that the type of bank protection (common riprap, gabions, concrete, etc.) is suitably sized to resist hydraulic forces (tractive shear, impingement, buoyancy, etc.) at the design frequency peak flow.

Appropriate hydraulics and structural calculations should be provided for review. Refer to the local jurisdiction's *Policies and Standards Manual* for requirements.

Consideration shall be given to how the upstream and downstream floodplain conditions will impact the proposed channel. The effects of existing and potential mining and fill operations shall be addressed. Overbank flooding upstream of the channelization shall be analyzed to demonstrate that design flows enter and are contained within the improved channelization. The design and analysis shall address the potential impacts of future modifications proposed by others. Gradual transition of the existing floodplain/floodway upstream and downstream of the channelization is required.

The minimum factor of safety applied to hydraulic forces on structural components shall be 1.5, based on the 100-year frequency peak flow.

The analysis shall address sediment transport, scour, lateral migration, and river mechanics as discussed in Chapter 11.

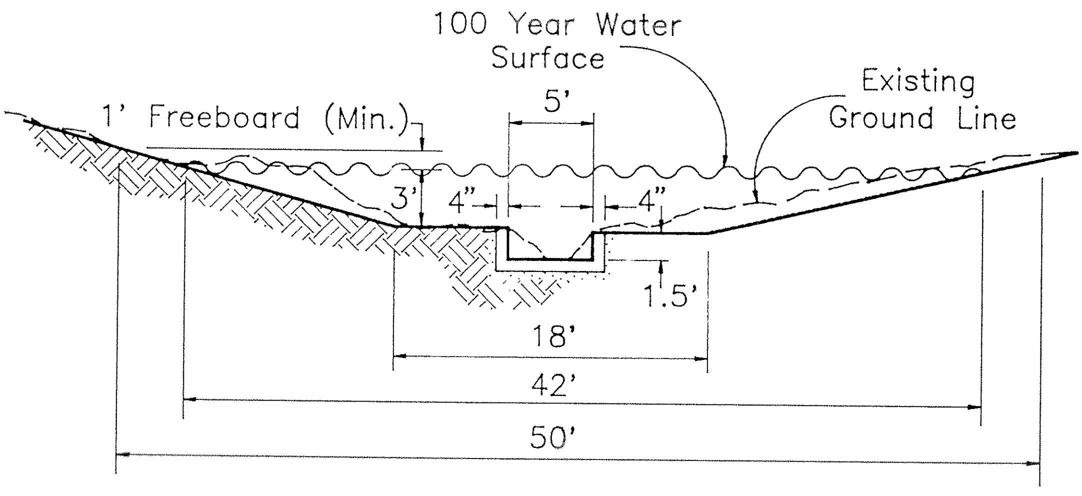
Plans submitted for review shall include profiles showing the top of levee protection, toe ground elevation, toe-down, hydraulic grade line, water surface elevation, existing and design invert elevations at the thalweg, and the low chord elevations for bridges. Also, road and railway crossing locations must be shown on plans and profiles.

6.6.8 Design Example

Problem Description

Improve a small unlined channel by adding an incised low flow channel and providing bank and edge erosion protection. Refer to Figure 6.20. The natural channel is characterized by S_o = 0.006 ft/ft and partially vegetated sandy silt material. The improved channel is to be designed to convey the 100-year design flow, Q_{100} = 565 cfs.

FIGURE 6.20
DESIGN EXAMPLE CHANNEL TYPICAL CROSS SECTION



Requirements

- Use available right-of-way width of 50 feet, with approximately 3 feet of depth above the low flow channel.

- Provide a grass-lined main channel with concrete-lined low flow.
- Use 4:1 side slopes and provide a minimum 1 ft freeboard allowance.

Solution

A. Use Equation (6.22) to determine channel capacity.

1. Find cross sectional area of flow, [Total area (A_T) = area of low flow channel (A_{lf}) + area of main channel (A_{mc})].

$$A_T = A_{lf} + A_{mc}$$

$$= (1.5)(5) + 3((18 + 42)/2)$$

$$= 7.5 + 90$$

$$A_T = 97.5 \text{ sq ft}$$

2. Find wetted perimeter and indicate a 4 inch thickness for low flow wall.

$$P_T = P_{lf} + P_{mc}$$

$$= [2(0.33) + 2(1.5) + 5] + [(18 - 5.67) + 2(3)(1^2 + 4^2)^{0.5}]$$

$$P_T = 45.7ft$$

3. Find hydraulic radius.

$$R = A_T / P_T$$

$$= 97.5/45.7$$

$$R = 2.13\ ft$$

4. Determine Manning's n from Table 7.6.

Find composite n-value:

Concrete lined low flow: $n = 0.015$

Grass-lined main channel: $n = 0.025$

$$n = \left[\Sigma_i \frac{P_i n_i^{1.5}}{P_T}\right]^{0.67}$$

$$n = \left[\frac{(P_{lf})(n_{lf}^{1.5}) + (P_{mc})(n_{mc}^{1.5})}{P_T}\right]^{0.67}$$

$$n = \left[\frac{8.67(0.015)^{1.5} + 37.0(0.025)^{1.5}}{45.7}\right]^{0.67}$$

$$n = \left[\frac{(0.016 + 0.15)}{45.7}\right]^{0.67}$$

$$n = 0.023$$

5. Substitute values to solve for slope and multiply Equation (6.22) by A_T and rearrange:

$$Q = (1.49/n)A_T R^{0.67} S_o^{\ 0.5}$$

$$S_o = [(Qn)/(1.49 A_T R^{0.67})]^2$$

$$= [565(0.023)/((1.49)(97.5)(2.13)^{0.67})]^2$$

$$S_o = 0.0029\ ft/ft$$

Since a channel bottom slope of $0.0029\ ft/ft$ is sufficient to convey the design flow of $565\ cfs$, the steeper existing S_o of $0.006\ ft/ft$ will convey the flow with a smaller cross sectional area. Equation (6.22) can be solved for $A_T R^{0.67}$, which can then be used to determine the actual cross section of flow by trial and error:

$$A_T R^{0.67} = Qn/1.49 S_o^{\ 0.5}$$

$$= 565(0.023)/(1.49(0.006)^{0.5})$$

$$= 112.6\ ft^3$$

By trial and error,

$$y_n = 2.45\,ft$$

$$A_T = 7.5 + 2.45((18 + 37.6)/2) = 75.6\,ft^2$$

$$P_T = 8.67 + ((18 - 5.67) + 2(2.45)(1^2 + 4^2)^{0.5}) = 41.2\,ft$$

$$R = A_T/P_T = 75.6/41.2 = 1.83\,ft$$

$A_T R^{0.67} = (75.6)(1.83)^{0.67} = 113.3 \approx 112.6\,ft^3$ therefore OK.

Flow along the channel at $S_o = 0.006\,ft/ft$ has reduced the water depth by $0.55\,ft$. Note that the composite *n-value* was not revised using the new values of P_{lf}, P_{mc} and P_T.

B. Check velocity and Froude Number.

1. Check velocity.

$$V = Q/A$$

$$= 565/75.6$$

$$V = 7.5\,fps > 6\,fps$$

6 ft/sec allowable for Bermuda grass lined channels with erosion resistant soil only.

2. Check the Froude number.

$$F_r = V/(gd)^{0.5}$$

$$= 7.5/[(32.2)(75.6)/(37.6)]^{0.5}$$

$$F_r = 0.93 > 0.86$$

The channel is just under critical flow conditions and will not be stable at a bottom channel slope of $0.006\,ft/ft$ for the design flow. One solution is to provide grade control structures to maintain $S_o = 0.0029\ ft/ft$, thereby providing:

$$V = Q/A = 565/97.5 = 5.8\ ft/\sec \quad \text{and}$$

$$F_r = V/(gd)^{0.5} = 5.8/((32.2)(97.5)/42)^{0.5} = 0.67$$

which is within the acceptable range of subcritical flow. See Chapter 8 for grade control structures.

3. Check channel transitions (see Chapter 8).

C. Check freeboard requirement using Equation (6.25).

Using the revised slope of $S_o = 0.0029\ ft/ft$, and velocity of $V = 5.8\ ft/\sec$.

$$FB = 0.25(y + V^2/2g)$$

$$= 0.25(3 + 5.8^2/2(32.2))$$

$$= 0.25(3.52)$$

$$FB = 0.88\ ft$$

Use 1 ft.

Summary

Use grass lined channel with 4:1 side slopes.

Velocity $= 5.8\ ft/\sec$; $F_r = 0.67$, subcritical flow. See Table 6.2 and Table 6.3 for allowable soil and grass types.

Channel slope $= 0.0029 \ll 0.006\ ft/ft$ (existing).

Provide grade control.

Provide 1 foot minimum freeboard allowance.

Check flow velocities and hydraulic properties for other flows anticipated. In particular, check for sedimentation problems that may result from smaller more frequent storms with resultant lower velocities.

It should be noted that erosion may occur along the edges of the concrete-lined low flow channel, especially at a bend. Grouted riprap and/or loose riprap should be installed along the edges to prevent scour.

6.7 REFERENCES

6.7.1 Cited in Text

Aldridge, B.N., and Garrett, J.M., 1973, Roughness Coefficients for Stream Channels in Arizona: U.S. Geological Survey Open-File Report, 87 p.

American Concrete Institute (ACI), 2005, *Guide to Shortcrete, Reported by ACI Committee 506* http://www.concrete.org/PUBS/newpubs/506.htm.

------, 1990, *State Of The Art Report On Soil Cement.* 230.1. http://www.cement.org/.

American Society for Testing and Materials (ASTM), 2007, C127 - 07 Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate

American Society of Civil Engineers (ASCE), 2006, Sedimentation Engineering, ASCE Manual 54, Sedimentation Committee of the Hydraulics Division, Edited by Vito Vanoni, New York, N.Y.

Arizona Department of Transportation, 1989, *Urban Highways Channel Lining Design Guidelines*.

Arizona Department of Water Resources, 1997, *Requirement for Flood Study Technical Documentation*. [ADWR Publications WEB Page]

Barnes, Harry H., Jr., 1967, *Roughness Characteristics of Natural Channels*, U.S. Geological Survey Water-Supply Paper, 1849.

Berry, N.K., 1948. The Start of Bedload Movement, Thesis, University of Colorado.

Brock, Richard R., 1967. "Development of Roll Waves in Open Channels," Technical Report, Report No. KH-R-16 California Institute of Technology. http://caltechkhr.library.caltech.edu/33/01/KH_R_16.pdf.

Chien, N. and Wan, Z., 1998. Mechanics of Sediment Transport, ASCE.

Chow, Ven Te, 1959, *Open Channel Hydraulics*, McGraw Hill.

Daugherty, Robert L. and Franzini, Joseph B., 1977, *Fluid Mechanics with Engineering Applications, Seventh Edition*, McGraw Hill.

Davidian, J., 1984, *Computation of Water Surface Profiles in Open Channels*, U.S. Geological Survey, Techniques of Water-Resources Investigations, Book 3, Chapter A15.

Federal Emergency Management Agency, 2012, *Guidelines and Specifications for Flood Hazard Mapping Partners*. [FEMA Flood Hazard WEB Page]

Florida Department of Transportation, 2010. "Specifications Section 530 Riprap," ftp://ftp.dot.state.fl.us/LTS/CO/Specifications/SpecBook/2010Book/530.pdf.

Hejl, H.R., 1977, *A Method for Adjusting Values of Manning's Roughness Coefficients for Flooded Urban Areas*, Journal of Research, U.S. Geologic Survey Volume 5, Number 6, pages 541-545.

Linder, W.M., 1976, *Design and Performance of Rock Revetment Toes*, Proceedings, Third Interagency Sedimentation Conference, Denver, Colorado, pages 2-168 to 2-179.

Lorenz, E.A, Lobrecht, M.N., Robinson, K.M., 2000. "An Excel Program to Design Rock Chutes for Grade Stabilization," ASAE Annual International Meeting, Milwaukee, Wisconsin, July 9-12.

Los Angeles County Flood Control District (LACFCD), 1982. "Design Manual - Hydraulic," 2250 Alcazar Street, Los Angeles, California. http://dpw.lacounty.gov/wrd/publication/engineering/design_manual.pdf.

Maricopa Association of Governments (MAG), 2012. "Uniform Standard Specifications for Public Works Construction," 302 North 1st Avenue, Suite 300, Phoenix, Arizona 85003.

Maricopa County, 2007, *Drainage Policies and Standards for Maricopa County, Arizona*.

Missouri Department of Natural Resources, 2009. "Water Protection Program Missouri General Water Quality Certification Conditions for NWP 23. http://www.dnr.mo.gov/env/wpp/401/nwp23.pdf.

Montana Department of Environmental Quality, 2011. "Guidelines for Materials for Streambank Stabilization," http://www.deq.mt.gov/wqinfo/WaterDischarge/RIPRAP_GUIDELINES.pdf.

U.S. Natural Resource Conservation Service (NRCS), 1956, *National Engineering Handbook, Section 5 - Hydraulics*. ftp://ftp-nhq.sc.egov.usda.gov/NHQ/pub/outgoing/jbernard/CED-Directives/neh-1of2/neh05/neh-5.htm.

Peterka, A.J., 1978. Hydraulic Design of Stilling Basins and Energy Dissipators, USBR, Engineering Monograph No. 25, January.

Phillips, J.V., and Ingersoll, T.L., 1998, *Verification of Roughness Coefficients for Selected Natural and Constructed Stream Channels in Arizona*, U.S. Geological Survey Professional Paper 1584, Prepared in cooperation with the Flood Control District of Maricopa County, Phoenix, Arizona.

Portland Cement Association, 1987, *Soil-Cement for Water Control, Bank Protection Short Course*, Simons, Li and Associates, Aurora, Colorado.

Racin, J.A., and Hoover, T.P., 2001, *Gabion Mesh Corrosion, Field Study of Test Panels and Full Scale Facilities*, 2nd Edition, State of California, Department of Transportation and the U.S. Department of Transportation, Report No. FHWA-CA-TL-99-23 Study No. F93TL02 S.

Robinson, K.M., Rice, C.E., and Kadavy, K.C., 1998. Design Rock Chutes, Transactions of the American Society of Agricultural Engineers, 41(3): 621-626.

Simons, D. B., and Senturk, F., 1992. Sediment Transport Technology: Water and Sediment Dynamics, Water Resources Publications, Littleton, CO.

Simons, Li and Associates, Inc., 1981, *Design Guidelines and Criteria for Channels and Hydraulic Structures on Sandy Soil*, Urban Drainage and Flood Control District and City of Aurora, Colorado.

——, 1989, *Sizing Riprap for the Protection of Approach Embankments and Spur Dikes and Limiting the Depth of Scour at Bridge Piers and Abutments*, prepared for Arizona Department of Transportation, Report No. FHWA-AZ89-260.

——, 1989a, Sizing Riprap for the Protection of Approach Embankments and Spur Dikes and Limiting the Depth of Scour at Bridge Piers and Abutments. Arizona Department of Transportation.

——, 1989b, (revised July, 1998). Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona. Prepared for City of Tucson, Department of Transportation, Engineering Division.

Subramanya, K., 1997. Flow in Open Channels. McGraw-Hill, 2nd edition.

Thomsen, B.W., and Hjalmarson, H.W., 1991, *Estimated Manning's Roughness Coefficients for Stream Channels and Floodplains in Maricopa County, Arizona*, U.S. Geological Survey, Water Resources Division, Tucson, Arizona, Prepared for the Flood Control District of Maricopa County.

U.S. Army Corps of Engineers (USACE), 1981, *Final Report to Congress, The Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251*, Washington, D.C.

——, 1990, *HEC-2 Water Surface Profiles, User's Manual*.

——, 1990, *Engineering and Design - Construction with Large Stone, Engineer Manual EM 1110-2-2302.*

——, 1992, "Engineering and Design: Design and Construction of Grouted Riprap," Technical Letter ETL No. 1110-2-334, US Army Corps of Engineers, Washington, DC 20314-1000.

——, 1994, Hydraulic Design of Flood Control Channels. Engineer Manual 1110-2-1601.

——, 2001a, *HEC-RAS River Analysis System, Hydraulic Reference Manual*. [HEC WEB Site]

——, 2001b, *HEC-RAS River Analysis System, User's Manual*. [HEC WEB Site]

U.S. Department of the Interior, Bureau of Reclamation, undated, *Lining for Irrigation Canals*.

U.S. Department of Transportation (USDOT), Federal Highway Administration, 1961, *Hydraulic Design Series No. 3, Design Charts for Open-Channel Flow*. [USDOT Hydraulics WEB Site]

——, 1988, *Design of Roadside Drainage Channels with Flexible Linings, Hydraulic Engineering Circular No. 15, Publication No. FHWA-IP-87-7*.

——,1989, *Design of Riprap Revetment, Hydraulic Engineering Circular No. 11, Publication No. FHWA-IP-89-016*.

U.S. Government Printing Office (USGPO), Code of Federal Regulations, 2000. "44CFR Section 65.10: Mapping of Areas Protected by Levee Suystems." http://www.gpo.gov/fdsys/pkg/CFR-2000-title44-vol1

Wright-McLaughlin Engineers, 1969, *Urban Storm Drainage Criteria Manual*, Urban Drainage and Flood Control District, Denver, Colorado.

6.7.2 References Relevant to Chapter

Arizona Department of Transportation, 2000, *Standard Specifications for Road and Bridge Design Construction*.

Brater, E. F. and King, H. W., 1976, *Handbook of Hydraulics for the Solution of Hydraulic Engineering Problems, Sixth Edition*.

Laursen, Emmett M. and Duffy, Dennis M., 1980, *A Study to Advance the Methodology of Assessing the Vulnerability of Bridges to Floods*, University of Arizona.

Maricopa Association of Governments, 1978 (revised 1984), *Uniform Standard Specifications for Public Works Construction*.

National Archives and Records Administration, 1990, *Code of Federal Regulations, Protection of Environment, 40 CFR, Part 230, Section 404*.

Pima County Department of Transportation and Flood Control District, 1984, *Drainage and Channel Design Standards for Local Drainage for Floodplain Management within Pima County, Arizona*.

Richardson, E.V., Simons, D.B., Lagasse, P.F., 2001, *River Engineering for Highway Encroachments, Highways in the River Environment, Hydraulic Design Series No. 6*, U.S. Department of Transportation, Federal Highway Administration.

Sabol, George V., Nordin, Carl F. and Richardson, E.V., 1990, *Scour at Bridge Structures and Channel Degradation and Aggradation Field Data Measurements*, Arizona Department of Transportation.

Simon, Andrew, 1981, *Practical Hydraulics*, John Wiley & Sons

Simons, Li and Associates, Inc., 1985, *Design Manual for Engineering Analysis of Fluvial Systems*, Arizona Department of Water Resources.

U.S. Department of Transportation (USDOT), Federal Highway Administration, 1965, *Hydraulic Design Series No. 4, Design of Roadside Channels*.

7 FRICTION LOSSES IN OPEN CHANNELS

TABLE OF CONTENTS

7.1 SYMBOLS

The following symbols will be used in equations throughout Chapter 7.

A	=	Cross sectional area of channel, ft^2
B	=	Percentage of flow blocked by vegetation
BM	=	Bending moment, ft-lbs
$C_{blocking}$	=	Vegetation-blocking coefficient
C_{dist}	=	Vegetation-distribution coefficient
C_{depth}	=	Flow-depth coefficient
d_{50}	=	Intermediate diameter of bed material that equals or exceeds that of 50 percent of the particles, ft

d_{84}	=	Intermediate diameter of bed material that equals or exceeds that of 84 percent of the particles, ft
D_{max}	=	Maximum depth of flow in cross section, ft
K	=	Conveyance
K_t	=	Conveyance for the cross section
K_v	=	Vegetation-susceptibility index, ft-lbs
m	=	Manning's n-value adjustment for meanders
n	=	Manning's n-value
n_{veg}	=	Vegetation component of Manning's n
P	=	Wetted perimeter, ft
Q	=	Rate of flow, cfs
R	=	Hydraulic radius, ft
S_e	=	Energy gradient, ft/f
S_w	=	Water surface slope, ft/ft
SP	=	Stream power, ft-lbs/sec per ft^2
V	=	Mean velocity, ft/sec
V_{flex}	=	Vegetation-flexibility factor, ft-lbs

7.2 CONVERSION FACTORS

TABLE 7.1
CONVERSION FACTORS

Multiply	By	To Obtain
Length		
inches (in)	25.4	millimeters (mm)
feet (ft)	0.3048	meters (m)
miles (mi)	1.609	kilometers (km)
Area		
square miles (mi^2)	2.590	Square kilometers (km^2)
Flow rate		
cubic feet per second	0.02832	cubic meters per second (m^3/s)

7.3 INTRODUCTION

Forward: This chapter is derived from the U.S. Geological Survey Scientific Investigations Report 2006-5108 (Phillips and Tadayon, 2006). It is reproduced here in its entirety, with some slight changes in organization, formatting, and table and figure titles. It is intended to be a guide for estimation of friction losses in both natural and constructed open channels through selection of Manning's n. In addition, guidance and design considerations for friction loss estimation are provided for planning for ongoing management of vegetation so that public safety can be maintained in conjunction with other design goals including preservation and enhancement of riparian habitat, and landscape character.

The U.S. Geological Survey, in cooperation with the Flood Control District of Maricopa County, has been studying the hydraulic effects associated with channel-roughness elements in streams in Arizona. Computation of flow in an open channel requires evaluation of the channel's resistance to flow, which is typically represented by a roughness parameter, such as Manning's n. The characteristics of natural channels and of some constructed channels and the factors that affect channel roughness can vary greatly; however, the combinations of these factors are numerous. In many cases, components of Manning's n cannot be determined with sufficient accuracy by direct measurement of roughness characteristics, such as vegetation and variations in channel shape. Therefore, selection of roughness for natural and constructed channels typically is based on field judgment and skill, which are acquired mainly through experience. The expertise necessary for proper selection of roughness coefficients can be obtained, in part, by examining characteristics of channels that have known or verified coefficients (Barnes, 1967; Aldridge and Garrett, 1973; Phillips and Ingersoll, 1998), or have been selected by experienced personnel (Thomsen and Hjalmarson, 1991). The roughness coefficient can be verified by computations made by using data from streamflow measurements and from measurements of the physical features of the channel. Photographs of channel segments for which n-values have been verified can be used as a comparison standard to aid in assigning n-values to similar channels. Semi-empirical equations that relate hydraulic and channel properties have been derived from verified values of Manning's n. The equations also can be used as a tool for selection of n-values.

In the arid to semi-arid southwestern United States, one factor that retards flow and that can have the greatest single impact on energy losses and resulting computed water surface elevations is the vegetation occupying the channel bed, banks, and overflow areas. Vegetation characteristics for particular channel reaches may have a larger effect than all other flow resistance elements by a factor of three to four (Phillips and Ingersoll, 1998). Vegetation is a constantly changing factor as well; it can be laid over or removed during floodflows, or grow to substantial spatial densities and heights in just a few years' time. Different species of vegetation also have different flexural strengths for a given size or height, which further complicates assessing flow impacts on vegetation, and the subsequent impact of vegetation on flow-energy losses. When vegetation for a particular channel either grows to significant heights and densities or is laid over and possibly removed during floodflows, the roughness coefficients selected for that channel for earlier hydraulic studies, years, or decades may have changed significantly, possibly significantly impacting the earlier computed conveyance and water surface elevations for the design discharge. A semi-empirical relation has been developed that relates hydraulic properties of flow to vegetation characteristics and conditions within the channel (Phillips, et al., 1998). The relation will allow the user to determine impact of flow on the vegetation so estimates of n-values for the vegetation component can be more accurately selected. The relation is restricted primarily to vegetation growing in the main channel of natural and constructed stream channels.

In past decades, these heavy growths of vegetation may have been modified or removed completely to allow for adequate conveyance of floodflows. With a shift in emphasis in recent years toward preserving riparian vegetation to provide habitat for many species of wildlife and aesthetically pleasing multi-use areas for homeowners and businesses, however, engineering-based vegetation maintenance guidelines are now deemed to be necessary. Vegetation maintenance guidelines presented in this document are intended to optimize the preservation of riparian habitat and the aesthetics of multi-use areas, while mitigating damage from floodflows along natural and constructed channels.

7.3.1 Purpose and Scope

Limerinos (1970) stated that it is unlikely the determination of n-values for channels will ever be an exact science; and Barnes (1967) indicated the selection of n-values remains chiefly an art primarily developed through experience. According to Chow (1959), veterans at selecting n-values should exercise sound engineering judgment and experience; for a beginner, selection of n-values can be no more than a guess, and different individuals will obtain different results. The methods and guidelines herein, therefore, are intended to be an aid for development of experience necessary to negate gross errors in the selection of n-values for open channel flow hydraulic computations. These guidelines also are intended to be a tool for (1) selection of roughness coefficients by veteran engineers and hydrologists, (2) assessment of flow on vegetation conditions, and (3) evaluation of vegetation conditions in constructed channels to determine the potential need for vegetation maintenance.

Engineering based vegetation assessment and maintenance guidelines are necessary to optimize preservation of riparian habitat and aesthetic value of multi-use areas, while ensuring channel conveyance is adequate to mitigate flood damage. The compilation of information from past publications into a new comprehensive manual, as well as newly developed vegetation-maintenance plan guidelines, can provide a substantive mechanism by which private sector managers and engineers; and local, state, and federal officials, as well as the public, can acquire better estimates of n-values for open channel flow computations in central Arizona, as well as similar arid to semi-arid regions of the United States and the world.

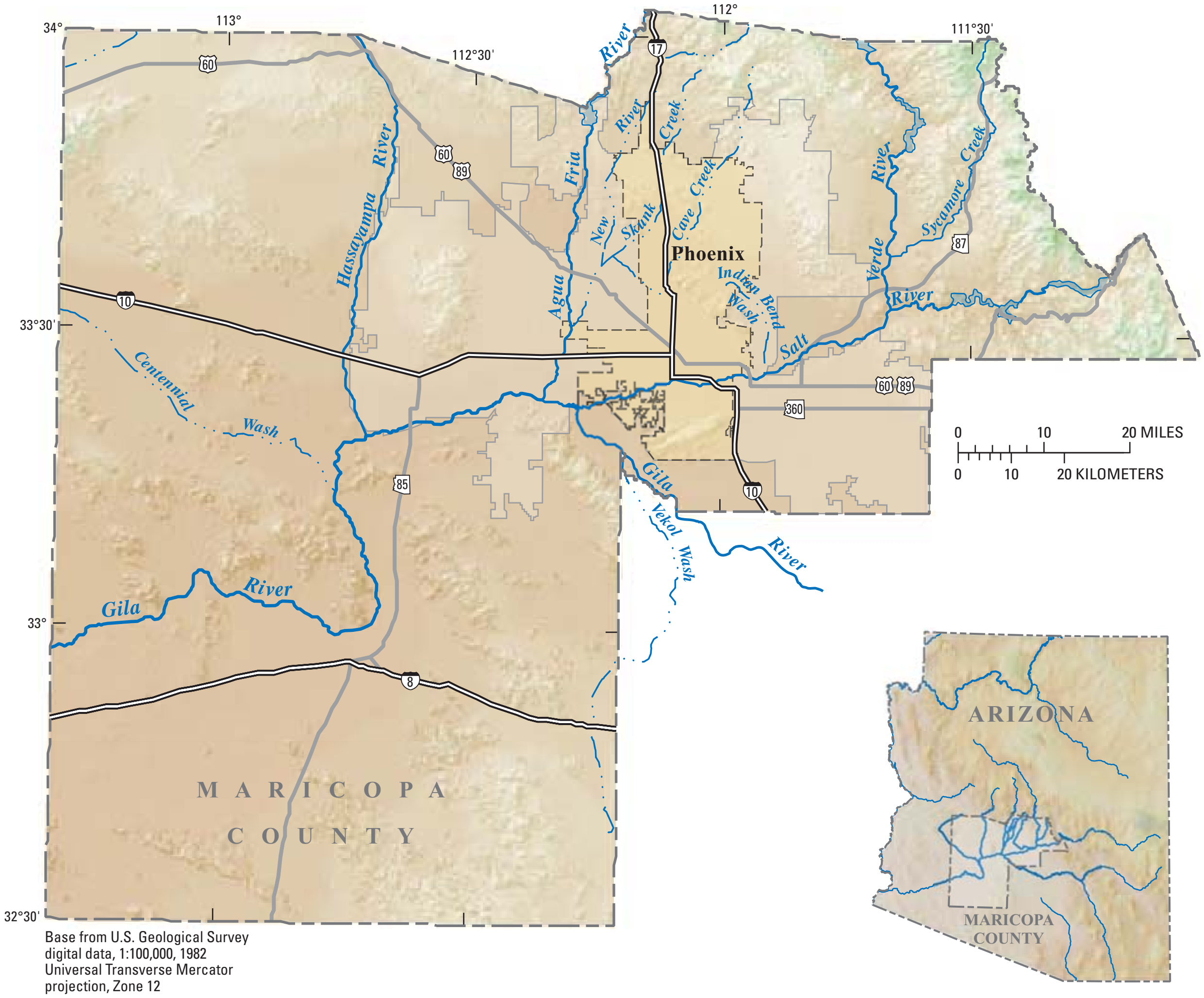
7.4 DESCRIPTION OF STUDY AREA

The basin and range topography typical in most parts of Arizona is characterized by steep block-faulted mountains separated by gently sloping valleys. Ephemeral and intermittent streams in the study area (Figure 7.1) cover a wide variety of conditions ranging from unstable alluvial channels, generally stable channels of cobble to boulder-sized bed material, and extremely stable bedrock channels. Sand-dominated streambeds commonly are characterized by unstable boundary conditions, high sediment loads, and long periods of low or no flow punctuated by brief periods of flooding that increase discharge several orders of magnitude within minutes. Although generally more stable than sand channels, some gravel-dominated channels in Arizona are ephemeral or intermittent and subject to flooding for brief periods. Flash flooding and the general instability of the beds of natural channels in Arizona complicate the task of accurately selecting roughness characteristics that may represent conditions during peak flow. Many stream channels in urban areas are manmade and fairly stable. They may be composed of either soil, cement, concrete, riprap, grouted and wire-enclosed rock, firm earth, grass, or a combination of these materials.

The type, distribution, and density of riparian vegetation can vary in the study area. Vegetation types found in and along many streams in central Arizona include saltcedar, willow, cottonwood, mesquite, palo verde, and many shrub and grass species. Effluent-dominated streams in the study area may contain elevated nutrient levels resulting in increased vegetation growth. Vegetation in ephemeral and intermittent streams and constructed channels in central Arizona can be the primary factor in estimating total resistance to flow.

Mean annual precipitation in the study area ranges from about 7 in. near Phoenix to more than 30 in. in adjacent mountain ranges. Precipitation in Arizona mainly occurs during June through October and December through March; rainfall is about equal in each period. Summer precipitation normally is produced by convective thunderstorms. These storms are characterized by rainfall of high intensity and short duration. They usually cover small areas and may result in flash floods. Winter precipitation normally is produced by regional frontal systems that are characterized by low-intensity rainfall of long duration that covers a large areal extent. Dissipating tropical cyclones cause storms in Arizona that occur primarily in September and October (Webb and Betancourt, 1992). These storms can cause record floods of regional extent.

FIGURE 7.1
MAP SHOWING STUDY AREA IN MARICOPA COUNTY, ARIZONA



7.5 MANNING'S EQUATION

Owing to its simplicity of form and to the satisfactory results it lends to practical applications, Manning's equation has become the most widely used of all uniform-flow equations for open-channel flow computations (Chow, 1959). Manning's equation in the following form is commonly used to compute discharge in natural channels:

$$Q = \left(\frac{1.486}{n}\right) A R^{2/3} S_e^{\ 1/2} \tag{7.1}$$

where: Q = discharge, in cubic feet per second,

A = cross section area of channel, in square feet,

R = hydraulic radius [A/P in feet, where P = wetted perimeter],

S_e = energy gradient, in feet per foot, and

n = Manning's roughness coefficient.

Equation (7.1) was developed for conditions of uniform flow in which the area, depth, and velocity are constant throughout the reach (Thomsen and Hjalmarson, 1991). The equation is also valid for non-uniform reaches if the energy gradient is modified to reflect only the losses due to boundary friction. In applying Manning's equation, the greatest difficulty lies in the determination of the roughness coefficient, *n* (Chow, 1959).

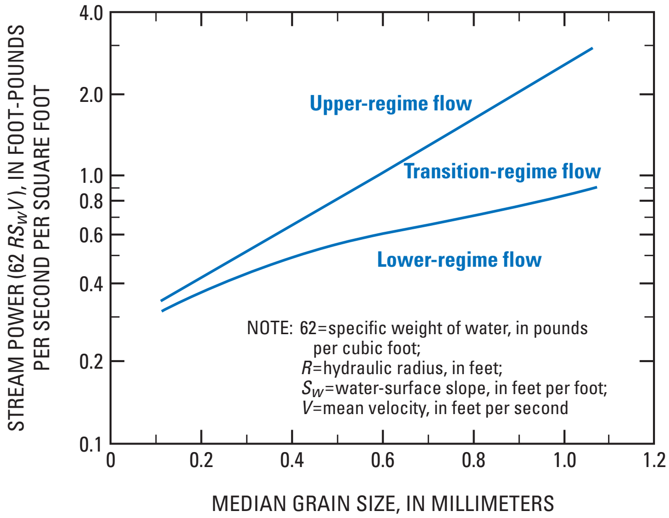
7.6 VALUES OF MANNING'S n FOR MAIN CHANNELS AND OVERBANK AREAS

Values of Manning's n may be assigned for conditions that exist at the time of a specific flow event, for average conditions over a range in water-flow depths, or for anticipated conditions at the time of some future flow event. The value assigned to a reach should represent the composite effects of the factors that tend to retard flow (Aldridge and Garrett, 1973). In developing the ability to assign n-values, a person must rely to a great degree on values that have been verified and on values that have been assigned by experienced personnel (Aldridge and Garrett, 1973; Thomsen and Hjalmarson, 1991).

7.6.1 Base Values of *n* for Unstable Channels

An unstable, or sand channel is defined as a channel in which the bed has an unlimited supply of sand (Aldridge and Garrett, 1973). Sand ranges in grain size from 0.062 to 2 mm. Resistance to flow varies greatly in sand channels because the bed material moves easily and takes on different configurations or bed forms. The type of bed form is a function of many components, including velocity of flow, grain size, boundary shear, and other variables. The magnitude of Manning's *n* may relate directly to the type of bed form that is manifested. The flows that produce the bed forms are classified as lower regime and upper regime flows separated by a transition zone (Figure 7.2).

FIGURE 7.2
RELATION OF STREAM POWER AND MEDIAN GRAIN SIZE TO FLOW REGIME



The flow regime is governed by the size of the bed material and the stream power, which is a measure of energy transfer. Simons and Richardson (1966) defined stream power (SP) as

$$SP = 62.4RS_wV \tag{7.2}$$

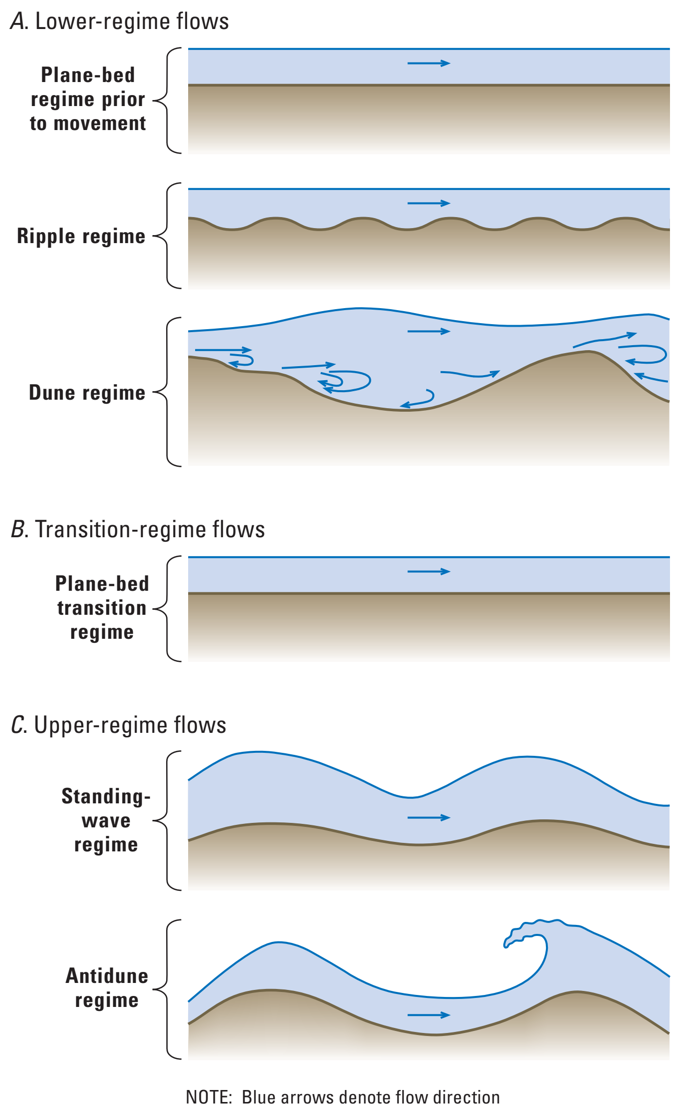
where:

where:	*62.4*	=	specific weight of water, in pounds per cubic foot,
	R	=	hydraulic radius, in feet,
	S_w	=	water surface slope, in feet per foot, and
	V	=	mean velocity, in feet per second.

In lower-regime flow, the bed may have a plane surface with little or no movement of sand or small, uniform waves (ripples), or it may have large, irregular waves (dunes) that are formed by sediment moving downstream. Water surface undulations manifested in lower-regime flow generally are out of phase with the bed surface (Figure 7.3). The fact that the water surface is out of phase with the bed surface is a positive indication that the flow is tranquil or subcritical (Simons and Richardson, 1966, p. J9).

The bed configuration in the transition-zone regime can be erratic and may manifest bedforms typical to those in upper-regime flow depending mainly on antecedent conditions (Simons and Richardson, 1966, p. J11). Resistance to flow and sediment transport also has the same variability as the bed configuration in the transition zone.

FIGURE 7.3
IDEALIZED DIAGRAM OF BED AND SURFACE CONFIGURATIONS
for Alluvial Streams for Various Regimes of Flow



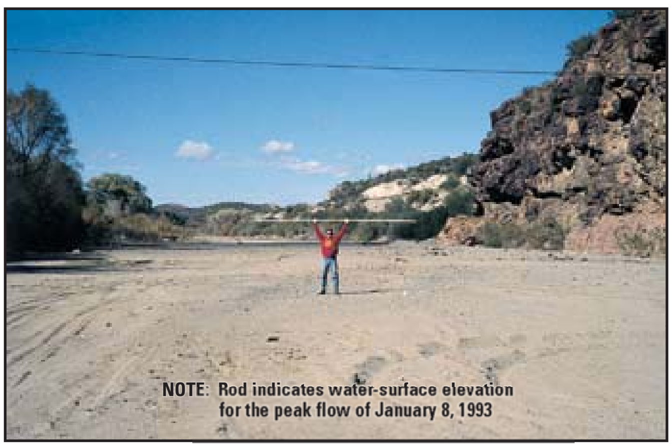
In upper-regime flow, the bed may have a plane surface or it may have long, smooth sand formations in phase with the surface waves (Leopold, et al., 1964; Karim, 1995). These surface waves are known as standing waves or antidunes (Figure 7.3; Simons and Richardson, 1966). As the size of the antidunes grow, the water surface slope on the upstream side of the waves becomes steeper, and the antidune may eventually collapse. Following collapse of the antidunes, the flow generally will shift back to plane-bed conditions. When antidune formations occur in upper-regime flow and the water and bed surface are in phase, the flow is rapid or supercritical (Simons and Richardson, 1966, p. J9).

The n-value for a sand channel is generally assigned for upper-regime flow, and the flow regime is checked by computing the velocity and subsequently the stream power that corresponds to the assigned n-value. The computed stream power is compared with the n-value necessary to cause upper regime flow.

Aldridge and Garrett (1973, p. 5) suggest that n-values for lower- and transitional-regime flows can vary greatly and depend on the bed forms present at a particular time; these values generally will be much larger than the values for upper-regime flow. Unfortunately, there is a lack of definition of roughness coefficients available for the lower regime (Benson and Dalrymple, 1967). Most flood peaks on sand channels, however, occur when the bed configuration is in the upper regime (Figure 7.4A and B). According to Benson and Dalrymple (1967), the n-values for upper-regime flow are dependent on the median grain size of bed material (Table 7.2).

Figure 7.4
Typical Unstable Sand Channel in Central Arizona
A, View Upstream of Midchannel During No-Flow Period.
B, View from Cableway Looking Upstream During Flow of February 9, 1993.

A. No-flow view



B. Flow view

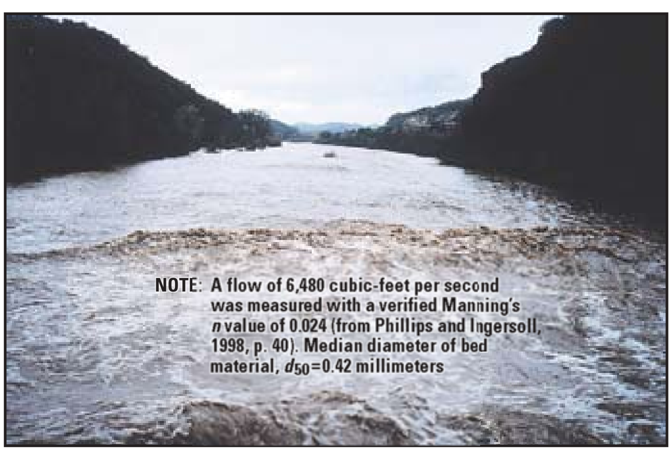


TABLE 7.2
BASE VALUES OF n FOR UPPER-REGIME FLOWS IN SAND CHANNELS
[Modified from Benson and Dalrymple (1967)]

Median Size of Bed Material (mm)	Base n-value
0.2	0.012
0.3	0.017
0.4	0.020
0.5	0.022
0.6	0.023
0.8	0.025
1.0	0.026

7.6.2 Base Values of *n* for Stable Channels

A stable channel is defined as a channel in which the bed is composed of firm earth, gravel, cobbles, boulders, or bedrock and remains relatively unchanged through most of the range in flow (Aldridge and Garrett, 1973). Base n-values for stable channels have been determined mainly from field-verification studies. Base n-values for firm earth, gravel, cobble, and boulder channels can be selected by visually comparing the characteristics with those of channels that have known or verified coefficients (Barnes, 1967; Aldridge and Garrett, 1973; Phillips and Ingersoll, 1998), by comparing measured size of bed material with verified values of Manning's *n* (Table 7.3), or by use of equations derived from channel and hydraulic parameters and verified values of Manning's *n*. Base n-values for bedrock channels can be selected by visual comparison with bedrock channels where Manning's *n* has been verified.

TABLE 7.3
BASE VALUES OF MANNING'S n FOR CHANNELS CONSIDERED STABLE

Channel Type	Median Size of Bed Material		Base n-value	
	Millimeters	Inches	Benson and Dalrymple (1967)	Chow (1959)
Firm earth	---	---	0.025–0.032	0.020
Coarse sand	1–2	---	0.026–0.035	---
Fine gravel	---	---	---	0.024
Gravel	2–64	0.08–2.5	0.028–0.035	---
Coarse gravel	---	---	---	0.028

TABLE 7.3
BASE VALUES OF MANNING'S n FOR CHANNELS CONSIDERED STABLE

Channel Type	Median Size of Bed Material		Base n-value	
	Millimeters	Inches	Benson and Dalrymple (1967)	Chow (1959)
Cobble	64–256	2.5–10.5	0.030–0.050	---
Boulder	> 256	> 10	0.040–0.070	---

7.6.3 Equations for Selection of Base n-values for Stable Channels

Base n-values for stable channels also can be assigned through the use of equations developed from verified channel reaches that relate Manning's *n* to easily measured hydraulic and channel parameters (equations (7.3) and (7.4)). Several investigators have presented data that indicate trends exist among depth or hydraulic radius, median grain size diameter, and verified base values of *n*. For example, Limerinos (1970) examined verified values of *n* for 11 streams in California (Figure 7.6). Limerinos developed an equation to assign base n-values for stable channels that is expressed as:

$$n = \frac{0.0926R^{1/6}}{1.16 + 2.0\ \log\left(\frac{R}{d_{84}}\right)} \tag{7.3}$$

where: R = hydraulic radius, in feet, and

d_{84} = intermediate diameter of bed material, in feet, that equals or exceeds that of 84 percent of the particles.

FIGURE 7.5
TYPICAL COBBLE-BED CHANNEL IN CENTRAL ARIZONA
for Which Manning's **n** was Verified
(Used for Development of Equation (7.4))

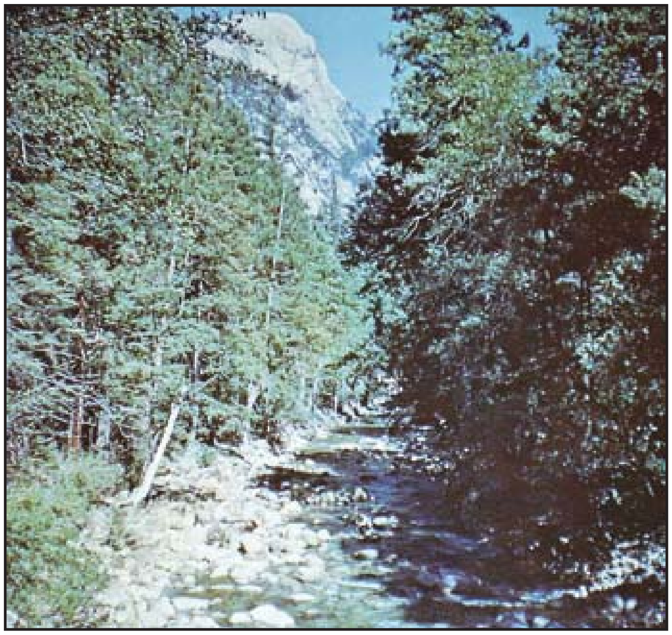
A similar equation was developed for generally lower-gradient stable channels in central Arizona for which the base n-value was the only perceivable factor that contributed to total roughness (Figure 7.5; Phillips and Ingersoll, 1998). That equation is in the form of:

$$n = \frac{0.0926R^{1/6}}{1.46 + 2.23\,\log\left(\frac{R}{d_{50}}\right)} \tag{7.4}$$

where: d_{50} = intermediate diameter of bed material, in feet, that equals or exceeds that of 50 percent of the particles.

The equation was developed by utilizing channels with a median diameter of bed material that ranged from 0.28 to 0.36 foot. These equations have their limitations, but can be utilized as a check or reference for assigning base values of *n*.

FIGURE 7.6
TYPICAL HIGH-GRADIENT COBBLE-BED CHANNEL IN CALIFORNIA
for Which Manning's **n** was Verified and Utilized for Development of Equation (7.3)



7.6.4 Flow Depth and Channel Gradient

Previous investigations indicate there is a relation between depth of flow and n-values (Jarrett, 1985; Phillips and Ingersoll, 1998). In the absence of bank vegetation and other obstructions, the roughness coefficient for flows in a uniform stable streambed generally decreases with increasing depth of flow (equations (7.3) and (7.4)). With increased flow depth, the energy losses associated with the channel-bed roughness elements generally become less significant. As flow approaches bank-full stage, the roughness coefficient may approach a constant value for a given median bed-size material (Limerinos, 1970; Jarrett, 1985; Phillips and Ingersoll, 1998).

Channel roughness seems to be directly related to channel gradient or slope (Riggs, 1976; Jarrett, 1985). Channels with low gradients have been shown to have lower roughness coefficients than channels with high gradients (Jarrett, 1985). Because of the relation between channel slope, size of bed material, and energy losses, the effect of slope on *n* should be considered in the selection of base n-values (Aldridge and Garrett, 1973). Information presented by Jarrett (1985) can be used as a reference for selecting n-values that may be impacted by the channel gradient.

7.6.5 Values and Descriptions For Components of Manning's n

The general procedure for determining n-values is to select a base value of *n* for the bed material (Table 7.2 and Table 7.3) and then select *n*-value adjustments for channel irregularities, alignment, obstructions, vegetation, and other factors (Table 7.4; Cowen, 1956). Utilizing this procedure, the value of *n* is computed as follows:

$$n = (n_0 + n_1 + n_2 + ... + n_n)m \tag{7.5}$$

where: n_0 = base value of *n* for a straight, uniform channel,

$n_{1,} n_2, ..., n_n$ = adjustments for roughness factors other than meanders, and

m = adjustments for meanders.

Degree of Channel Irregularity

The impact of channel irregularity may be negligible where channel margins are extremely smooth (Figure 7.7). Roughness caused by eroded and scoured banks, projecting points, and exposed tree roots along the channel margins, however, can be accounted for by adding adjustments to the base value of *n* (Figure 7.8 and Figure 7.9). Chow (1959) and Benson and Dalrymple (1967) indicate that severely eroded and scoured banks can increase n-values by as much as 0.020 (Figure 7.10; Table 7.4).

FIGURE 7.7
THE MANNING'S n COMPONENT FOR CHANNEL BANK IS CONSIDERED SMOOTH
with a Corresponding Component of 0.000 (Table 7.4)

FIGURE 7.8
THE MANNING'S n COMPONENT FOR THE ERODED AND SCOURED BANKS
is Considered Moderate with a Range of 0.006 to 0.010 (Table 7.4)

FIGURE 7.9
THE MANNING'S n COMPONENT FOR THE ERODED AND SLIGHTLY SCOURED BANKS
is Considered Minor with a Range of 0.001 to 0.005 (Table 7.4)

Variation in Channel Cross Section

Gradual changes in the size and shape of a channel cross section should have no impact on energy losses (Figure 7.11). Where large and small cross sections alternate occasionally, or the main flow occasionally shifts from side to side, adjustments to the base n-value can range from 0.001 to 0.005. Chow (1959) gave a maximum increase of 0.015 in channels where large and small cross sections alternate frequently or where the low-water channel frequently shifts from side to side (Table 7.4).

TABLE 7.4
ADJUSTMENT FACTORS OR COMPONENT RANGES FOR VARIOUS CHANNEL CONDITIONS
Used to Determine Manning's n-values

(Adjustment to degree of meandering values apply to flow confined in the channel and does not apply where flow crosses meanders; Modified from Cowen, 1956; and Chow, 1959.)

Channel Conditions	Manning's n Adjustment	Example
Degree of irregularity		
Smooth	0.000	Smoothest channel attainable in a given bed material.
Minor	0.001–0.005	Channels with slightly scoured or eroded side slopes.
Moderate	0.006–0.010	Channels with moderately sloughed or eroded side slopes.
Severe	0.011–0.020	Channels with badly sloughed banks; unshaped, jagged, and irregular surfaces of channels in rock.
Variation in channel cross section		
Gradual	0.000	Size and shape of channel cross sections change gradually.
Alternating occasion-ally	0.001–0.005	Large and small cross sections alternate occasionally, or the main flow occasionally shifts from side to side owing to changes in cross section shape.
Alternating frequently	0.010–0.015	Large and small cross sections alternate frequently, or the main flow frequently shifts from side to side owing to changes in cross section shape.
Effects of obstructions		
Negligible	0.000–0.004	A few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, piers, or isolated boulders, which occupy less than 5 percent of the channel.
Minor	0.005–0.015	Obstructions occupy from 5 to 15 percent of the cross section area and spacing between obstructions is such that the sphere of influence around one obstruction does not extend to the sphere of influence around another obstruction. Smaller adjustments are used for curved, smooth-surfaced objects than are used for sharp-edged, angular objects.
Appreciable	0.020–0.030	Obstructions occupy from 15 to 50 percent of the cross section area, or the space between obstructions is small enough to cause the effects of severe obstructions to be additive, thereby blocking an equivalent part of a cross section.

TABLE 7.4
ADJUSTMENT FACTORS OR COMPONENT RANGES FOR VARIOUS CHANNEL CONDITIONS
Used to Determine Manning's n-values

(Adjustment to degree of meandering values apply to flow confined in the channel and does not apply where flow crosses meanders; Modified from Cowen, 1956; and Chow, 1959.)

Channel Conditions	Manning's n Adjustment	Example
Severe	0.040–0.060	Obstructions occupy more than 50 percent of the cross section area, or the space between obstructions is small enough to cause turbulence across most of the cross section.
Amount of vegetation		
Negligible	0.000–0.002	Grass, shrubs, or weeds were permanently laid over during flow.
Small	0.002–0.010	Dense growths of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation where the vegetation is not laid over. Trees, such as willow, cottonwood, or saltcedar, growing where the average depth of flow is at least three times the height of the vegetation. Flow depth is about two times the tree height, and the trees are laid over.
Medium	0.010–0.025	Moderately dense grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of vegetation; brushy, moderately dense vegetation, similar to 1- to 2-year-old willow trees growing along the banks. A few 8 to 10-year old willow, cottonwood, mesquite, or palo verde, which blocks flow by approximately 1 to 10 percent, and spheres of influence or turbulence do not overlap.
Large	0.025–0.050	8- to 10-year-old willow, cottonwood, mesquite or palo verde trees (block flow by approximately 10 to 30 percent where the sphere's of influence overlap) intergrown with some weeds and brush where the hydraulic radius exceeds 2 feet.
Very large	0.050–0.100	Bushy willow trees about 1-year old intergrown with weeds alongside slopes or dense cattails growing along the channel bottom; trees intergrown with weeds and brush. Moderately dense (blocks flow by approximately 30 to 50 percent and the sphere's of influence overlap) 8- to 10-year old trees spaced randomly throughout channel where depth of flow approximates height of vegetation.

TABLE 7.4
ADJUSTMENT FACTORS OR COMPONENT RANGES FOR VARIOUS CHANNEL CONDITIONS
Used to Determine Manning's n-values

(Adjustment to degree of meandering values apply to flow confined in the channel and does not apply where flow crosses meanders; Modified from Cowen, 1956; and Chow, 1959.)

Channel Conditions	Manning's n Adjustment	Example
Extremely large	0.100–0.200	Mature (greater than 10 years old) willow trees and tamarisk intergrown with brush and blocking flow by more than 70 percent of the flow area, causing turbulence across most of the section. Depth of flow is less than average height of the vegetation. Dense stands of palo verde or mesquite that block flow by 70 percent or more and hydraulic radius is about equal to or greater than average height of vegetation.
Degree of meandering		
Minor	1.00	Ratio of the channel length to valley length is 1.0 to 1.2.
Appreciable	1.15	Ratio of the channel length to valley length is 1.2 to 1.5.
Severe	1.30	Ratio of the channel length to valley length is greater than 1.5.

FIGURE 7.10
THE MANNING'S n COMPONENT FOR THE SLOUGHED BANKS
(Jagged and irregular surfaces are considered severe with a range of 0.011 to 0.020 (Table 7.4))

FIGURE 7.11
CHANNEL REACH WHERE THE SIZE AND SHAPE OF SECTIONS CHANGES GRADUALLY
(The Manning's n component for this example is considered negligible or 0.000 (Table 7.4))

Effect of Obstructions

Isolated boulders, debris deposits, logs, power poles and towers, and bridge piers that disturb the flow pattern in the channel increase energy losses, or n-values (Figure 7.12 - Figure 7.16). The amount of increase depends on the shape of the obstruction, its size in relation to other roughness elements in the cross section, the number, arrangement, and spacing of the obstructions, and the magnitude of flow velocity (Aldridge and Garrett, 1973). When the flow velocity is high, an obstruction exerts a sphere of influence that can be much larger than the obstruction because the obstruction can affect the flow pattern for considerable distances on each side. At velocities that generally occur in channels that have gentle to moderately steep slopes, the sphere of influence is about 3 to 5 times the width of the obstruction (Figure 7.12; Aldridge and Garrett, 1973). Several obstructions create overlapping spheres of influence and can cause considerable disturbance and loss of energy even though the obstructions may occupy only a small part of the cross section. Aldridge and Garrett (1973) assigned values to four degrees of obstructions (Table 7.4).

FIGURE 7.12
GENERAL FLOW DISTURBANCE CAUSED BY BRIDGE PIERS
at Colorado River near Moab, Utah

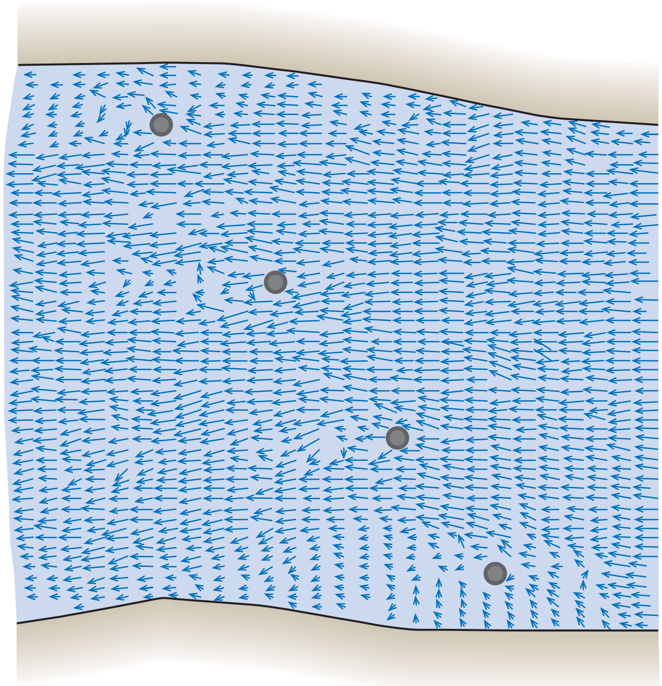


FIGURE 7.13
LARGE ANGULAR BOULDER IN MIDCHANNEL

FIGURE 7.14
POWER POLE OBSTRUCTING LESS THAN 5 PERCENT OF THE CHANNEL AREA
(The Manning's n component for the obstruction is considered negligible, with a corresponding range of 0.000 to 0.004 (Table 7.4))

FIGURE 7.15
REMOVED BRUSH CAUGHT ON MORE FLOW RESISTANT VEGETATION
(Resulting in a localized angular obstruction with a larger sphere of influence than the resistant vegetation alone)

FIGURE 7.16
BRIDGE PIER DEBRIS
(The Manning's n component is considered to range from 0.005 to 0.015 (Table 7.4))

Amount of Vegetation

The degree to which vegetation affects flow depends on the depth of flow relative to vegetation height, the percentage of flow obstructed by the vegetation, the degree to which vegetation is affected or flattened by high water, and the alignment of vegetation relative to the flow (Figure 7.17 - ; Phillips, et al., 1998). In wide channels having small depth to width ratios and no vegetation on the channel bed, the effect of bank vegetation is generally small, and the maximum adjustment is about 0.005. If the channel is relatively narrow and has steep banks covered by dense vegetation that hangs over the channel, the maximum adjustment would be about 0.030. The larger adjustment values given in Table 7.4 apply primarily in places where vegetation covers most of the main channel. If vegetation is the primary factor that affects *n*, as in flood plains, in parts of a channel that are seldom flooded, or in the main channel of ephemeral or intermittent streams, the n-value is assigned for the vegetation rather than for the material in which it is growing (Thomsen and Hjalmarson, 1991). Similar to the impact of obstructions on energy losses, at flow velocities that generally occur in channels that have gentle to moderately steep slopes, the sphere of influence can be about 3 to 5 times the width of the vegetation. Closely clumped trees or reaches where flow-resistant vegetation blocks flow by more than 50 percent of the cross sectional area can create overlapping spheres of influence and can cause considerable disturbance and loss of energy with n-value adjustments that range from 0.050 to 0.200 (Table 7.4).

FIGURE 7.17
TALL GRASS LAID OVER AS A RESULT OF A FLOW OF 6,480 CUBIC FEET PER SECOND

FIGURE 7.18
LONE TREE THAT IS APPROXIMATELY 20 FEET IN HEIGHT

FIGURE 7.19
RANDOMLY SCATTERED SHRUBS
(Flow elevation approximated at the level of the survey rod for a discharge of 403 cubic feet per second)

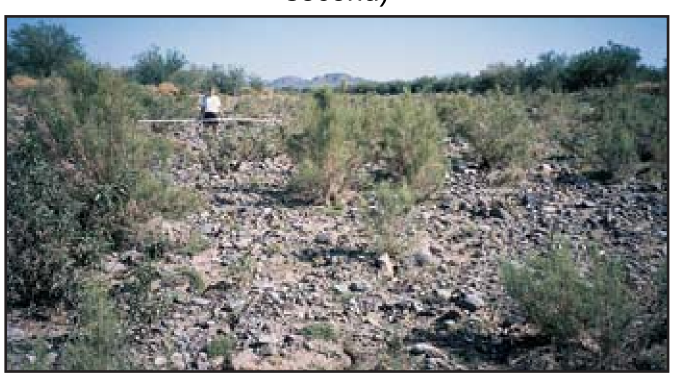


FIGURE 7.20
LARGE MESQUITE WITH BRANCHES THAT HANG OVER THE MAIN-CHANNEL AREA

FIGURE 7.21
RANDOMLY DISTRIBUTED MESQUITE AND PALO VERDE
Approximately 15 to 20 feet in Height (Table 7.4)

FIGURE 7.22
IMAGE SHOWING FLOW ALTERED BY VEGETATION
(Table 7.4)

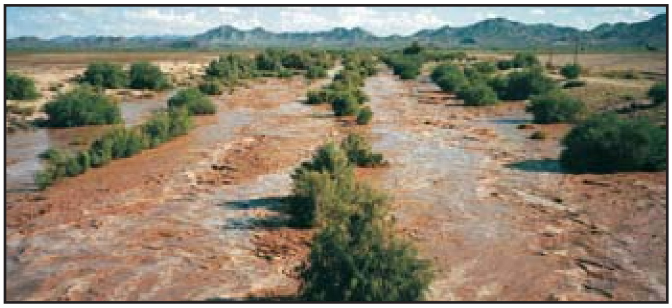


FIGURE 7.23
MANNING'S n COMPONENT FOR THE VEGETATION IS CONSIDERED EXTREMELY LARGE
(with a corresponding range in of 0.100 to 0.200 (Table 7.4))

FIGURE 7.24
EXTREMELY DENSE VEGETATION IN THE CHANNEL THAT DRAINS THIS URBAN AREA
(A, Downstream from midchannel before the flow of December 10, 1991; B, Upstream from left bank during the flow of December 10, 1991)

A. View before flood event

B. View of flooding

Utilizing verified roughness coefficients for a site in central Arizona (Skunk Creek above Interstate 17), Phillips and Ingersoll (1998) developed a semi-empirical relation for non-submerged and randomly-distributed shrubs. The relation or equation is in the form of

$$n_{veg} = 0.0008B - 0.0007 \tag{7.6}$$

where: n_{veg} = vegetation component of Manning's *n*, and

B = percentage of flow blocked by vegetation.

Use of the equation is somewhat limited to channel and vegetation conditions similar to those in Skunk Creek above Interstate 17, Arizona (Figure 7.19; Phillips and Ingersoll, 1998). Extrapolations to other channels with similar types of flow, channel, and vegetation conditions can be made, but should be done so with caution.

7.6.6 Values of Manning's n For Agriculture or Overbank Areas

Values of *n* for fields with crops, as well as for natural vegetation in overbank areas, can be selected on the basis of the work of Chow (1959; Table 7.5). Mature cotton plants are comparable to dense brush in the summer, and defoliated cotton is comparable to medium to dense brush in the winter (Figure 7.25 A and B). For overbank areas, the value of *n* generally varies with the stage of submergence of the vegetation (Thomsen and Hjalmarson, 1991). In general, higher stages should result in lower Manning's n-values.

TABLE 7.5
VALUES OF MANNING'S n FOR AGRICULTURE OR OVERBANK AREAS
[Modified from Chow (1959) and Thomsen and Hjalmarson (1991)]

Description	Manning's *n*		
	Minimum	Normal	Maximum
Pasture, no brush			
Short grass	0.025	0.030	0.035
High grass	0.030	0.035	0.050
Cultivated areas			
No crop	0.020	0.030	0.040
Mature row crops	0.025	0.035	0.045
Mature field crops	0.030	0.040	0.050
Shrubs			
Scattered shrubs, heavy weeds	0.035	0.050	0.070
Light shrubs and trees, in winter	0.035	0.050	0.060
Light shrubs and trees, in summer	0.040	0.060	0.080
Medium to dense shrubs, in winter	0.045	0.070	0.110
Medium to dense shrubs, in summer	0.070	0.100	0.160

TABLE 7.5
VALUES OF MANNING'S n FOR AGRICULTURE OR OVERBANK AREAS
[Modified from Chow (1959) and Thomsen and Hjalmarson (1991)]

Trees			
Dense willows, mesquite, saltcedar	0.110	0.150	0.200
Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
Same as above, but heavy growth of sprouts	0.050	0.060	0.080
Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
Same as above, but with flood stage reaching branches	0.100	0.120	0.160

FIGURE 7.25
EXAMPLES OF COTTON IN SUMMER AND FALL
Fields of Mature Cotton in the Summer (A) and Defoliated Cotton in the Fall (B)

A. Mature cotton

B. Defoliated cotton

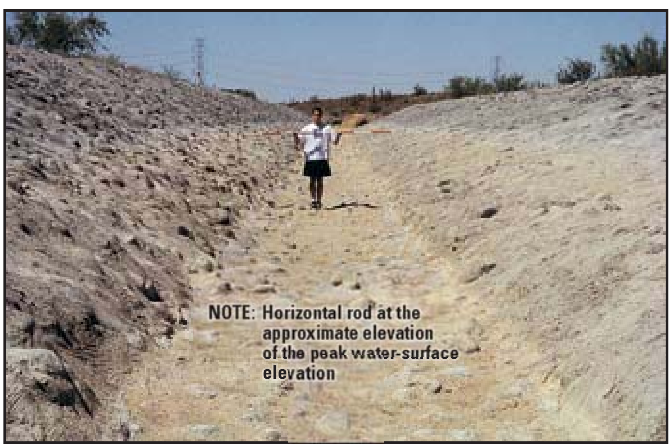
7.6.7 Composite Values of n For Constructed Channels

Composite values of *n* are presented in Table 7.6 for various types of stable constructed channels. The degree of the n-value for a selected channel type is related to the newness of the channel and degree of subsequent maintenance (Figure 7.26 A and B). For example, minimum values correspond to new construction, normal values correspond to good maintenance, and the maximum n-value corresponds to deteriorated or poor maintenance.

FIGURE 7.26
MANNING'S n-VERIFICATION MEASUREMENT
Made at a Well-Maintained Constructed Channel (Phillips and Ingersoll, 1998)
(*A*, Channel survey made for verification of Manning's *n*. *B*, Channel conditions following flow.)

A. Channel survey

B. Channel conditions following flow



7.6.8 Procedure For Subdivision of Cross Sections

The Manning's equation was designed for uniform steady flow in trapezoid channels. Most natural channels, however, are not uniform. The hydrologist or engineer using Manning's equation, therefore, should be aware of its shortcomings and use reasonable judgment to come up with the best results (Cruff, 1999). One of the largest shortcomings of the equation when working with natural channels, and even some constructed channels, is the change in energy loss, or *n*, across or perpendicular to the channel. Because of these changes there is a tendency to subdivide the channel section at changes in roughness. This subdivision method can greatly affect the computation for hydraulic radius, *R*, and significantly and erroneously impact the final computations.

TABLE 7.6
COMPOSITE VALUES OF n FOR STABLE CONSTRUCTED CHANNELS

Type of Channel and Description	n-value[1]		
	Minimum	**Normal**	**Maximum**
A. LINED OR BUILT-UP CHANNELS			
a. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Unfinished	0.014	0.017	0.020
4. Shotcrete, good section	0.016	0.019	0.023
5. Shotcrete, wavy section	0.018	0.022	0.025
b. Soil cement	0.018	0.020	0.025
c. Gravel mulch (1-inch, flow depth 0.5-3.3 ft)	0.031	0.033	0.040
d. Gravel mulch (2-inch, flow depth 0.5-3.3 ft)	0.038	0.042	0.056
e. Cobble and Riprap	See Table 7.7		
f. Grouted riprap	0.028	0.030	0.040
g. Gabions	same as for cobble and riprap linings		
h. 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
B. EVACUATED OR DREDGED CHANNELS			
a. Earth, straight and uniform			
1. Clean, after weathering	0.018	0.022	0.025
2. Gravel, uniform section, clean	0.022	0.025	0.033
b. Earth, winding and sluggish			
1. Earth bottom and rubble sides	0.028	0.030	0.035
2. Stony bottom	0.025	0.035	0.040
3. Cobble bottom and clean sides	0.030	0.040	0.050

TABLE 7.6
COMPOSITE VALUES OF n FOR STABLE CONSTRUCTED CHANNELS

Type of Channel and Description	n-value[1]		
	Minimum	**Normal**	**Maximum**
c. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050

1.Excerpt from: Simons, Li and Associates (1981). Adapted from Chow (1959), Aldridge and Garret (1973), and USDOT (2005).

TABLE 7.7
COMPOSITE MANNING'S n-VALUES FOR ROCK RIPRAP LINED CHANNELS
Source: USDOT (2005), page 6-1, equation 6.1

Average Flow Depth (A/T) (ft)	d_{50} (in)									
	Cobble		**Riprap**							
	2.5	**3.0**	**5.0**	**6.0**	**9.0**	**12.0**	**15.0**	**18.0**	**21.0**	**24.0**
1.0	0.045	0.049	0.062	0.069	n/a	n/a	n/a	n/a	n/a	n/a
1.5	0.042	0.044	0.054	0.059	0.073	0.088	n/a	n/a	n/a	n/a
2.0	0.040	0.042	0.051	0.054	0.066	0.077	0.089	n/a	n/a	n/a
2.5	0.039	0.041	0.048	0.052	0.061	0.070	0.080	0.090	n/a	n/a
3.0	0.038	0.040	0.047	0.050	0.058	0.066	0.074	0.082	0.091	0.099
3.5	0.037	0.039	0.046	0.048	0.056	0.063	0.070	0.077	0.084	0.092
4.0	0.037	0.039	0.045	0.047	0.055	0.061	0.067	0.074	0.080	0.086
4.5	0.036	0.038	0.044	0.046	0.053	0.059	0.065	0.071	0.077	0.082
5.0	0.036	0.038	0.043	0.046	0.052	0.058	0.063	0.069	0.074	0.079
5.5	0.036	0.038	0.043	0.045	0.051	0.057	0.062	0.067	0.072	0.077
6.0	0.036	0.037	0.043	0.045	0.051	0.056	0.061	0.065	0.070	0.074

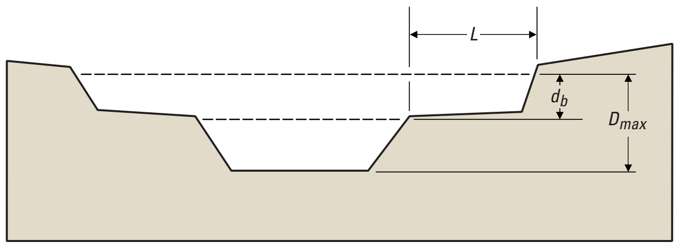
where:

A = cross sectional area of channel at flow depth,

T = channel top width at flow depth.

In most cases the main channel should not be subdivided, and an average *n* should be selected (Cruff, 1999). Cross sections with distinct changes in shape, however, should be subdivided into subsections and the n-values determined separately for each subsection. In this manner the Manning's equation will solve a series of near rectangular or trapezoidal channels, which can produce much more accurate results (Davidian, 1984). Cross sections should be subdivided if the flow-depth in the main-channel is greater than or equal to twice the flow depth at the stream edge of the overflow area (Thomsen and Hjalmarson, 1991; Figure 7.27). Subdivision also should be considered where the width of the overflow area is at least five times the flow depth in the overflow area (Figure 7.27).

FIGURE 7.27
SUBDIVISION CRITERIA COMMONLY USED FOR STREAMS IN MARICOPA COUNTY, ARIZONA

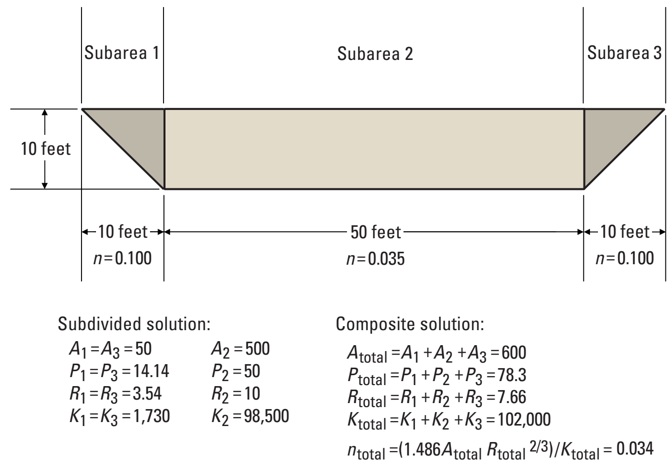


Subdivide if D_{max} is greater than or equal to $2d_b$
Subdivide if D_{max} is approximately equal to $2d_b$ and if L/d_b is equal to or greater than 5

L = width of floodplain
d_b = depth of flow on floodplain
D_{max} = maximum depth of flow in cross section

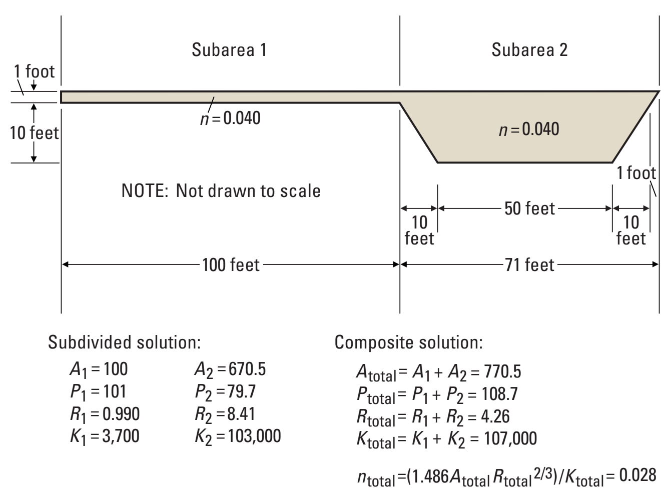
Davidian (1984) presents several examples illustrating the effects of improper subdivision. Figure 7.28 illustrates a cross section of a trapezoidal shaped channel having dense shrubs and trees on the banks; the section was subdivided near the bottom of each bank because of the abrupt change in roughness. A large percentage of the wetted perimeters (P) of the triangular subareas (A_1 and A_3) and possibly of the main channel (A_2) are eliminated. A smaller wetted perimeter abnormally increases the hydraulic radius ($R = A/P$), and this in turn results in a computed conveyance different from the conveyance determined for a section with a complete wetted perimeter. Conveyance (K_1) computed for the cross section in Figure 7.28 would require a composite value of 0.034. This is smaller than the n-values 0.035 and 0.100 that describe the roughness for the various parts of a basic trapezoidal shaped channel. The trapezoidal-shaped cross section in Figure 7.28, therefore, should be left unsubdivided.

FIGURE 7.28
EFFECTS OF SUBDIVISION ON A TRAPEZOIDAL SECTION



At the other extreme, the panhandle of the cross section in Figure 7.29, which has a main channel and an overbank area, should be subdivided into two parts at the abrupt change in geometry. The value of *n* is 0.040 throughout the section. If the section is not subdivided, the increase in wetted perimeter of the overbank area is relatively large with respect to the increase in area. The hydraulic radius is abnormally reduced, and an erroneous, lower n-value of 0.028 is needed to obtain the conveyance equivalent to that of the combined conveyances (K_1 and K_2, Figure 7.29). Irregular cross sections with major breaks in channel geometry (Figure 7.27), therefore, should be subdivided to create individual basic shapes.

FIGURE 7.29
EFFECTS OF NOT SUBDIVIDING A PANHANDLE SECTION



7.6.9 Procedure for Selection of *n* For Changing Vegetation Conditions

Cowen (1956) indicated that channel vegetation can have the single greatest potential effect on the total roughness coefficient selected for a reach. Thomsen and Hjalmarson (1991) describe the major effect of vegetation on total roughness for streams in semi-arid to arid climates typical of the southwestern United States. For intermittent and ephemeral channels in these types of environments, vegetation may grow to substantial heights and densities in only a few years. Such growth throughout the main channels of natural and manmade streams can result in significant reduction in flow velocities and large increases in estimates of *n* (Aldridge and Garrett, 1973; Thomsen and Hjalmarson, 1991; Phillips, et al., 1998; Fischenich, 2000; Table 7.4). In some cases, however, although the vegetation may appear substantial, peak flows during moderate to large flooding can be powerful enough to layover or remove weaker vegetation (Burkham, 1976; Phillips and Hjalmarson, 1994; Phillips, et al., 1998). The flattened or removed vegetation may markedly decrease preflow estimates of *n*. Assuming the impact on vegetation occurs prior to peak flow, the decrease in *n* would increase peak-flow channel conveyances. Increased conveyance effectively lowers peak-flow water surface elevation compared with preflow simulations.

A study was conducted in central Arizona to better understand the relation between the power of flow, the changes in main-channel vegetation conditions, and the impact of the changes on computed water surface elevations (Phillips, et al., 1998). Flow and vegetation characteristics data were collected for development of a method to determine the impact of flow on vegetation conditions. Flow data included channel slope, channel cross section geometry, and measured or computed discharge. Stream power was computed from these data (equation (7.2)). Vegetation characteristics or conditions, such as average height and density, were measured or estimated, described, and photographed before and after peak flows. A fundamental assumption needed to determine flow impact on vegetation conditions is that a critical stream power exists for specific vegetation conditions and that vegetation will bend or fracture when the critical stream power value is exceeded.

Adequately describing all the physical components that collectively characterize vegetation conditions in stream channels in central Arizona can be a complex and difficult task. Four vegetation characteristics were used to model the impact of flow on vegetation. The characteristics include the following: (1) flexural strength of the specific type and size of vegetation, (2) percent of flow blocked by the vegetation, (3) distribution of vegetation within the channel, and (4) depth of flow relative to the average vegetation height (Phillips, et al, 1998).

The vegetation characteristics comprise a composite value called the vegetation-susceptibility index. The vegetation-susceptibility index is defined by:

$$K_v = V_{flex} C_{blocking} C_{dist} C_{depth} \tag{7.7}$$

where: K_v = vegetation-susceptibility index, in foot-pounds,

V_{flex} = vegetation-flexibility factor, in foot-pounds,

$C_{blocking}$ = vegetation-blocking coefficient,

C_{dist} = vegetation-distribution coefficient, and

C_{depth} = flow-depth coefficient.

The vegetation flexibility factor, V_{flex}, is considered the most significant factor in determining whether vegetation will bend or remain in a generally upright position when subjected to the power of flow. The unique physical properties of many types of vegetation enable them to bend to extreme angles when force is applied. The degree of bending generally varies for a given applied force. The force required to bend and lay over vegetation was quantified to obtain the flexural strength of different vegetation types (Phillips, et al, 1998).

Dynamometers, which are mechanical instruments that measure the magnitude of tension in cables, were used to determine the force required to lay over four types of vegetation of varying size. The vegetation (saltcedar, willow, mesquite, and palo verde) ranged in height from 3 to 18 feet. Bending moments were determined by computing the product of the moment arm (distance from the base or pivot point to the location where the force was applied) and the force required to bend the vegetation to 45 degrees from vertical. Equations were developed from regression techniques of the bending moment with height for each of the four vegetation types (Table 7.8).

TABLE 7.8
REGRESSION EQUATIONS RELATING BENDING MOMENT TO VEGETATION HEIGHT
(For mesquite, palo verde, saltcedar, and willow. [BM, bending moment, in foot-pounds; H, height of vegetation, in feet])

Vegetation Type	Equation	Coefficient of Determination, r^2
Mesquite	$BM = 10^{0.124H + 0.935}$	0.88
Palo verde	$BM = 10^{0.171H + 0.848}$	0.86
Saltceder	$BM = 10^{0.102H + 0.880}$	0.87
Willow	$BM = 10^{0.122H + 0.581}$	0.98

The bending moment (also referred to as flexural strength or stiffness) of the vegetation at varying heights can be estimated from the equations in Table 7.8. For example, a flexural strength of 63.2 ft-lb is estimated for a 10-foot-tall willow, whereas a flexural strength of 361 ft-lb is estimated for a 10-foot-tall palo verde. It is assumed that a lone palo verde in midchannel is substantially more likely to resist bending than a lone willow in midchannel when they are subjected to a similar magnitude of stream power and degree of submergence. Data acquired and analyzed during method development seem to support this conclusion (Phillips, et al, 1998). For example, Figure 7.30A shows a lone willow about 15 feet tall that was laid over during a flow calculated at 6,590 ft/s; Figure 7.30B shows a lone 16-foot-tall palo verde that remained erect throughout a flow of

9,760 ft/s. Depth of flow was about equal. The magnitude of the stream power that affected the palo verde was 20.2 (ft-lb/s)/ft (Table 7.8). The magnitude of stream power to which the willow was subjected was equal to 12.9 (ft-lb/s)/ft (Table 7.8). These data indicate that the large flexural strength of palo verde enabled it to resist a computed stream power that was substantially larger than the computed stream power that altered or laid over the willow with similar dimensions.

FIGURE 7.30
IMPACT OF SIMILAR FLOWS, OR STREAM POWER, ON DIFFERENT VEGETATION SPECIES
(of Similar Heights)

A. Willow

B. Palo Verde

A separate analysis of the flexural strength of arrowweed and other types of shrubs was not done. The flexural strength of shrubs studied during the investigation (Phillips, et al, 1998) was assumed to be similar to that of willow. Other prevalent types of vegetation common in central Arizona, such as cottonwood and ironwood, were assumed to behave in a similar manner as willow and mesquite, respectively.

During the course of the study, the percent of the flow area blocked by vegetation was assumed to account for the combined resistant force associated with the vegetation (Phillips, et al, 1998). The vegetation-blocking coefficient value, $C_{blocking}$, was determined for each site by assigning a weighted value to the estimated percentage of the cross section area of flow blocked by vegetation (Table 7.9).

The spatial distribution of riparian vegetation in natural and constructed channels can substantially influence the effect of flow on the vegetation (Phillips, et al, 1998). Vegetation aligned parallel to the direction of flow generally results from consistent base flow in a channel. Due to the combined resistant effect of the vegetation during high flow conditions, vegetation aligned parallel to flow can result in the redistribution of velocities across a channel section (Figure 7.31). The combined resistance causes a decrease in the velocities at the immediate location of the vegetation and may lessen the effect of flow on vegetation conditions. When vegetation is randomly distributed throughout a channel, velocity distribution is assumed to be fairly constant across a channel section. Vegetation-distribution coefficients (C_{dist}) were, therefore, determined for vegetation aligned parallel to flow and for vegetation situated in a generally random manner throughout the main channel (Table 7.10).

TABLE 7.9
VEGETATION-BLOCKING COEFFICIENTS
for Selected Areas of Flow Blocked by Vegetation

[<, less than; >, greater than]

Area of Flow Blocked by Vegetation (percent)	Vegetation-Blocking Coefficient
< 30	1
30 to 70	4
> 70	9

TABLE 7.10
VEGETATION-DISTRIBUTION COEFFICIENTS FOR VEGETATION ORIENTATION TO FLOW

Orientation to Flow	Vegetation-Distribution Coefficient
Parallel	3
Random	1

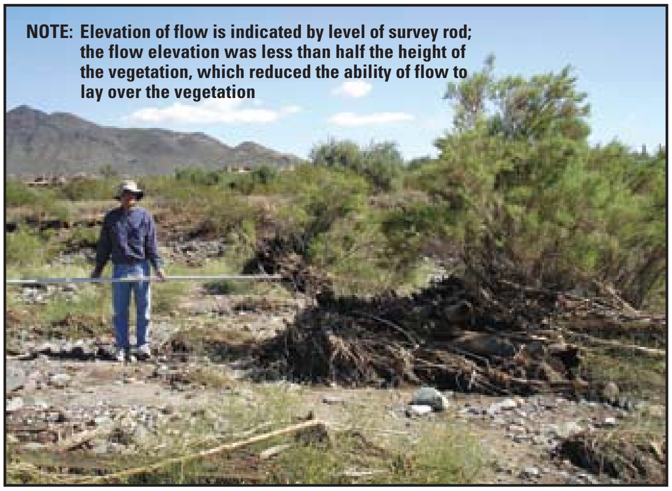
FIGURE 7.31
VEGETATION ALIGNED PARALLEL TO FLOW
as a Result of Consistent Base Flow in a Low-Flow Channel

Flows in vegetated channels do not always result in total submergence of the vegetation. Because strength of vegetation generally increases as the ratio of the moment arm and vegetation height decreases, the depth of flow in relation to vegetation height requires consideration (Figure 7.32).

Flow-depth coefficients (C_{depth}) were determined for five categories that relate hydraulic radius to average vegetation height (Table 7.11). Computed hydraulic radius is assumed to approximate depth of flow at the immediate location of the vegetation.

Vegetation-susceptibility indices were derived from vegetation conditions at selected sites in central Arizona. Stream power was computed for flow events that occurred at these sites. Impact of flow on vegetation conditions was documented shortly following flow. Vegetation-susceptibility indices were compared to stream power, which indicates a trend (Phillips, et al, 1998; Figure 7.33).

FIGURE 7.32
VEGETATION THAT WAS AFFECTED LITTLE BY FLOW



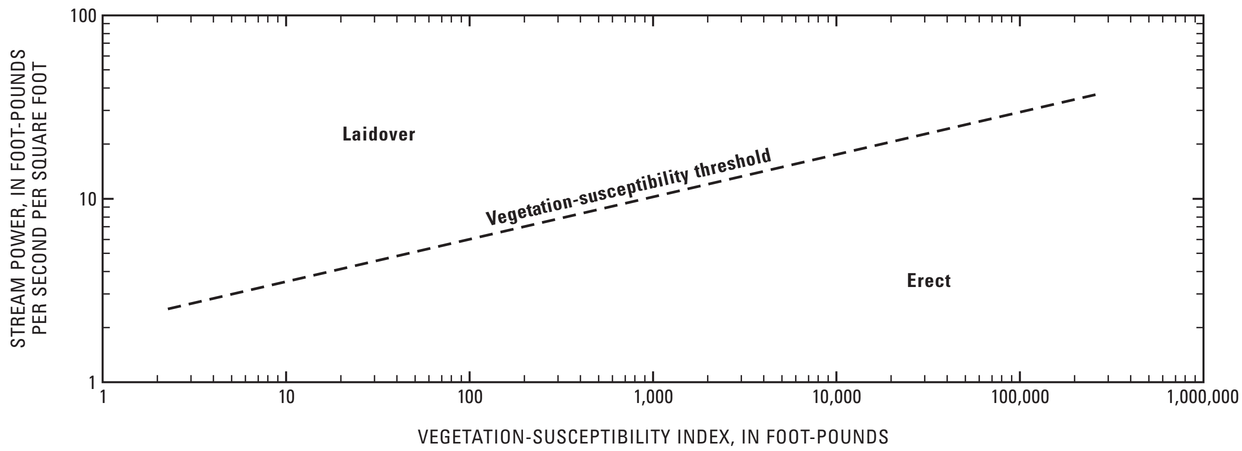
The trend indicates a relation exists between the vegetation-susceptibility index value and the magnitude of stream power (Figure 7.33). According to the relation, as computed vegetation-susceptibility indices increase, the stream power required to significantly impact and lay over the vegetation also increases. The trend line was defined as the vegetation-susceptibility threshold. In general, for stream power values that plot above this threshold, the vegetation can be expected to layover. For method use, the vegetation conditions and flow characteristics studied should be similar to the values used to develop the relationship (Phillips, et al, 1998; Figure 7.33).

TABLE 7.11
FLOW-DEPTH COEFFICIENTS
for Ratios of Hydraulic Radius to Average Vegetation Height

[<, less than; >, greater than]

Ratio of Hydraulic Radius to Average Vegetation Height	Flow-depth Coefficient
< 0.4	60
0.4-0.6	20
0.7-0.9	5
1.0-1.5	3
> 1.5	1

FIGURE 7.33
RELATIONSHIP BETWEEN STREAM POWER AND A VEGETATION-SUSCEPTIBILITY INDEX
for Estimating the Effect of Flow on Vegetation Conditions



7.6.10 Selection Procedure of *n* for Natural and Constructed Channels

The procedure given in this section originally presented by Aldridge and Garrett (1973) involves a series of decisions that are based on the interaction of roughness elements. Decisions required to use the procedure can be difficult to explain in written material (Aldridge and Garrett, 1973). The procedure, therefore, is discussed by steps that are arranged to permit charting in logical order (Figure 7.34). After using the procedure a few times, the user may wish to combine steps or change the order of the steps. Experienced personnel may have the ability to perform the entire operation without the aid of the procedures, but the inexperienced user may find it useful. Steps outlined in Figure 7.35 can be used as a guide for estimating flow impact on main-channel vegetation conditions.

Two example cases for determining total Manning's *n* for a channel reach are provided at the end of this section. The example cases are for a specific design discharge that is confined within the banks of the channel. The hypothetical channel in example 1 consists of parallel bands of material, each of which has a different degree of roughness (Figure 7.36, Figure 7.37, Table 7.12 and Table 7.13). The channel in example 2 consists of gravel and cobbles uniformly distributed in the channel (Figure 7.38, Figure 7.39, Table 7.14 and Table 7.15). The channel also consists of randomly distributed shrubs. The stream power relation is employed to determine impact of flow on the vegetation conditions.

Step 1. Determine the channel type—stable channel, sand channel, or a combination of both—and whether the conditions would be representative of those that would exist during the design flow being considered. Look especially for possible high-water marks, bed movement, and excessive amounts of bank scour (from previous events). Attempt to visualize the conditions that would occur during the peak for the design discharge. Compare with other similar channels for which the roughness coefficient, *n*, has been verified or assigned by experienced personnel in order to estimate the possible range in n-values (Aldridge and Garrett, 1973; Thomsen and Hjalmarson, 1991; Phillips and Ingersoll, 1998).

FIGURE 7.34
FLOW CHART FOR ASSIGNING n-VALUES
(modified from Aldridge and Garrett, 1973).

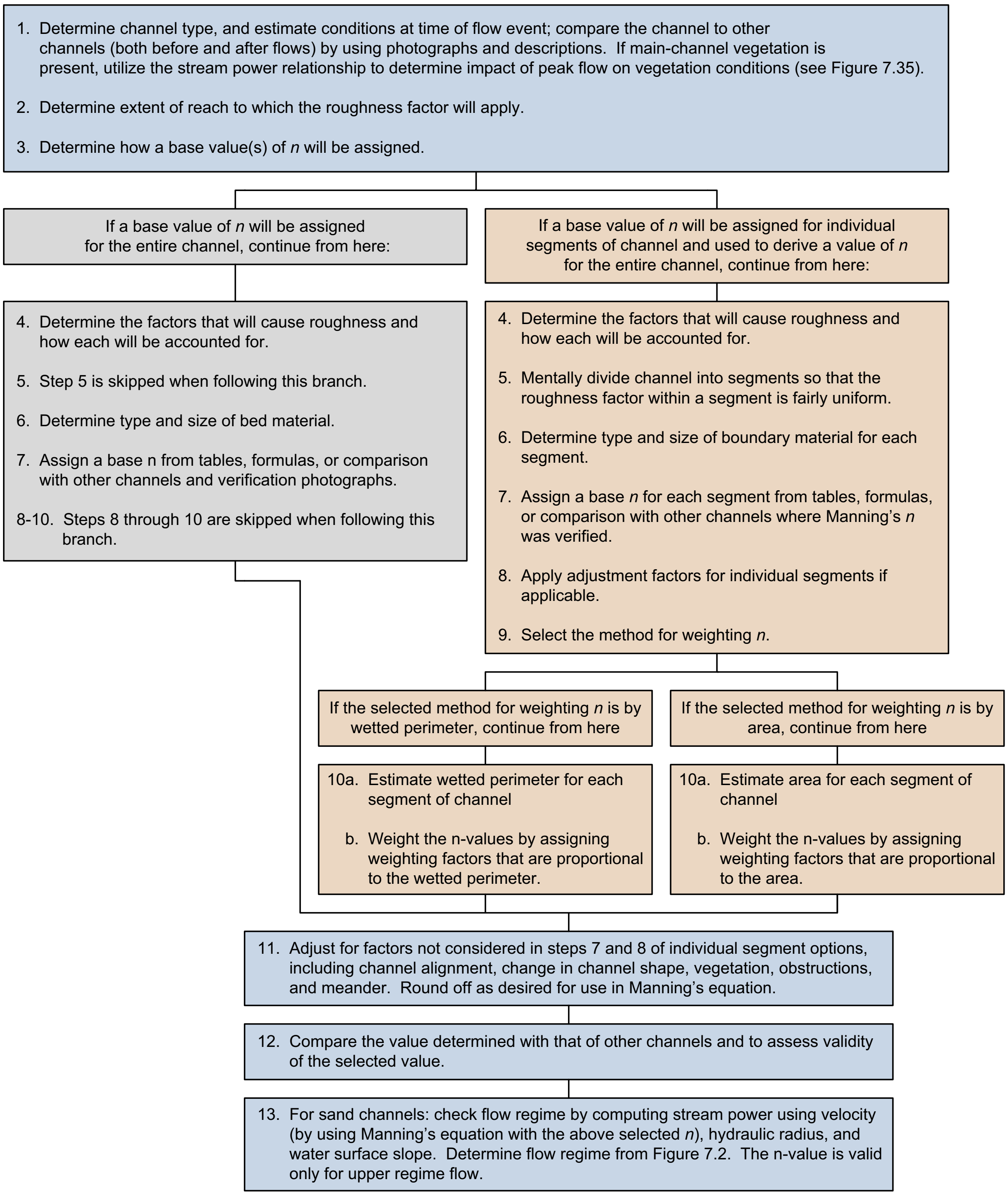


FIGURE 7.35
FLOW CHART FOR ESTIMATING FLOW-INDUCED CHANGES TO VEGETATION CONDITIONS

1a. Survey the channel to obtain parameters necessary for standard-step computations. In the estimation of roughness coefficients, fully weight the pre-flow vegetation characteristics for the selected discharge (see Figure 7.34, step 1).

1b. Run standard-step computations using the surveyed channel parameters and selected roughness coefficients. From the computations, obtain average velocity, hydraulic radius, and water surface slope for the selected cross section.

1c. By using the type and average height of vegetation in the selected cross section, estimate the vegetation-flexibility factor, V_{flex} for each vegetation type using equations found in Table 7.8.

1d. Determine the orientation of the vegetation (either oriented randomly or parallel to the flow), the percent cross section area of flow blocked by vegetation, and the ratio of hydraulic radius to average vegetation height for the selected discharge. From this information, determine the values for the vegetation-blocking coefficient, $C_{blocking}$, the vegetation-distribution coefficient C_{dist}, and the flow-depth coefficient, C_{depth} (see Tables 7.9, 7.10 and 7.11).

1e. Compute stream power for the selected discharge.

1f. Compute the vegetation-susceptibility index for the selected discharge.

1g. Plot the values for the vegetation-susceptibility index and stream power for each type of vegetation present in the channel. If the values plot below the vegetation-susceptibility threshold, the vegetation probably will not be significantly altered by flow, and estimated values of *n* should be weighted accordingly. If the values plot above the vegetation-susceptibility threshold, the vegetation will be altered and possibly laid over. Use engineering judgment to estimate Manning's n-values for the laid over vegetation.

In addition to visualizing conditions at peak flow, especially vegetation conditions, utilize the stream power and vegetation-susceptibility index relation described in the previous section to assist in determining flow impact on vegetation (Figure 7.35). Example case 2 at the end of this section illustrates the use of this method for estimating peak-flow vegetation conditions (Figure 7.38, Figure 7.39, Table 7.14 and Table 7.15).

Step 2. Determine the extent of the reach to which the roughness factor will apply. Although *n* may be applied to an individual cross section that is typical of a reach, it must account for the roughness in the reach of channel that encompasses the section (Thomsen and Hjalmarson, 1991). When two or more cross sections are being considered, the reach that encompasses any one section is considered to extend halfway to the next. For example, see Figure 7.37. In example 1, the n-value for section 2 represents the roughness in reach B. If the roughness is not uniform throughout the reach being considered, n should be assigned for the average condition (Aldridge and Garrett, 1973).

Step 3. If the roughness is not uniform across the width of the channel, determine whether a base *n* should be assigned to the entire cross section, or whether a composite *n* should be developed by weighting values for individual segments of the channel having different amounts of roughness (Aldridge and Garrett, 1973; Jarrett, 1985; Thomsen and Hjalmarson, 1991). When the base value of *n* is assigned to the entire cross section, the channel constitutes one segment being considered, and steps 5, 8, 9, and 10 do not apply in such a case.

Step 4. Determine the factors or individual components that contribute to roughness and how each is to be taken into account. Particular factors may be dominant in a particular segment of the channel, or they may impact the flow for the entire cross section equally. The manner in which each factor is determined depends on how it combines with the other factors (Aldridge and Garrett, 1973). For example, a gently sloping bank may constitute a separate segment of the cross section; whereas, a vertical bank may add roughness either to the adjacent segment or the entire channel. Isolated boulders generally should be considered as obstructions (Aldridge and Garrett, 1973), but if boulders are scattered across the entire reach, it may be necessary to determine the median size of the bed material. Flow resistant vegetation growing in a distinct segment of channel may be assigned an n-value of its own (Aldridge and Garrett, 1973; Thomsen and Hjalmarson, 1991); whereas energy loss caused by vegetation growing on or along steep banks or scattered along the channel bottom will be accounted for by using an adjustment factor that can be applied either to a segment of the channel or to the entire cross section (Aldridge and Garrett, 1973; Phillips and Ingersoll, 1998). Parts of the channel that have dense vegetation and vegetation downstream from projections of banks may be areas of dead water or backwater areas. The backwater areas can be eliminated from the cross section, however, the Manning's n-value for the adjacent segment should be sufficiently high to account for roughness along the streamward side of the brush. If a composite *n* is derived from segments, the user should continue to step 5. For all other instances, step 5 is omitted from the procedure.

FIGURE 7.36
PROCESS FOR COMPUTATION OF MANNING'S n, EXAMPLE 1

Stream and location: See Figure 7.37
Reach or section: Sections 1-3; example for section 2, reach B
Event or design for which *n* is assigned: Flood Insurance Study for the 100-year design discharge

1. Describe channel (if needed draw sketch on back of sheet): Reach B has a low-water sand channel bounded by bedrock on one side and a sloping bar of gravel, cobbles, and boulders on the other. Section should be divided into segments - (1) bedrock, (2) sand, (3) gravel and cobble 1 to 6 inches in diameter, and (4) boulders 1 to 3 feet in diameter.

Does the use of the stream power relation indicate the vegetation (shrubs) will be laid over or remain in a relatively upright position (use flow chart in Figure 7.35 **and information in the previous section)?** The stream-power relation is not utilized as no vegetation is present in the channel.

2. Are present conditions representative of those during flood? Manning's n-value assigned for present conditions as no past flood information is available for this site.

3. Is roughness uniformly distributed across the channel? No **If no, on what basis should *n* for individual segments be weighted?** By wetter perimeter.

4. How will the roughness producing effects of the following roughness components be accounted for?
Bank roughness: Bedrock bank will be used as a separate segment
Bedrock outcrops: Not applicable
Isolated boulders: Add adjustment for 2 large boulders at start of reach
Bank roughness: Bedrock bank will be used as a separate segment
Vegetation: Not applicable
Obstructions: Not applicable
Meander: Not applicable

5. Refer to Table 7.12 **and** Table 7.13 **for example n-value computations.**

FIGURE 7.37
DIAGRAM OF HYPOTHETICAL CHANNEL SHOWING REACHES AND SEGMENTS
Used in Assigning n-values for Example 1

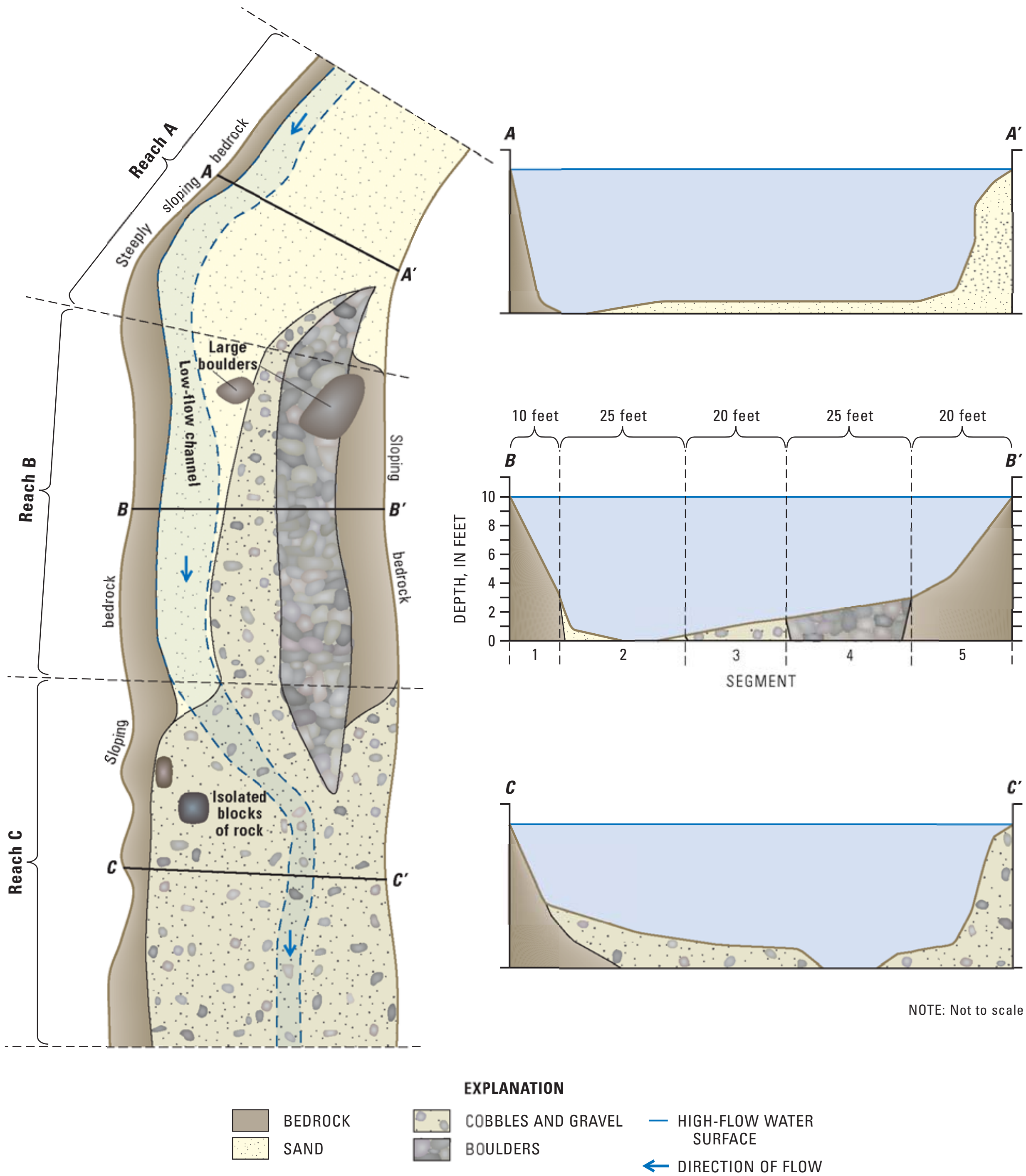


TABLE 7.12
COMPUTATION OF WEIGHTED MANNING'S n, Example 1

Segment number and material	Approximate dimensions (feet)		Wetted perimeter (feet)	Area (square -feet)	Median grain size (inches)	Base *n* for segment	Adjustments	Adjusted *n*	Weight factor	Adjusted *n* X weight factor
	Width	Depth								
(1) Bedrock	10	0-7	20		-	0.045	---	0.045	0.12	0.0054
(2) Sand	25	7-9	43		0.8mm ($^{1}/_{32}$)	0.025	---	0.025	0.25	0.0062
(3) Gravel	20	9-8	37		6	0.035	---	0.035	0.22	0.0077
(4) Boulders	25	8-7	40		24	0.050	---	0.050	0.23	0.0115
(5) Bedrock	20	7-0	30		-	0.045	---	0.045	0.18	0.0081
			Sum = 170						**Sum =** 1.00	**Sum =** 0.0389
									Weighted *n* = 0.039	

TABLE 7.13
ADJUSTMENTS, EXAMPLE 1

Factor	Describe conditions briefly	Adjustment
Banks	Included above	---
Channel alignment (curves and bends)	Bend in reach A causes some turbulence	+0.002
Changes in shape	Channel has fairly uniform shape within reach B	0
Obstructions	2 large boulders at upstream end of reach - add roughness	+0.002
Vegetation	Not used	0
Meander	Not used **Multiply by: -**	**Add: ---**
Other		
		Weighted n + added adjustments = 0.043
		Use *n* = 0.043

FIGURE 7.38
PROCESS FOR COMPUTATION OF MANNING'S n, *EXAMPLE 2*

Stream and location: See Figure 7.39
Reach or section: Sections 1-3; example for section 2, reach B
Event or design for which *n* is assigned: Flood Insurance Study for the 100-year design discharge

5. Describe channel (if needed draw sketch on back of sheet): Reach B has a low-water sand channel bounded by bedrock on one side and a sloping bar of cobbles on the other. Shrubs grow randomly throughout the channel. Flow depth is almost 2 times the height of the shrubs.

Does the use of the stream power relation indicate the vegetation (brush) will be laid over or remain in a relatively upright position (use flow chart in Figure 7.35 and information in the previous section)? Use of stream power relation indicates all the shrubs will be laid over as a result of the power of flow (see Table 7.14, Table 7.15 and).

6. Are present conditions representative of those during flood? The shrubs were probably laid over during flow.

7. Is roughness uniformly distributed across the channel? Yes **If no, on what basis should *n* for individual segments be weighted?** N/A

8. How will the roughness producing effects of the following roughness components be accounted for?
Bank roughness: Bedrock bank will be added under "adjustments"
Bedrock outcrops: Not applicable
Isolated boulders: Add adjustment for 2 large boulders at start of reach
Vegetation: Shrubs are randomly distributed in the channel
Obstructions: Not applicable unless mats of shrubs catch on the boulders
Meander: Not applicable

FIGURE 7.39
DIAGRAM OF HYPOTHETICAL CHANNEL SHOWING REACHES
Used in Assigning n-values for Example 2

TABLE 7.14
COMPUTATION OF WEIGHTED MANNING'S n, EXAMPLE 2

Segment number and material	Approximate dimensions (feet)		Wetted perimeter (feet)	Area (square-feet)	Median grain size (inches)	Base *n* for segment	Adjustments	Adjusted *n*	Weight factor	Adjusted *n* X weight factor
	Width	Depth								
			Sum =						**Sum =**	**Sum =**
									Weighted *n* =	

TABLE 7.15
ADJUSTMENTS, EXAMPLE 2

Factor	Describe conditions briefly	Adjustment
Banks and bed	Right bank is fairly smooth bedrock, similar in roughness to cobbles. d_{50} cobbles = 6"	+0.035
Channel alignment (curves and bends)	Bend in reach A causes some turbulence	+0.002
Changes in shape	Channel has a fairly uniform shape within reach B	0
Obstructions	2 large boulders at upstream end of reach - add roughness	+0.002
Vegetation	Stream power relation indicated vegetation impact on energy losses will be negligible	0
Meander	Not used **Multiply by: -**	**Add: ---**
Other		
		Weighted *n* + added adjustments = 0.039
		Used *n* = 0.039

FIGURE 7.40
COMPUTED STREAM POWER IMPACT ON THE VEGETATION-SUSCEPTIBILITY INDEX FOR SHRUBS

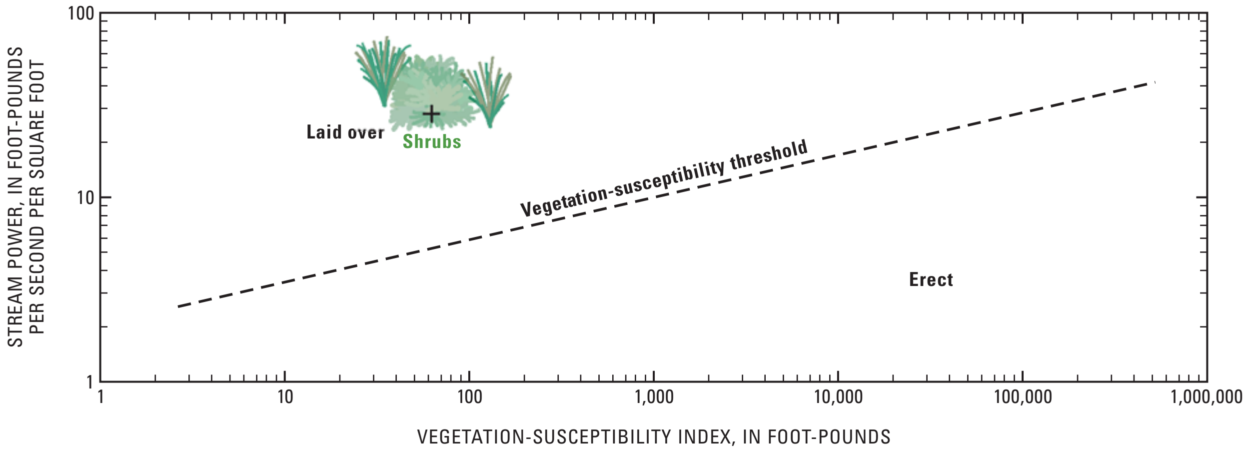


TABLE 7.16
VEGETATION CHARACTERISTICS, COEFFICIENTS, AND SUSCEPTIBILITY INDEX FOR SHRUBS

Vegetation Type	Shrubs
Average vegetation height, in feet	5
Vegetation-flexibility factor, V_{flex} (ft-lb)	15.5
Flow blocked by vegetation (percent)	30–70
Vegetation-blocking coefficient, $C_{blocking}$	4
Vegetation distributed randomly or parallel to flow	Randomly
Vegetation-distribution coefficient, C_{dist}	1
Ratio of hydraulic radius to average vegetation height	1.6
Flow-depth coefficient, C_{depth}	1
Vegetation-susceptibility index, $K_v = (V_{flex}C_{blocking}C_{dist}C_{depth})ft\text{-}lb = 62.0\ ft\text{-}lb$ (shrubs)	

TABLE 7.17
HYDRAULIC PARAMETERS USED TO COMPUTE STREAM POWER

[$(ft\text{-}lb/s)/ft^2$, foot-pounds per square foot]

Specific Weight of Water (lb/ft^3)	Hydraulic Radius (R) (ft)	Water Surface Slope (S_w) (ft/ft)	Mean Velocity (V) (ft/sec)
62.4	8	0.006	9.5
Stream Power, $SP = (62.4RS_wV) = 28.45(ft\text{-}lb/s)/ft^2$			

Step 5. Divide the channel width into segments according to general roughness (Jarrett, 1985; Thomsen and Hjalmarson, 1991). If distinct parallel bands of bed material of different particle sizes or of different roughness are present, use of segments can facilitate defining the contact between the different types of material (Figure 7.37). The dividing line between any two segments should parallel the general flow lines in the stream and should be located to represent the average contact between the differing types of material (Thomsen and Hjalmarson, 1991). The dividing line must extend through the entire reach, as defined in step 2, even if one of the different types of bed material may not be present throughout the entire reach. If a segment contains more than one type of roughness, it may be necessary to use an average size of bed material, which would apply in Figure 7.37 if the sand in segment 3 extended further downstream and the gravel and cobbles started closer to section 1. Figure 7.37 shows two distinct segments in reach B having material in the gravel- to boulder-size range. In the field, however, material of this size usually grades from fine-grained material at the edge of the sand channel to boulders near the shrub or vegetation line. In both instances, segments 3 and 4 should be combined as one segment. Where sand is mixed with gravel, cobbles, and boulders throughout a channel, it may be impractical to divide the main channel (Aldridge and Garrett, 1973).

Step 6. Determine the type of material that occupies each segment of channel, and determine the median particle size in each segment.

If the particles can be separated by size by screening, small samples of the bed material should be collected at 8 to 12 sites in the segment of the reach (Aldridge and Garrett, 1973). The samples are combined and the composite sample for the particular segment is passed through screens that divide the sample into a minimum of five size ranges. The volume or weight of material in each size range is measured and converted to a percentage of the total. The size or weight that corresponds to the 50^{th} percentile is obtained from a distribution curve developed by plotting particle size versus the percentage of the size smaller than that indicated (Phillips and Ingersoll, 1998).

If the material is too large to be screened, the median size of a random sample of the bed material in the segment is measured (Thomsen and Hjalmarson, 1991; Phillips and Ingersoll, 1998). Approximately 100 cobbles or boulders are sampled. For determination of d_{50}, particle diameter equals that of 50 percent of the particles.

Experienced personnel generally can make a fairly accurate estimate of the median particle size by inspection of the channel bed material if the range in particle size is small (Aldridge and Garrett, 1973).

Step 7. Determine the base value of *n* for each segment of channel using Table 7.2 or Table 7.3, equations (7.3) or (7.4), the comparisons made in step 1, or a combination of these. If a composite n-value is derived from segments, the user should proceed to step 8. If *n* is assigned for the channel as a whole, the user should go to step 11.

Step 8. Add adjustment factors from Table 7.4 that contribute to energy loss; these factors apply only to individual segments of the channel.

Step 9. Select the basis for weighting *n* for the channel segments. Wetted perimeter should be used for trapezoidal and U-shaped channels that have banks composed of one material and the channel bed composed of another. Wetted perimeter also should be used where the depth across the channel is fairly uniform. Weighting *n* for channel segments by area should be used where the depth varies considerably or where dense shrubs or trees occupy a large and distinct part of the channel (Aldridge and Garrett, 1973; Thomsen and Hjalmarson, 1991).

Step 10. Estimate the wetted perimeter or area for each segment and assign a weighting factor for each segment that is proportional to the total wetted perimeter or area. Multiply the *n* for each segment by its weighting factor, and divide the sum of the products by the sum of the weighting factors (Figure 7.38, Table 7.14 and Table 7.15) (Thomsen and Hjalmarson, 1991).

Step 11. Select the adjustment factors from Table 7.4 for conditions that influence *n* for the entire channel. Do not include adjustment factors for any items used in steps 7 and 8, and consider upstream conditions that may cause a disturbance in the study reach (Aldridge and Garrett, 1973). Add the adjustment factors to the weighted *n* from step 10 to derive the overall *n* for the reach being considered. When a multiplying factor for a meander is required, it is applied only after the other adjustments have been added to the base *n*. Repeat steps 3 through 11 for each additional reach when more than one reach is used for the hydraulic computations.

Step 12. Compare the n-values computed for the study reach with n-values estimated and verified for other channels (as discussed in step 1) to determine if the final values of *n* obtained in step 11 appear reasonable.

Step 13. Check the flow regime for all sand channels. Use the n-value from step 11 and the Manning's equation to compute velocity (equation (7.1)). Velocity, hydraulic radius, and water surface slope are then used to compute stream power. The flow regime is determined by utilizing information in Figure 7.2.

7.7 VEGETATION MAINTENANCE PLAN GUIDELINES

Vegetation has the ability to grow to significant heights and densities in a matter of a few years, and stream power may not be sufficient to alter vegetation in some stream channels. Homes and businesses have been built directly adjacent to some of these vegetated channels. If substantial amounts of mature vegetation are not included in n-value estimates in the initial design of the channel, then the vegetation may result in decreased channel conveyance and flood waters overtopping channel banks when design flows do occur.

In the past, vegetation may have been removed completely to ensure adequate conveyance of floodflows. In recent years, however, emphasis has shifted toward preservation of riparian vegetation that can provide habitat for wildlife, as well as aesthetically pleasing, multiuse areas for homeowners and businesses.

An engineering-based approach was used to develop vegetation-maintenance guidelines with the primary objective of optimizing the preservation of riparian habitat and to provide aesthetically pleasing multiuse areas for homeowners, while mitigating damage from floodflows along stream channels. The new guidelines described in subsequent sections of this document can be used as a tool for maintenance of vegetation and for development of vegetated channels. The new guidelines were developed for hydrologists, engineers, conservationists, and developers. To ensure that the guidelines are as robust as possible with respect to engineering design, the procedures used to develop these guidelines were based on a series of decisions that focus on selected values of Manning's *n*. Tables and photographs presented earlier in this report were used as the primary resource for selection of these roughness coefficients. Several case examples are presented at the end of this section, which should provide the user with a better understanding of the procedures defined in the guidelines.

7.7.1 Freeboard

Freeboard can be defined as an additional amount of conveyance area measured by using height above a flood level. The purpose of freeboard is to mitigate risk by providing a factor of safety. The flood level considered is normally the design water surface elevation computed for the design discharge, or the Base Flood Elevation (BFE) used for Flood Insurance Studies (FIS). The design water surface elevation is used to describe both situations. For the purposes of example cases 2, 3, and 4 at the end of this section, the minimum amount of freeboard required above the design water surface elevation is 1 foot. An alternate vegetation-maintenance process is illustrated in example case 1, in which freeboard is not considered. Channel banks are not levied in any of the example cases.

The importance of maintaining the minimum factor of safety is significant; therefore, vegetation management and maintenance plans should adhere to maintaining the minimum required freeboard. Vegetation can grow quickly, which can cause channel conveyance to decrease and freeboard, or the factor of safety, to diminish or be consumed completely. This is the primary purpose for making periodic inspections of vegetation conditions.

Ideally, for stream channels with newly computed BFEs and void of all vegetation, Manning's n-values are adjusted according to the amount of vegetation anticipated for future conditions (TABLE 7.4). For example, a newly constructed channel that has a firm earth base and concrete banks requires assessment of current roughness factors, including those for future vegetation conditions. If a Manning's n-value of 0.030 is selected for a channel void of vegetation, and it would be desirable to allow mesquite to grow to a density of approximately 1 tree per 100 feet of channel; the adjusted vegetation component may be in the range of 0.025 to 0.050. The vegetation conditions and corresponding n-value should not increase above the design value, or freeboard, may be partially or completely lost.

For channels that were originally designed under no-vegetation conditions and for which future-vegetation conditions were not taken into account, only flexible grasses and other types of vegetation determined to layover during design flows should be allowed to grow within the channel. Any vegetation that may decrease velocity, and consequently increase design flow area, should be considered for removal from the channel.

Examples of Guideline Use

Stream channels that are addressed in the example cases include trapezoidal-shaped channels for which the original design is for zero-vegetation influences on *n*, and current and future-vegetation conditions are included in the original design.

Alternate vegetation-thinning criteria developed prior to methods developed and described in this document are used in example case 1. The alternate criteria are used to illustrate the need to address Manning's n-value and freeboard when maintaining vegetation and developing vegetation-maintenance plans. The vegetation-maintenance plans presented in example cases 2, 3, and 4 use thinning criteria on the basis of Manning's *n* and freeboard according to guidelines suggested in this document.

Example Case 1

The use of, and the rationale for, the new guidelines can be illustrated by examining alternative vegetation-maintenance activities in a constructed channel (Figure 7.41 A and B). The constructed flood-control channel originally was designed for no or very sparse vegetation conditions. Subsequently, however, growths of mesquite, palo verde, and shrubs such as desert broom have grown to large spatial densities and to heights that surpass the flood-channel banks (Figure 7.41 *A*). Owing to a growing concern that channel conveyance has been reduced and flood banks could be overtopped by the design discharge, local representatives determined that vegetation in the wash needed to be maintained or thinned (Figure 7.41 *B*). By using thinning criteria that was primarily based on tree height and trunk diameter, shrubs and smaller palo verde and mesquite were removed and lower branches on the remaining palo verde and mesquite trees were trimmed to allow for greater channel conveyance. USGS staff, by using information acquired before and after maintenance, determined the roughness coefficients for five surveyed cross sections of the channel (Table 7.18). Manning's n-values were selected for initial, pre- and post-vegetation conditions on the basis of information contained in Table 7.4. The U.S. Army Corps of Engineers Hydrologic Engineering Centers River Analysis System (HEC-RAS) was used to simulate water surface elevations for the channel under the various vegetation conditions. The step-backwater computer simulations were run by using the design discharge, the channel geometry from the surveyed sections, and the selected roughness coefficients (A, B, and *C*). When the channel was originally designed, it appears there would have been adequate freeboard (*A*). Simulation results using HEC-RAS, however, indicate that the design discharge for the channel for full-grown vegetation conditions would overtop channel banks and flood adjacent areas (*B*). Velocities would be slowed significantly compared to initial channel conditions and cross section area would compensate with a rise in water-surface elevations by an average of 3.92 feet (Table 7.18). Simulations conducted for post-vegetation maintenance conditions indicate that the design discharge would remain within most of the channel. Because of the thinning criteria, many large trees were left within the reach from sections 3 to 5. The remaining cluster or clump of trees resulted in selection of larger roughness coefficients for this area of the study reach. Consequently, the design discharge overtopped the right channel bank at sections 4 and 5 (*C*). Additionally, the simulated water surface removed all conveyance that should have been available for freeboard. Use of the guidelines in this report would have resulted in more vegetation being removed, lower roughness coefficients, and larger conveyance, allowing design flow to remain below required freeboard levels.

FIGURE 7.41
CONSTRUCTED CHANNEL THAT REQUIRED VEGETATION MODIFICATION
(*A*, Before maintenance of vegetation, July 28, 2005. *B*, after maintenance of vegetation, August 3, 2005.)

A. Before maintenance

B. After maintenance

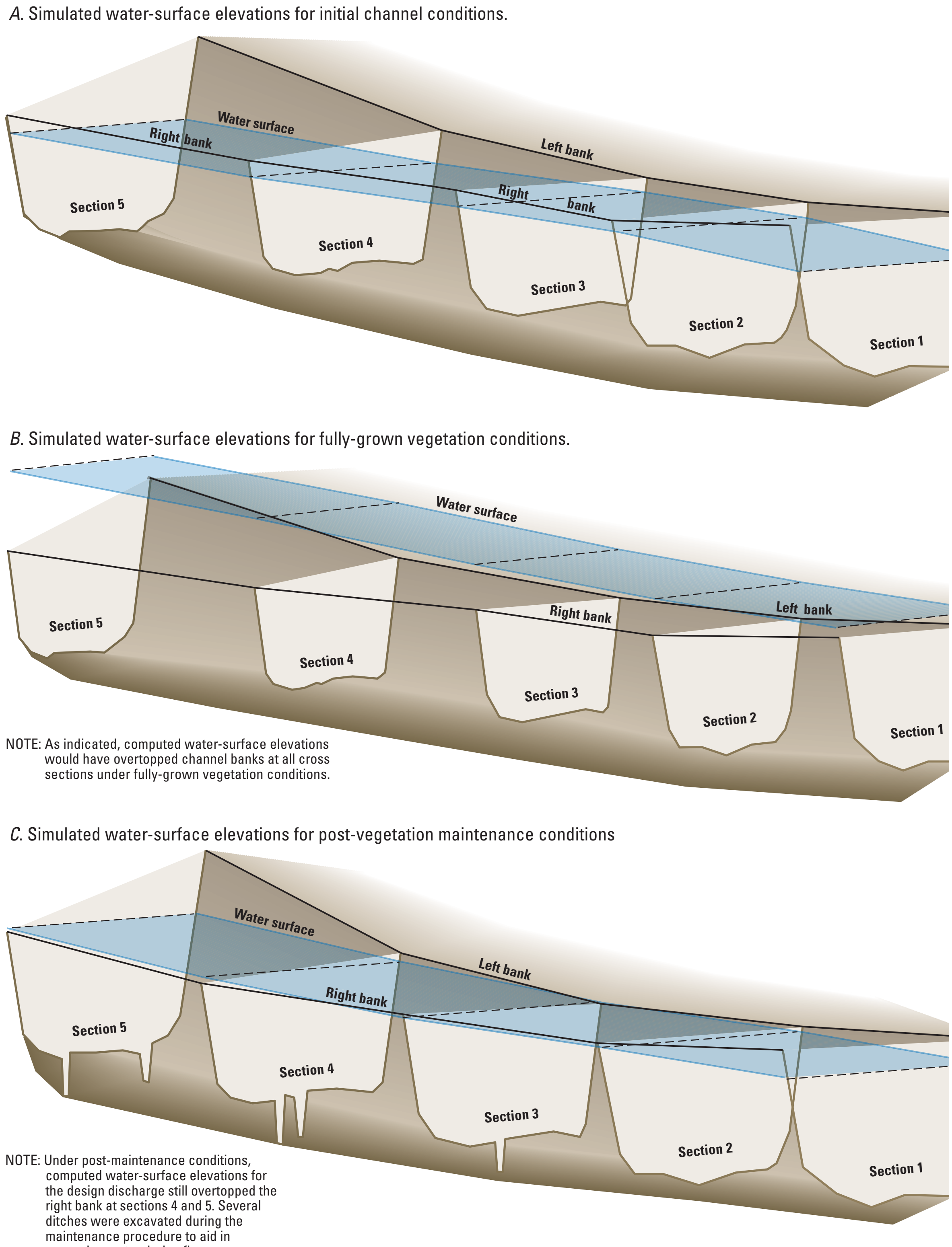
TABLE 7.18
HYDRAULIC PROPERTIES OF FLOW FOR THE CONSTRUCTED CHANNEL IN EXAMPLE CASE 1
(Velocity, area, and water surface elevations were computed by using estimated Manning's n-values and a design discharge. [See for sections and simulated water surface elevations. ft, feet; ft/s, feet per second; ft^2, square feet])

Cross Section No.	Manning's Value	Velocity (ft/s)	Area (ft)	Water Surface Elevation, Arbitrary Datum (ft)
Initial conditions (void of vegetation)				
1	0.028	11.86	320	13.00
2	0.028	9.56	325	14.20
3	0.028	10.80	367	14.29
4	0.028	12.17	325	14.62
5	0.028	12.37	320	15.88
Pre-vegetation maintenance conditions (vegetation fully grown)				
1	0.080	7.12	557	16.00
2	0.080	6.41	617	16.90
3	0.100	6.35	623	17.80
4	0.150	5.81	681	19.46
5	0.150	5.53	716	21.41
Post-vegetation maintenance conditions				
1	0.035	9.76	406	14.00
2	0.035	8.81	450	14.67
3	0.040	9.30	426	15.04
4	0.050	9.21	430	15.87
5	0.060	9.61	412	17.07

FIGURE 7.42

SIMULATED WATER SURFACE ELEVATIONS FOR CONSTRUCTED CHANNEL

in Example Case 1: *A*, Initial channel conditions. *B*, Fully-grown vegetation conditions. *C*, Post-vegetation maintenance conditions



Example Case 2

For example case 2, consider a constructed trapezoidal-shaped channel that originally was void of any vegetation. The initial Manning's n-value selected was 0.030 and an additional 0.015 was estimated for future conditions when vegetation is anticipated to grow in the bed of the channel. An *n*-value of 0.045 was used for the final design computations, allowing 1 foot of additional conveyance or freeboard. A vegetation maintenance plan was established on completion of the channel. Over the next 10 years, however, the vegetation assessment and maintenance plan was neglected and forgotten. After 10 years, mesquite rooted in the channel substrate and grew to a height that averaged 16 feet, surpassing the height of the channel banks. Furthermore, shrubs took root that averaged about 5 feet in height. The average amount of mesquite that blocks flow is approximately 60 percent, and the approximate amount of shrubs that blocks flow is 20 percent. According to information in Table 7.4, Manning's *n* for the design flow increased from the initial composite value of 0.045 to a range of 0.100 to 0.200 (average 0.150). According to standard-step simulations, the channel no longer has adequate conveyance to carry the design flow, thus freeboard will be lost and banks will be overtopped when a design flow occurs (Figure 7.43 A and B).

It would seem that significant thinning of the vegetation is now warranted. Before any maintenance activities are engaged, however, the stream power relation should be utilized to determine if the design flow has the power to lay over the shrubs and possibly the mesquite.

Values acquired in the field needed to compute the vegetation-susceptibility index for the shrubs and mesquite are given in Table 7.19. Hydraulic values acquired from the HEC-RAS simulations for peak design flow were used for the stream-power computations (Table 7.20). The resultant values for each vegetation type are then plotted with corresponding stream power (Figure 7.44). As indicated, although shrubs plot close to the threshold, both shrub and mesquite need to be considered for thinning to decrease the Manning's n-value for the vegetation component back to its original value of 0.015 (or a composite n-value of 0.045).

For this example case, the Manning's n-value for the vegetation component should be no more than 0.015, which allows for a select amount of shrubs and trees to remain in the channel (Table 7.4). There are many vegetation maintenance schemes or scenarios that could be developed to meet the criteria for freeboard. For example case 2, however, only two vegetation-maintenance scenarios are presented. For scenario 1, native vegetation was left randomly distributed to diminish the potential additive effect of the sphere of influence for turbulence on flow caused by the vegetation (Figure 7.45; Table 7.4). Scenario 1 may be more aesthetically pleasing to local residents (Figure 7.46). For scenario 2, native mesquite trees and some shrubs would be clumped where possible (Figure 7.47). Clumping the vegetation may present a better habitat environment for wildlife (Figure 7.52). The additive strength of clumped vegetation will make it much more resistant to flow and, therefore, could be a good method for protecting vegetation from the power of flow. The trees for scenario 2 should be arranged or maintained so that there is one clump per three cross section lengths of channel to ensure spheres of influence do not overlap (Figure

7.47). These procedures should allow the vegetation component of Manning's n to be approximately 0.015 for this constructed channel (Table 7.4).

A new maintenance plan should be enacted that includes periodic inspection or assessment of vegetation conditions. Maintenance of vegetation should be conducted if deemed necessary.

FIGURE 7.43
HEC-RAS COMPUTER SIMULATIONS FOR CONSTRUCTED CHANNEL IN EXAMPLE CASE 2
(*A*, Original design computed water surface elevation for n = 0.045 (base n-value = 0.030 and future vegetation n-value component of 0.015). *B*, Water surface elevation for a fully vegetated channel at an average n = 0.150.)

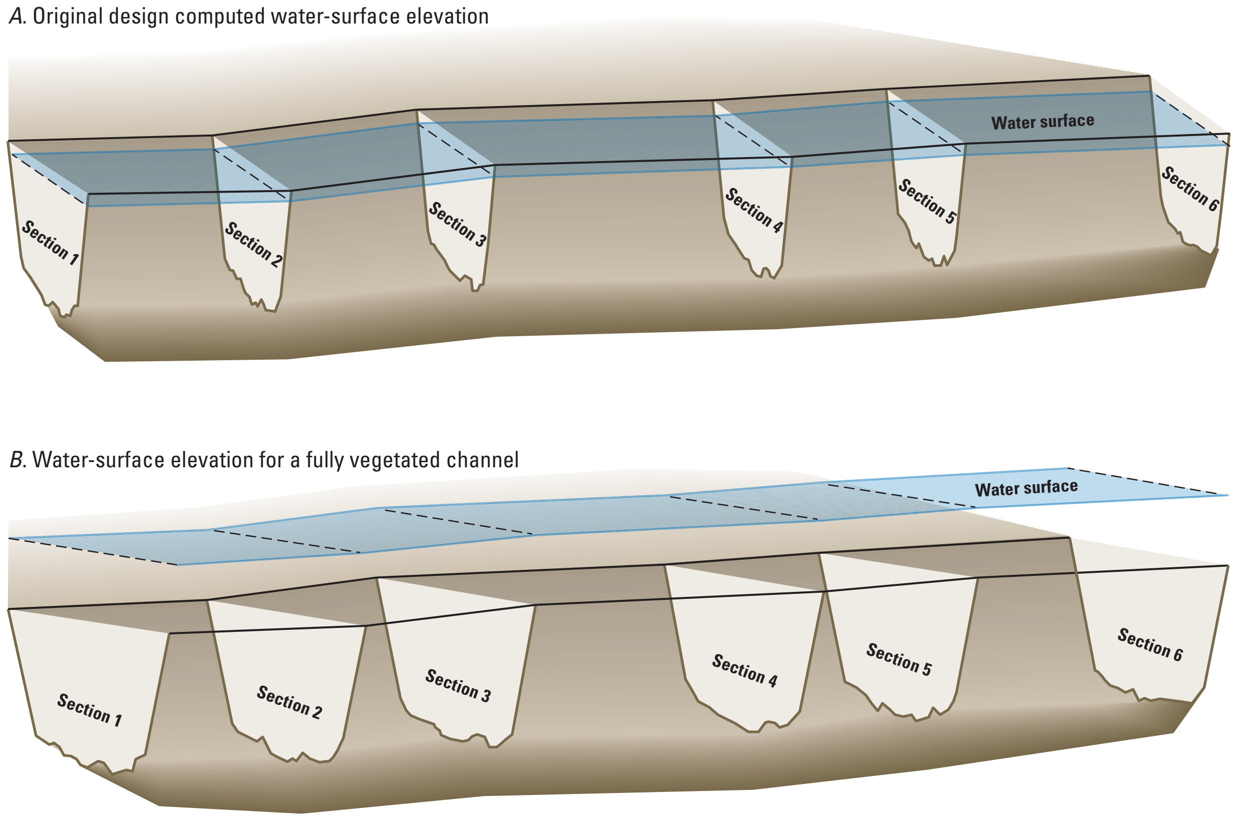


TABLE 7.19
VEGETATION COEFFICIENTS AND SUSCEPTIBILITY INDEX FOR SHRUBS AND MESQUITE
Example Case 2, [$ft\text{-}lb$, foot-pound]

Vegetation Type	Shrubs	Mesquite
Average vegetation height, in feet	5	16
Vegetation-flexibility factor, V_{flex} (ft-lb)	15.5	830
Flow blocked by vegetation (percent)	< 30	30–70
Vegetation-blocking coefficient, $C_{blocking}$	1	4
Vegetation distributed randomly or parallel to flow	Randomly	Randomly
Vegetation-distribution coefficient, C_{dist}	1	1
Ratio of hydraulic radius to average vegetation height	1.2	0.4
Flow-depth coefficient, C_{depth}	3	20
Vegetation-susceptibility index, $K_v = (V_{flex}C_{blocking}C_{dist}C_{depth}) = 46.5\,ft\text{-}lb$ (shrubs) and $66,400\,ft\text{-}lb$ (mesquite)		

TABLE 7.20
HYDRAULIC PARAMETERS USED TO COMPUTE STREAM POWER

Example Case 2,[$(ft\text{-}lb/s)/ft^2$, foot-pounds per second per square foot]

Specific Weight of Water (lb/ft^3)	Hydraulic Radius (R) (ft)	Water Surface Slope (S_w) (ft/ft)	Mean Velocity (V) (ft/sec)
62.4	6	0.003	3
Stream Power, $SP = (62.4RS_wV) = 3.37(ft\text{-}lb/s)/ft^2$			

FIGURE 7.44
IMPACT OF COMPUTED STREAM POWER FOR SHRUBS AND MESQUITE
Example Case 2

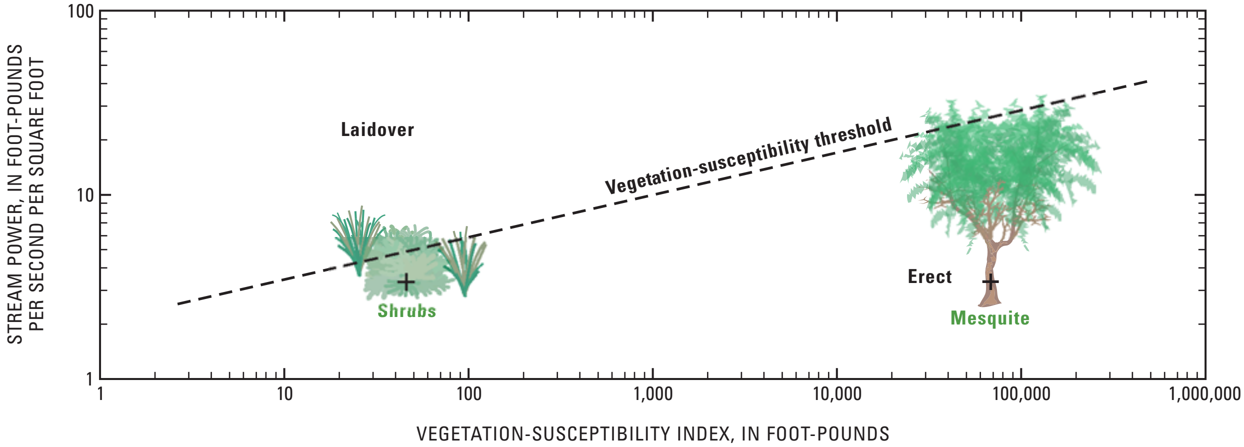


FIGURE 7.45
PLAN VIEW ILLUSTRATION OF CONSTRUCTED CHANNEL FOR EXAMPLE CASE 2, SCENARIO 1

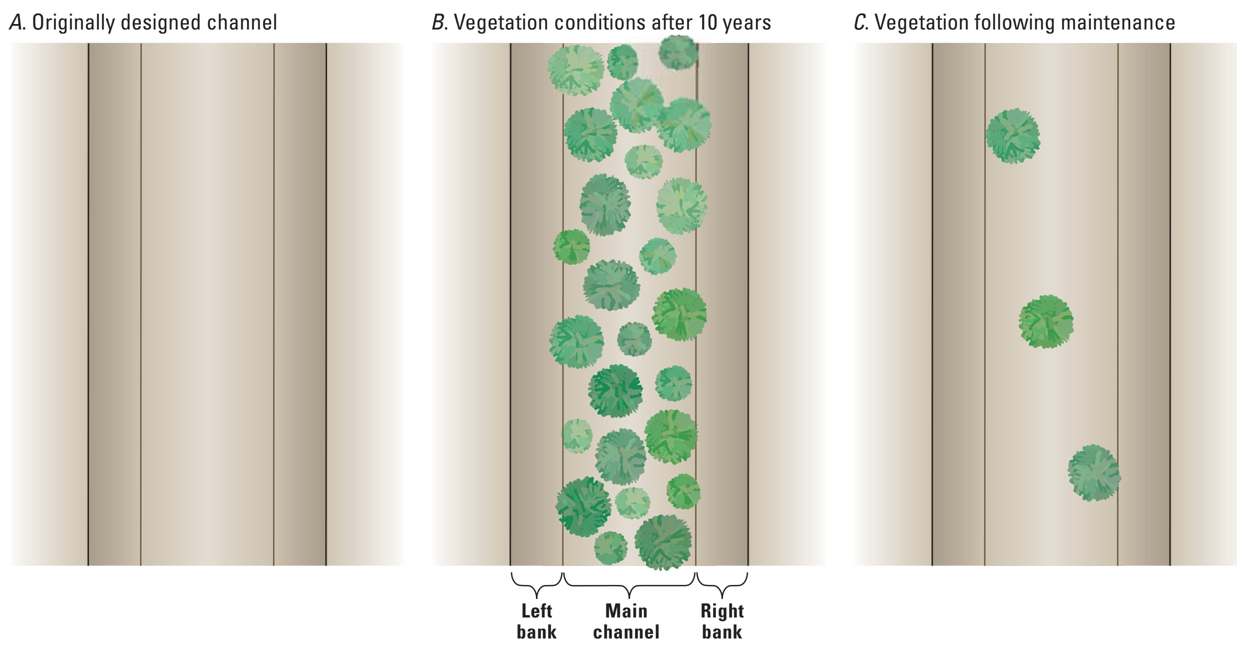


FIGURE 7.46
A CONSTRUCTED CHANNEL FOR EXAMPLE CASE 2, SCENARIO 1
(*A*, Manning's composite roughness coefficient is estimated to be 0.150 prior to vegetation maintenance. *B*, Energy loss components subsequent to vegetation maintenance. *C*, Vegetation conditions approximately six months following maintenance.)

A. Before maintenance

B. After maintenance

C. Approximately six months following maintenance

FIGURE 7.47
PLAN VIEW ILLUSTRATION OF CONSTRUCTED CHANNEL FOR EXAMPLE CASE 2, SCENARIO 2
(Following vegetation maintenance, trees are clumped together primarily to provide better habitat for wildlife.)

FIGURE 7.48
A CONSTRUCTED CHANNEL FOR EXAMPLE CASE 2, SCENARIO 2
(*A*, Manning's composite roughness coefficient is estimated to be 0.150 prior to vegetation maintenance. *B*, Manning's composite roughness coefficient is estimated to be 0.045 subsequent to vegetation maintenance.)

A. Before maintenance

B. After maintenance

Example Case 3

The vegetation-maintenance plan considered in example case 3 is for a gravel- and cobble-bed, straight uniform channel that recently had experienced a high-flow event. Following the event, a few palo verde trees in the channel remained in an upright position, and had fairly significant amounts of debris on the upstream side. Shrubs were evident in the reach prior to the flow (Figure 7.49), but laidover and/or removed during the event (Figure 7.50). The area adjacent to this channel (right and left banks) was designated for a new housing and business development. A FIS was conducted prior to development. A base Manning's n-value of 0.033 was selected for the cobble substrate (Figure 7.50). The vegetation component of Manning's *n* selected for the few standing palo verde trees was selected to be 0.020 (Figure 7.51). No vegetation-component addition was made to account for future growth of shrubs or other vegetation, and no other components of *n* were believed to contribute to energy losses within the channel. Total composite *n*, therefore, for the hydraulic computations was 0.053. The FIS indicated that the previous flow event approximated a statistical 25-year flow. A 100-year design flow was determined, and the BFE was computed with 1 foot of freeboard (Figure 7.52). Within 1 year of completion of the FIS, homes and business were constructed adjacent to the channel.

After a 5-year period, no additional trees grew in the channel; however, shrubs grew throughout the channel to a density greater than about 70 percent and heights averaging 5 feet (Figure 7.53). Because the originally selected n-value of 0.053 did not account for future growth of vegetation in the channel, and homes and business were constructed immediately adjacent the channel, there was concern the channel may no longer be capable of conveying the design discharge (Figure 7.54). Vegetation maintenance was considered; however, first the stream-power relation was used to determine if the shrubs would be fully laidover on the rising limb of the 100-year design flood hydrograph.

From standard-step computations made after the 5-year period, and a Manning's n-value for present conditions (selected to be 0.083), values were acquired for computation of stream power. For shrubs and palo verde averaging 5 and 16 feet in height, respectively, the vegetation flexibility factor is 15.5 and 3,848 ft-lbs, respectively. The percent of flow blocked by the shrubs is estimated to be greater than 70 percent, while the palo verde is estimated at less than 30 percent. The vegetation blocking coefficients, therefore, are 9.0 for shrubs and 1.0 for palo verde. The palo verde and shrubs are randomly distributed in the main channel. The vegetation distribution coefficient, therefore, is 1.0 for palo verde and shrubs. From the standard-step computations, hydraulic radius is equal to 3.6 feet. The flow-depth ratio, therefore, is 0.7 and 0.2 for shrubs and palo verde, respectively. Hence, the flow-depth coefficient is 5.0 and 60, respectively. The vegetation-susceptibility index is 698 for the shrubs, and 231,000 ft-lbs for the palo verde (Table 7.21 and Table 7.22).

Subsequently, stream power was computed and plotted with the vegetation-susceptibility indices for the shrub and palo verde (Figure 7.55). According to their plotting positions, the shrubs would be laidover on the rising limb of the 100-year flow hydrograph. Thus, the roughness component that represents the shrub can be considered negligible and not be added to the composite n-value. The palo verde, however, probably would remain in an upright position. The impact of the palo verde on total roughness was included in the original FIS when the BFE was determined. It was determined that it should not be maintained. For this example case, the guidelines indicate that shrubs also should not be maintained, and a vegetation assessment and maintenance plan should be enacted to periodically document any noticeable future changes in vegetation conditions.

FIGURE 7.49
EXAMPLE OF A CHANNEL WITH RANDOMLY DISTRIBUTED SHRUBS
Prior to the Statistical 25-year Event

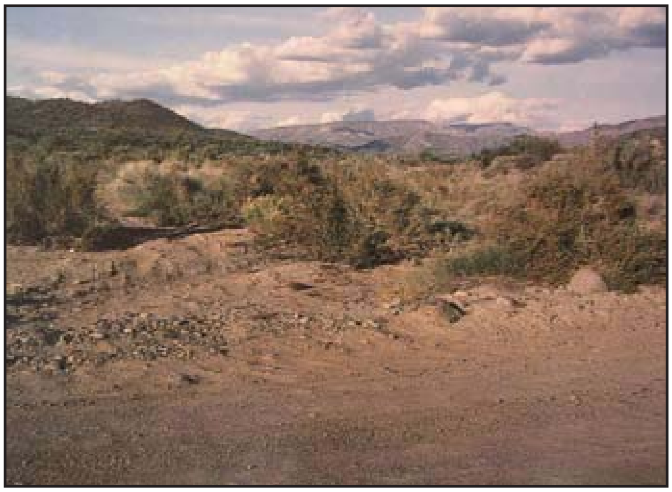


FIGURE 7.50
EXAMPLE OF A CHANNEL WITH REMOVED SHRUBS
Following the Statistical 25-year Event

FIGURE 7.51
LOCATIONS AND DENSITY OF PALO VERDE TREES IN THE CHANNEL
for Example Case 3 Following the 25-year Flow Event

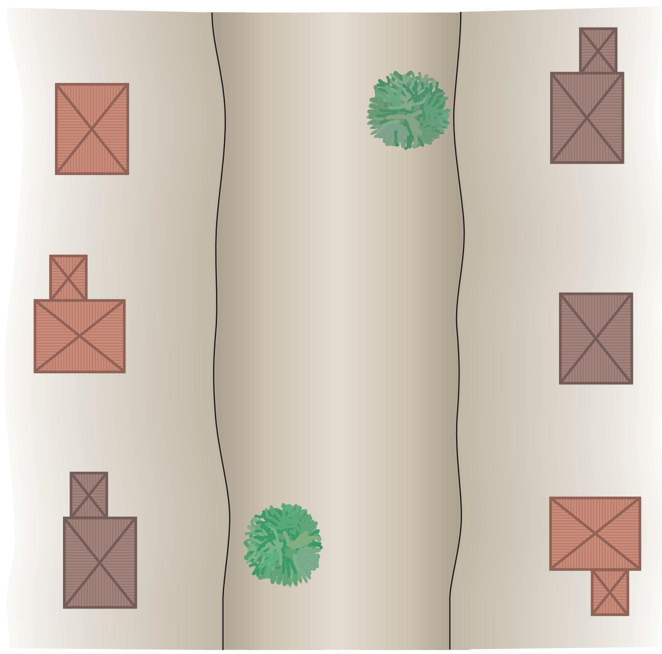


FIGURE 7.52
BASE FLOW ELEVATION (BFE) WITH 1 FOOT OF FREEBOARD
for the Channel Used in Example Case 3 for a 100-year Design Flood

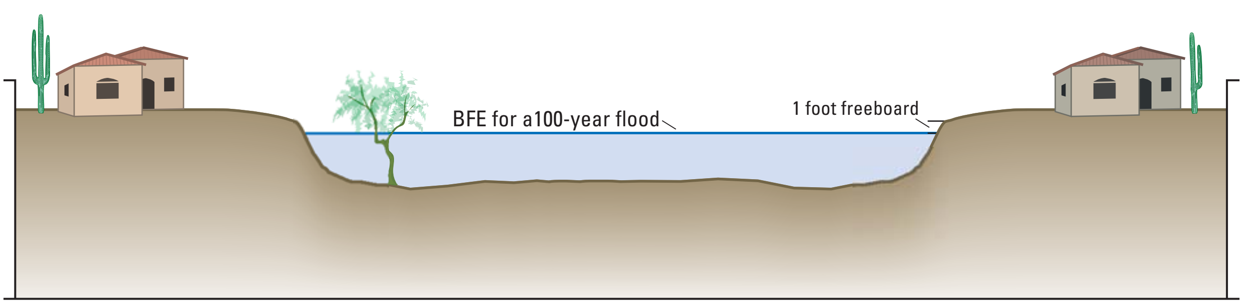


FIGURE 7.53
EXAMPLE OF A CHANNEL WITH A VEGETATION DENSITY GREATER THAN 70 PERCENT
Five Years After a Flood Insurance Study

FIGURE 7.54
EXAMPLE CHANNEL WITH DEVELOPMENT AND FIVE YEARS VEGETATION GROWTH
(*A*, Distribution of shrubs and trees in main channel and approximate location of homes.
B, Shrubs (smaller circles) and palo verde (larger circles) and homes along channel.)

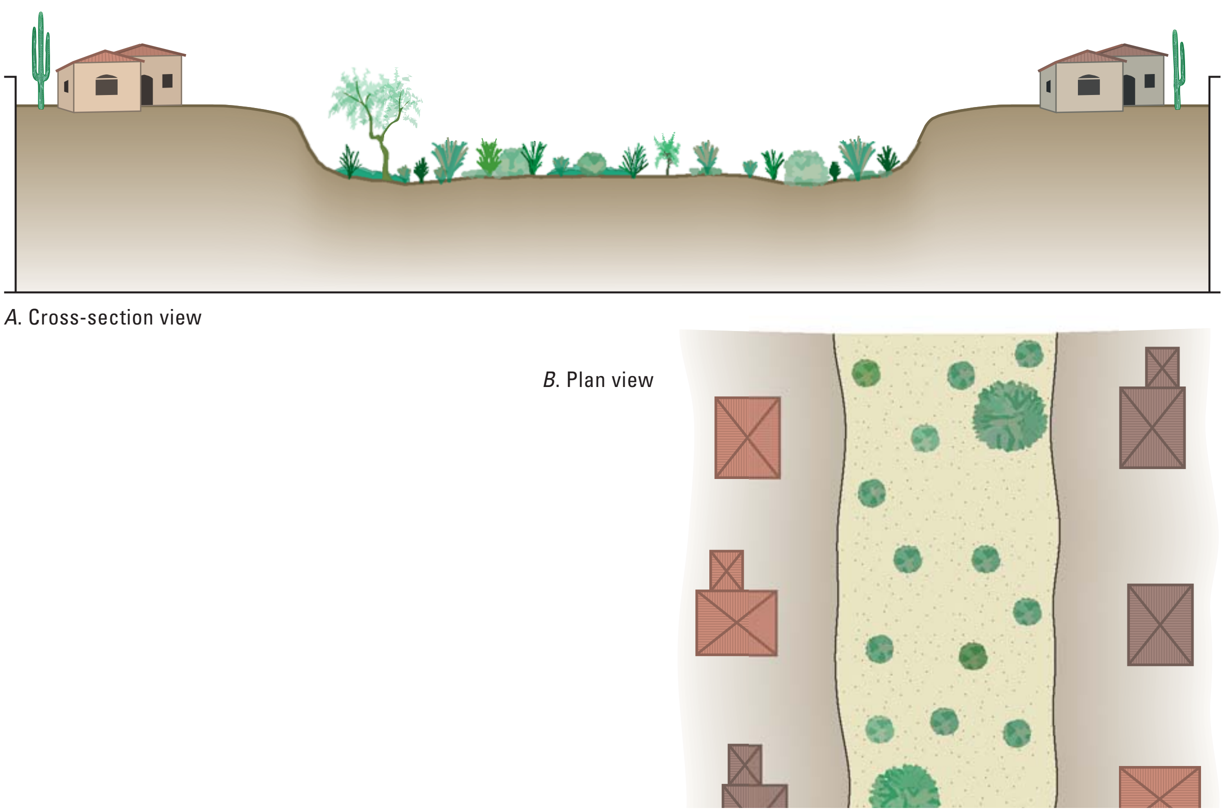


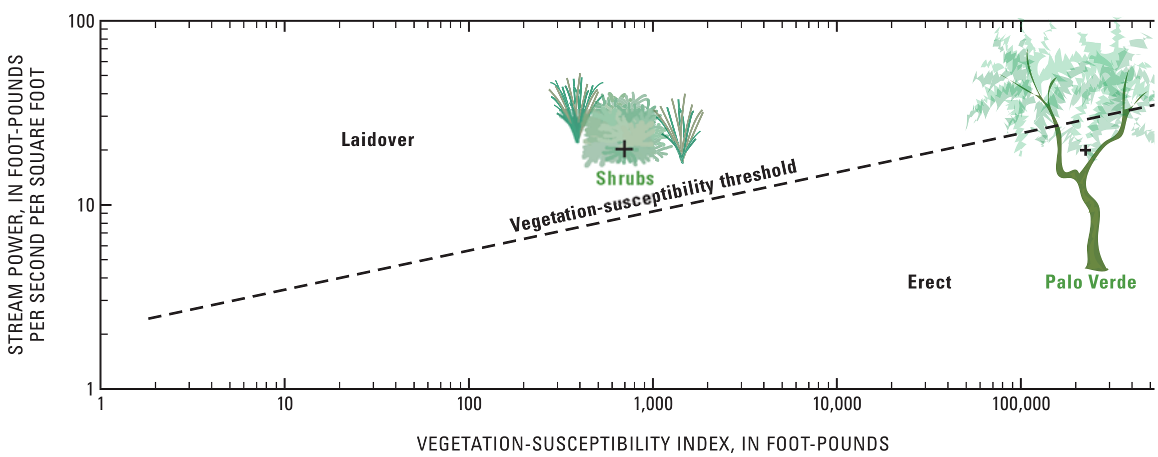
TABLE 7.21
VEGETATION COEFFICIENTS AND SUSCEPTIBILITY INDEX FOR SHRUBS AND PALO VERDE
Example Case 3, [$ft\text{-}lb$, foot-pound]

Vegetation Type	Shrubs	Palo Verde
Average vegetation height, in feet	5	16
Vegetation-flexibility factor, V_{flex} (ft-lb)	15.5	3,848
Flow blocked by vegetation (percent)	> 70	< 30
Vegetation-blocking coefficient, $C_{blocking}$	9	1
Vegetation distributed randomly or parallel to flow	Randomly	Randomly
Vegetation-distribution coefficient, C_{dist}	1	1
Ratio of hydraulic radius to average vegetation height	0.7	0.2
Flow-depth coefficient, C_{depth}	5	60
Vegetation-susceptibility index, $K_v = (V_{flex}C_{blocking}C_{dist}C_{depth}) = 698\ ft\text{-}lb$ (shrubs) and $231,000\ ft\text{-}lb$ (palo verde)		

TABLE 7.22
HYDRAULIC PARAMETERS USED TO COMPUTE STREAM POWER
Example Case 3, [$(ft\text{-}lb/s)/ft^2$, foot-pounds per second per square foot]

Specific Weight of Water (lb/ft^3)	Hydraulic Radius (R) (ft)	Water Surface Slope (S_w) (ft/ft)	Mean Velocity (V) (ft/sec)
62.4	3.6	0.009	10
Stream Power, $SP = (62.4RS_wV) = 20.2(ft\text{-}lb/s)/ft^2$			

FIGURE 7.55
IMPACT OF COMPUTED STREAM POWER FOR SHRUBS AND PALO VERDE



Example Case 4

The vegetation-maintenance plan considered in case 4 is for a planned residential community that is built adjacent to a gravel- and grass-lined channel. A low-flow channel was constructed, which winds through the bottom of the main channel in this multiuse area. Small mesquite trees were planted along the side of the low-flow channel to a density of about 2 to 3 trees per cross section length of reach (Figure 7.56 A and B). Current and future conditions for the planted mesquite trees were considered in the selection of the vegetation component of Manning's *n* for the FIS. A base n-value of 0.027 was selected for the gravel- and grass-lined channel; the vegetation component selected for the mesquite for present conditions was 0.015. An estimated 0.025 was added to the component for the mesquite to account for future growth (Table 7.4). The total composite n-value for the design discharge for the FIS, therefore, was 0.067. A freeboard of 1 foot was added to the BFE, and the homes were subsequently constructed. The original engineers and developers initiated a vegetation assessment and maintenance plan to ensure that roughness coefficients would not exceed the design n-value of 0.067. The assessment plan, however, was neglected, and after 5 years many additional mesquite trees had taken root.

After 10 years the mesquite trees were mature, and the area maintained a density of approximately 6 to 7 mesquite trees per cross section length of reach (Figure 7.56 C). The mesquite trees averaged 16 feet in height. Homeowners generally were pleased with the aesthetic value of the dense and mature mesquite in the multiuse area, however, others, including the local floodplain manager, were concerned that the design discharge would result in the loss of available freeboard and overtop channel banks. The new estimated composite roughness coefficient was in the range of 0.100 to 0.200 (Table 7.4).

The stream-power relation was used to determine impact of the design discharge on the mesquite. For mesquite trees averaging 16 feet in height, the vegetation flexibility factor is 830 ft-lbs. The amount of flow blocked by these trees is about 60 percent. Standard-step HEC-RAS computations were run for the channel with an n-value that averaged 0.150. Velocity and hydraulic radius were acquired from these computations to determine the remaining vegetation-susceptibility index components (Table 7.23). Values used for computation of stream power also were acquired from the standard-step computations (Table 7.24). Stream power and the vegetation-susceptibility index were plotted for mesquite to determine if flow would have any impact on the vegetation (Figure 7.57).

The mesquite trees probably would not be altered by a 100-year design flow for this multiuse area (Figure 7.57). The design flow would, therefore, overtop the channel banks considerably (Figure 7.56 C and D). A substantially larger flow would be required to alter the mesquite trees. In order to maintain the original BFE and 1-foot freeboard, all mesquite trees should be removed except those originally planted for which future growth was considered when n-values were selected for the original FIS (Figure 7.56 E and F). The vegetation assessment and maintenance plan should be followed closely to ensure that estimates of Manning's *n* do not again exceed 0.067.

Figure 7.56

Plan View and Cross Section Views Showing Distribution of Mesquite

(in the main channel as simulated water surface elevation, and location of homes. *A*, Plan view after mesquite trees were initially planted. *B*, cross section view of simulated water surface for the design discharge for initial condition. *C*, Plan view showing mesquite trees after 10 years of growth. *D*, cross section view of simulated water-surface for the design discharge for vegetation conditions after 10 years of growth. *E*, Plan view showing remaining mesquite trees following vegetation maintenance. *F*, cross section view of simulated water surface for the design discharge for post-maintenance conditions.)

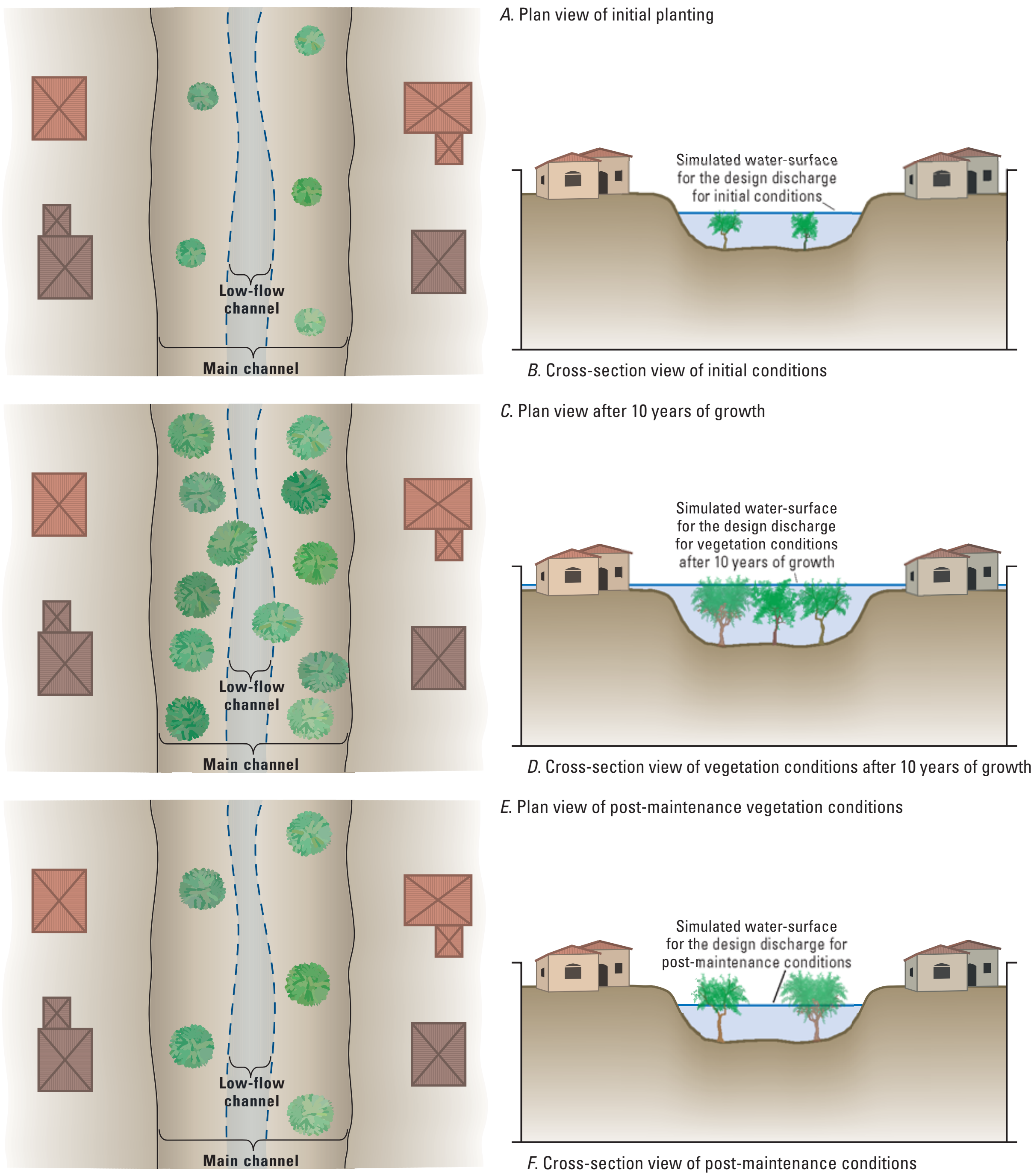


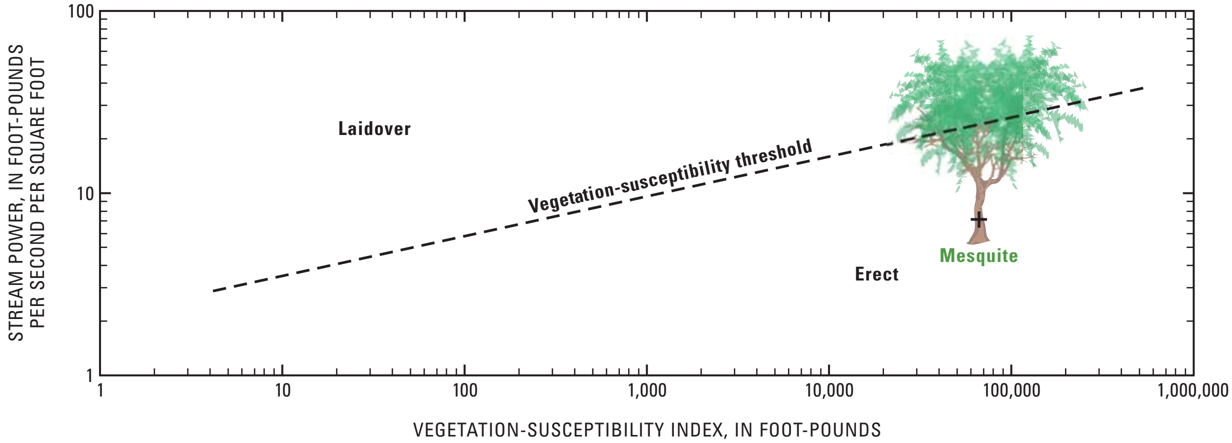
TABLE 7.23
VEGETATION COEFFICIENTS AND SUSCEPTIBILITY INDEX FOR MESQUITE
Example Case 4, [$ft\text{-}lb$, foot-pound]

Vegetation Type	Mesquite
Average vegetation height, in feet	16
Vegetation-flexibility factor, V_{flex} (ft-lb)	830
Flow blocked by vegetation (percent)	60
Vegetation-blocking coefficient, $C_{blocking}$	4
Vegetation distributed randomly or parallel to flow	Randomly
Vegetation-distribution coefficient, C_{dist}	1
Ratio of hydraulic radius to average vegetation height	0.6
Flow-depth coefficient, C_{depth}	20
Vegetation-susceptibility index, $K_v = (V_{flex}C_{blocking}C_{dist}C_{depth}) = 64,400\ ft\text{-}lb$	

TABLE 7.24
HYDRAULIC PARAMETERS USED TO COMPUTE STREAM POWER
Example Case 4, [$(ft\text{-}lb/s)/ft^2$, foot-pounds per second per square foot]

Specific Weight of Water (lb/ft^3)	Hydraulic Radius (R) (ft)	Water Surface Slope (S_w) (ft/ft)	Mean Velocity (V) (ft/sec)
62.4	9	0.003	5
Stream Power, $SP = (62.4RS_wV) = 8.42(ft\text{-}lb/s)/ft^2$			

FIGURE 7.57
IMPACT OF COMPUTED STREAM POWER FOR MESQUITE TREES



7.7.2 Vegetation Assessment and Maintenance Plan Outline

The intent of this section is to provide an outline that can be used as the minimum required information for a vegetation assessment and maintenance plan. Unlined constructed channels that rely on a range of n-values to meet design scour, deposition and freeboard requirements should have such a plan. The vegetation conditions should be monitored on a periodic basis, with the period being specified in the initial plan. Each time a periodic inspection is made a report form containing the new information should be added to the initial plan to document the history of morphology of the channel.

1. Site. ______________
2. Date. ______________
3a. Initial visit. ___ (y/n). 3b. If no, visit number. ___
4. Photograph (if available) and plan view sketch of initial conditions.
5. Photograph and plan view sketch of current conditions.
6. Initial Manning's n-value used to delineate design-flow elevations. _____
7. Current estimated Manning's n-value. _____
8. Survey or observe channel substrate. If aggradation or degradation has occurred in the reach, a new survey of cross sections may be necessary.
9. Document any channel migration or bank erosion.
10. After assessment with stream power relations, are current Manning's n-values outside the target range? ___ (y/n)

11. If yes, describe plan to bring Manning's n-values back to original n-value. If no, briefly describe rationale for the decision and recommendations for future years.
12. Sketch (plan view) of vegetation maintenance plan (if necessary, flag trees and brush to be removed and trees that require trimming).
13. Photograph and sketch (plan view) of channel and vegetation conditions following maintenance.

7.8 SUMMARY AND CONCLUSIONS

Hydraulic computations of open channel flow require evaluation of the channel's resistance to flow, which typically is represented by a roughness parameter. The Manning's roughness coefficient, *n*, commonly is used to represent flow resistance. Verified and estimated Manning's roughness coefficients for natural and constructed stream channels in Arizona have been presented in several previously published documents. Most of the information from which is available in the form of guidelines, tables, figures, and examples.

Proper estimation of n-values for open channels in arid to semi-arid environments can present difficulties in estimating channel resistance. In particular, vegetation in ephemeral and intermittent streams can be a constantly changing factor making estimation of *n* for this energy-loss component difficult. Vegetation can grow to large proportions in just a few seasons, and floods may dramatically alter the roughness characteristics of the channel by flattening or even removing vegetation, which acts to decrease Manning's *n*. Roughness coefficients selected in hydraulic studies years or decades earlier may change significantly. Consequently, earlier computed water surface elevations may no longer be valid for the design discharge. Semi-empirical relations and guidelines developed to estimate the impact of flow on channel vegetation conditions and the resultant impact on Manning's *n* are presented in this document.

In the past, heavy growths of vegetation, which were believed to substantially increase Manning's n-value and decrease channel conveyance, commonly were removed completely to enable adequate conveyance of floodflows. In recent decades, however, emphasis has shifted toward preservation of riparian vegetation to provide habitat for many species of wildlife, as well as aesthetically pleasing multiuse areas for homeowners and businesses. Developed and presented herein are engineering-based guidelines for optimizing the preservation of riparian habitat and the aesthetics of multiuse areas, while mitigating damage from floodflows along stream channels. The guidelines primarily are based on the vegetation component of Manning's *n* that should be maintained in a waterway to allow adequate freeboard, which is an additional amount of conveyance area intended to mitigate risk by providing a factor of safety.

The information, methods, and guidelines available in this report are presented to provide a tool for engineers, hydrologists, developers, and conservationists to gain experience and make better and informed decisions when selecting values of Manning's *n* based on channel and vegetation conditions.

7.9 REFERENCES

7.9.1 Cited in Text

Aldridge, B.N., and Garrett, J.M., 1973, *Roughness coefficients for stream channels in Arizona*, U.S. Geological Survey Open-File Report, 87 p.

Barnes, H.H., Jr., 1967, *Roughness characteristics of natural channels*: U.S. Geological Survey Water-Supply Paper 1849, 213 p.

Benson, M.A., and Dalrymple, T., 1967, *General field and office procedures for indirect discharge measurements*: U.S. Geological Survey Techniques of Water-Resources Investigations, book 3, chap. A1, 30 p.

Burkham, D.E., 1976, *Hydraulic effects in bottom-land vegetation on three major floods, Gila River in southeastern Arizona*: U.S. Geological Survey Professional Paper 655-J, p. 14.

Chow, V.T., 1959, *Open-channel hydraulics*: New York, McGraw-Hill, 680 p.

Cowen, W.L., 1956, *Estimating hydraulic roughness coefficients: Agricultural Engineering*, v. 37, no. 7, p. 473–475.

Cruff, R.W., 1999, C*hannel subdivision and bank selection in open channels*; Phoenix, Arizona. Flood Control District of Maricopa County engineering library, 17 p.

Davidian, Jacob, 1984, *Computation of water-surface profiles in open channels*: U.S. Geological Survey Techniques of Water Resources Investigations, book 3, chap. A15, 48 p.

Fischenich, C., 2000, *Resistance due to vegetation, EMRRP Technical Notes Collection*, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Jarrett, R.D., 1985, *Determination of roughness coefficients for streams in Colorado*, U.S. Geological Survey Water-Resources Investigations Report 85-4004.

Karim, F., 1995, *Bed configuration and hydraulic resistance in alluvial-channel flows*: Journal of Hydraulic Engineering, v. 121, no. 1, p. 15–25.

Leopold, L.B., Wolman, M.G., and Miller, J.P., 1964, *Fluvial processes in geomorphology*: New York, Dover Books on Earth Sciences, 503 p.

Limerinos, J.T., 1970, Determination of Manning's coefficient from measured bed roughness in natural channels: U.S. Geological Survey Water-Supply Paper 1898-B, 47 p.

Phillips, J.V., McDoniel, D., Capesius, J.P., and Asquith, W., 1998, *Method to estimate effects of flow-induced vegetation changes on channel conveyances of streams in Central Arizona*, U.S. Geological Survey Water-Resources Investigations Report 98-4040, 43 p.

Phillips, J.V., and Hjalmarson, H.W., 1994, *Floodflow effects on riparian vegetation in Arizona*: New York, American Society of Civil Engineers Proceedings of the 1994 National Conference on Hydraulic Engineering, v. 1, p. 707–711.

Phillips, J.V., and Ingersoll, T.L., 1998, *Verification of roughness coefficients for selected and natural channels in Arizona*: U.S. Geological Survey Professional Paper 1584, 77 p.

Phillips, J.V., and Tadayon, S., 2006, Selection of Manning's roughness coefficient for natural and constructed vegetated and non-vegetated channels, and vegetation maintenance plan guidelines for vegetated channels in Central Arizona: U.S. Geological Survey Scientific Investigations Report 2006-5108, 41 p.

Riggs, H.C., 1976, A simplified slope-area method for estimating flood discharges in natural channels: U.S. Geological Survey Journal of Research, v. 4, no. 3, May–June 1976, p. 285-291.

Simons, Li and Associates, Inc., 1981, *Design Guidelines and Criteria for Channels and Hydraulic Structures on Sandy Soil*, Urban Drainage and Flood Control District and City of Aurora, Colorado.

Simons, D.B., and Richardson, E.V., 1966, Resistance to flow in alluvial channels: U.S. Geological Survey Professional Paper 422-J, 61 p.

Thomsen, B.W., and Hjalmarson, H.W., 1991, Estimated Manning's roughness coefficients for stream channels and flood plains in Maricopa County, Arizona: Phoenix, Arizona, Flood Control District of Maricopa County, 126 p.

U.S. Department of Transportation (USDOT), Federal Highway Administration, 2005, *Design of Roadside Drainage Channels with Flexible Linings, Hydraulic Engineering Circular No. 15, Publication No. FHWA-IP-87-7*.

Webb, R.H., and Betancourt, J.L., 1992, Climatic variability and flood frequency of the Santa Cruz River, Pima County, Arizona: U.S. Geological Survey Water-Supply Paper 2379, 40 p

.**Page intentionally left blank**

8 HYDRAULIC STRUCTURES

TABLE OF CONTENTS

8.1 SYMBOLS

The following symbols are used in equations throughout Chapter 8:

α = Angular variation of sidewall with respect to channel centerline

α_1 = Kinetic energy correction coefficient

β = Standing wave front angle

θ = Main channel contraction angle used in Hager's side weir discharge equation, radians. Also used as the flow expansion angle downstream from culvert outlets, the V-notch weir spillway configuration, and for Nappe Flow on a stepped spillway.

π = Constant pi

τ = Shear stress on the bed caused by the flow of water, psi

γ = Specific weight, lb/ft^3

a_n = Net area of openings through the trashrack bars, sq ft

a_g = Gross area of the trashrack, including openings, bars and supports, sq ft

A = Area, sq ft

A_1 = Area upstream of the jump, sq ft

A_2 = Area downstream of the jump, sq ft

A_c = Cross sectional area of flow at critical depth. Also used as weir width.

b = Bottom width, ft

b_t = Trickle channel width, ft

B = Basin depth below downstream channel, ft

C = Weir Coefficient

C_0 = Tailwater parameter

C_d = Drag force coefficient

C_h = Variable term used in Hager's side weir discharge equation

C_o = Orifice Coefficient

C_p = Coefficient of mean pressure fluctuations from mean pressure levels in a hydraulic jump

$C_{p\text{-}max}$ = Coefficient of maximum pressure fluctuations from mean pressures level in a hydraulic jump

C_w = Lane's Weighted-creep ratio

d = Depth of flow, ft

d_2 = Depth of basin tailwater, ft

d_{15} = Grain diameter corresponding to 15% passing, by weight (or mass), ft, mm

d_{50} = Grain diameter corresponding to 50% passing, by weight (or mass), ft, mm

d_{85} = Grain diameter corresponding to 85% passing, by weight (or mass), ft, mm

d_{100} = Grain diameter corresponding to 100% passing, by weight (or mass), ft, mm

d_{max}, d_m = Maximum rock diameter, inches

d_{min} = Minimum rock diameter, inches

d_s = Depth of scour, ft

D = Jet plunge height, ft. Also used as diameter of circular pipe or equivalent diameter for non-circular culvert, ft, and as depth of flow over a weir.

D_b = Bedding layer thickness, ft

D_e = Equivalent circular diameter

D_g = Grout depth, ft

D_j = Distance to the hydraulic jump, ft

D_{jm} = Distance to the hydraulic jump, main channel, ft

D_{jt} = Distance to the hydraulic jump, trickle channel, ft

D_n = Drop number

D_r = Rock depth, ft

EGL_m = Energy grade line along the main portion of a drop, ft

EGL_t = Energy grade line along trickle channel through a drop, ft

El_c = Water surface elevation of criteria depth at the crest of a drop, ft

El_m = Elevation of crest of a drop at main channel invert drop, ft

El_t = Elevation of crest of a drop at trickle channel invert, ft

F_b = Force at bend, lb

F_j = Impact force of flow jet, lb

F_r = Froude number

F_{r1} = Froude number upstream of the jump

F_s	=	Specific force, ft^3
g	=	Acceleration due to gravity, 32.2 ft/sec^2
h	=	Height of the wingwalls above the main crest, ft. Also used as the difference between weir measured head and flow depth over a weir (h=H-D).
h_j	=	Height of hydraulic jump, ft
h_L	=	Head loss, ft
h_s	=	Depth of scour, ft. Also used as dissipator pool depth, ft.
h_v	=	Velocity head, ft
h_w	=	Height of weir or height of rectangular culvert, ft
H	=	Head on the weir or orifice, or the height of a rectangular culvert for subcritical flow, ft. Also used as height of a baffle, ft.
H_b	=	Height of a baffle, ft
H_{cw}	=	Height of seepage cutoff, ft
H_d	=	Desired drop across structure, ft
H_{df}	=	Differential head between analysis points, ft
H_g	=	Head loss through a trashrack, ft
H_m	=	Total energy head at the crest of the main drop, ft
H_s	=	Differential head, ft
H_t	=	Total head for Hager's side weir discharge equation measured from the top of the weir crest, ft. Also used as measured head for a sharp-crested weir.
H_{wt}	=	Wing wall height, ft
J	=	Ratio of Y_2 *to* Y_1
K_t	=	Trashrack loss coefficient (empirical)
L	=	Apron length for a riprap apron, ft. Also used as horizontal-crested weir width, ft and side weir length, ft.
L_1, L_2	=	Apron length to dissipator, ft
L_a	=	Approach or apron length, ft
L_A	=	Apron length for a riprap basin, ft
L_{bm}	=	Design basin length, main channel, ft

L_{bt} = Design basin length, trickle channel, ft

L_B , L_b = Basin pool length, ft. L_b also used as basin rock length.

L_d = Drop length, ft

L_{dm} = Nappe length, main channel, ft

L_{dt} = Nappe length, trickle channel, ft

L_f = Slope face length, ft

L_H = Horizontal creep distance along contact surfaces less than 45 degrees, ft

L_j = Length of the hydraulic jump, ft

L_s = Length of scour, ft; and dissipator pool length, ft

L_{td} = Downstream transition length, ft

L_{tu} = Upstream transition length ft

L_V = Vertical creep distance along any contact surfaces greater than 45 degrees, ft. Also used as transition length, ft.

L_{wt} = Wing wall length, ft

M = Mass rate of flow, lb sec/ft

m = Number of side weirs for Hager's side weir discharge equation

n = Manning's roughness coefficient

P = Height of the weir crest above the approach channel, ft. Also used as the force due to pressure on the bend.

ΔP = Maximum pressure fluctuation at a given location within a hydraulic jump, psi

q = Discharge per unit width, cfs/ft

q_c = Discharge per unit width of crest, cfs/ft

q_m = Discharge per unit width of the main channel at drop, cfs/ft

q_t = Discharge per unit width of the trickle channel at drop, cfs/ft

Q = Discharge, cfs

Q_w = Side weir discharge, cfs

r = Channel centerline radius of curvature, ft

r_w = Round-crested weir radius used in Hager's side weir discharge equation

R = Hydraulic radius, ft

s = Step height of stepped spillway, ft

s_o = Bed or drop slope (S is also used), ft/ft

t = Ratio of trapezoidal channel bottom width to sideslope horizontal width

T = Top width of flow in the channel, ft

T_c = Top width of riprap basin at apron, ft

TW = Tailwater depth, ft

v_c = Critical velocity, ft/sec

V = Velocity, ft/sec

ΔV = Change in magnitude of velocity through a bend, ft/sec

V_a = Approach velocity, ft/sec

V_{allow} = Allowable exit velocity

V_{ave} = Average velocity at culvert outlet, ft/sec

V_B = Basin exit velocity at critical depth, ft/sec

V_c = Basin exit velocity at critical depth, ft/sec

V_g = Velocity between the bars of a trashrack, ft/sec

$(V_L)_{ave}$ = Average velocity for distance L downstream of culvert outlet, ft/sec

V_n = Velocity through the net trashrack area, ft/sec

V_o = Average velocity at culvert outlet, ft/sec

W = Chute width, ft

W_B = Basin width at the basin exit, ft

W_h = Variable term used in Hager's weir discharge equation

W_o = Width of box culvert, diameter of pipe culvert, or span of pipe arch, ft

W_1 = Chute block width, ft

W_2 = Chute block spacing, ft

W_3 = Baffle block width, ft

W_4 = Baffle block spacing, ft

X = Where the subcritical depth of the jump forms, ft

y = Depth of flow, ft

y_B = Depth at basin exit, ft

y_c = Critical flow depth, ft

y_{cm} = Critical depth at a drop in the main channel, ft

y_{ct} = Critical depth at a drop in the trickle channel, ft

y_e = Equivalent depth, ft

y_f = Vertical fall at a drop, ft

y_h = Variable term used in Hager's side weir discharge equation

y_m = Hydraulic depth, ft

y_n = Normal depth, ft

y_o = Culvert outlet depth, ft

y_p = At a vertical drop, the pool depth under the nappe just below the crest, ft

Y_1 = Initial (upstream) flow depth, ft

Y_2 = The tailwater depth required to cause a jump to form immediately downstream of the initial depth location for Y_1, ft

$(Y_2)_m$ = Sequent depth, main channel, ft

$(Y_2)_t$ = Sequent depth, trickle channel, ft

Y_f = Effective fall height from the crest to the basin floor, ft

Y_p = Pool depth under the nappe downstream of the crest, ft

Y_{tw} = Actual tailwater depth present downstream of the drop, ft

z = For a vertical drop structure, the difference in the bed elevations of the approach channel at the weir and the downstream channel at the end of the structure, ft. Also used as the side slope, ft/ft.

Z = Channel side slope horizontal distance per foot of drop, ft/ft

Z_f = Drop face slope, ft/ft

8.2 USE OF STRUCTURES IN DRAINAGE

Hydraulic structures are used in storm drainage works to control water flow characteristics such as velocity, direction and depth. Structures may also be used to control the elevation and slope of a channel bed, as well as the general configuration, stability and maintainability of the waterway.

The use of hydraulic structures can increase the capital cost of drainage facilities while lowering O&M costs. The use of hydraulic structures should be limited by careful and thorough hydraulic engineering practices to locations and functions justified by prudent planning and design. On the other hand, use of hydraulic structures can reduce initial and future maintenance costs by changing the characteristics of the flow to fit the project needs, and by reducing the size and cost of related facilities.

Hydraulic structures include channel drop structures, spillways, grade control structures, energy dissipators, bridges, transitions, chutes, bends and many other specific drainage works. Depending on the function to be served, the shape, size and other features of hydraulic structures can vary widely from project to project. Hydraulic design procedures (including model testing in some cases) that examine the structure and related drainage facilities are a key part of the final design for all structures.

This chapter is oriented toward control structures for drainage channels, outlets for storm drains and culverts, and spillways for non-jurisdictional dams. Design guidelines for spillways for jurisdictional dams or other specialized conveyance measures are beyond the scope of this manual. The design professional is referred to the references cited at the end of this chapter.

8.2.1 Channel Drop Structures

Drop structures are used to reduce the effective slope of a natural or artificial channel. Typically, a drop structure extends across the entire width of the channel and provides grade control for a full range of flows. Check structures are similar in concept, but their objective is to stabilize and control the channel bed or low flow zone. During a major flood, portions of the flow circumvent the structure, but erosion is maintained at an acceptable level. Overall stability is maintained by control of the low flow area, which would otherwise degrade downward. A series of check structures can be an economical interim grade control measure for natural channels in urbanizing areas or for artificial channels where funding is inadequate for construction of drop structures.

8.2.2 Conduit Outlet Structures

Energy dissipation structures are necessary at the outlets of culverts or storm drains to reduce flow velocity and to provide a transition whereby the concentrated, high velocity flow exiting the conduit is changed to a wider, shallower and non-erosive flow. Outlet structures may be preformed rock riprap stilling basins or reinforced concrete structures such as impact basins.

Spillways

Spillways are conveyance features that permit outflow from stormwater basins. Engineering nomenclature divides these into principal spillways and emergency spillways. The principal spillway for a dam is that hydraulic structure that has been designed to pass the more frequent flow events while the hydraulic capacity of the emergency spillway is held in reserve for the rare flow events. Principal spillways are associated with water storage impoundments (i.e. those with a permanent pool) and stormwater detention basins (wet or dry). Emergency spillways, in one form or another, are provided at these facilities as well as stormwater retention basins. An emergency spillway is a flow conveyance feature designed to safely pass flows in excess of the facility design discharge in a manner that does not threaten the integrity of the principal spillway, facility embankment, or surrounding infrastructure. It also serves to pass flows normally conveyed by the principal spillway under circumstances when the principal spillway becomes plugged. This chapter presents the hydraulic equations used to determine hydraulic capacity for simple spillways. See Chapter 9 for a more detailed discussion pertaining to how these facilities are incorporated into stormwater basins.

8.2.3 Special Channel Structures

Bridges, spur dikes, channel transitions, bifurcations, constrictions and bends, and structures for lined channels and for long conduits are examples of hydraulic structures used for special applications. Access ramps, while not a hydraulic structure, are necessary components of a channel to facilitate maintenance.

Bridges and Related Structures

Bridges have the potential advantage of crossing a waterway without disturbing the flow. However, for overall economic and structural reasons, encroachments and piers in the waterway are a practical reality. A bridge structure can cause significant hydraulic effects, such as an increase in the water surface elevation, and channel scour. These conditions must be analyzed and measures must be designed for mitigation of negative impacts. Spur dikes, levees, drop or check structures, and pier and abutment protection are types of structures designed to control hydraulic effects at bridge crossings. Refer to Chapter 5 for further discussion on bridges.

Channel Transitions

Channel transitions are typically used to moderately vary the cross sectional geometry to allow the waterway to fit within a more confined right-of-way, or to purposely accelerate the flow to be carried by a specialized high velocity conveyance structure. Constrictions are designed to restrict and reduce the conveyance along a short reach. Examples are a bridge with roadway approach embankments that significantly encroach into a floodplain, or a structure designed to raise the upstream water surface to force spills into an off-channel storage facility. An expansion structure is usually required at the downstream end of a constricted channel reach or structure to provide a safe, non-eroding transition to the unconstricted channel. Potential conditions for creation of a hydraulic jump must be examined and provisions made for control of a jump and associated turbulent flow conditions. Bifurcations are structures that permit flow to be diverted within a channel. Similarly, side channel spillways also permit the diversion of flow. Finally, channel junctions pose interesting design considerations, especially under supercritical flow conditions.

Structures for Lined Channels and Long Conduits

Acceleration chutes can be used to maximize the use of limited downstream right-of-way, and to reduce downstream channel and pipe costs. However, chutes should only be used where good hydraulic and environmental design concepts permit the use of high velocity flow. In general, high velocity flow is not permitted in urban areas and applications in other areas will require careful scrutiny. Bends in lined channels and closed conduits require analysis to determine if super-critical flow occurs, or if special structural and other design considerations are needed.

8.2.4 Access Ramps

To facilitate maintenance, access ramps are required for all channels. Access ramps for maintenance are recommended at all street crossings on both sides of the street.

8.2.5 Trashracks and Access Barriers

Trashracks serve two purposes when utilized in conjunction with storm drains, culverts and detention basin outlets. First, trashracks prevent entrapment of person(s) inadvertently swept into flood waters. Secondly, these structures prevent debris from becoming lodged in the downstream conduit. Depending upon the flow characteristics, the analysis and design considerations vary.

Access barriers are placed at the downstream end of storm drains, culverts, and detention basin outlets to prevent the public from entering the conduit. Access barriers are typically the same configuration as trashracks.

8.2.6 Factors of Safety

Specific calculations to determine foundation stability and factors of safety against sliding, uplift, and overturning for a hydraulic structure are necessary in the design of safe structures. The factor of safety derived for a particular case depends, to a large degree, on the risk and consequence of failure. Therefore, the selected factor of safety must be appropriate for each structure being designed.

The factors of safety for sliding, uplift, and overturning all may be different for a particular structure. A general range of 1.5 to 2.0 for these factors is recommended for many types of structures subjected to a variety of loading conditions (see: *Design Manual, Foundations and Earth Structures* (U.S. Navy, 1982); *Design of Small Dams* (USBR, 1987); *Design of Gravity Dams* (USBR, 1976); and *Drainage of Roadside Channels with Flexible Linings* (USDOT, 1988)).

8.3 CHANNEL DROP STRUCTURES

8.3.1 General

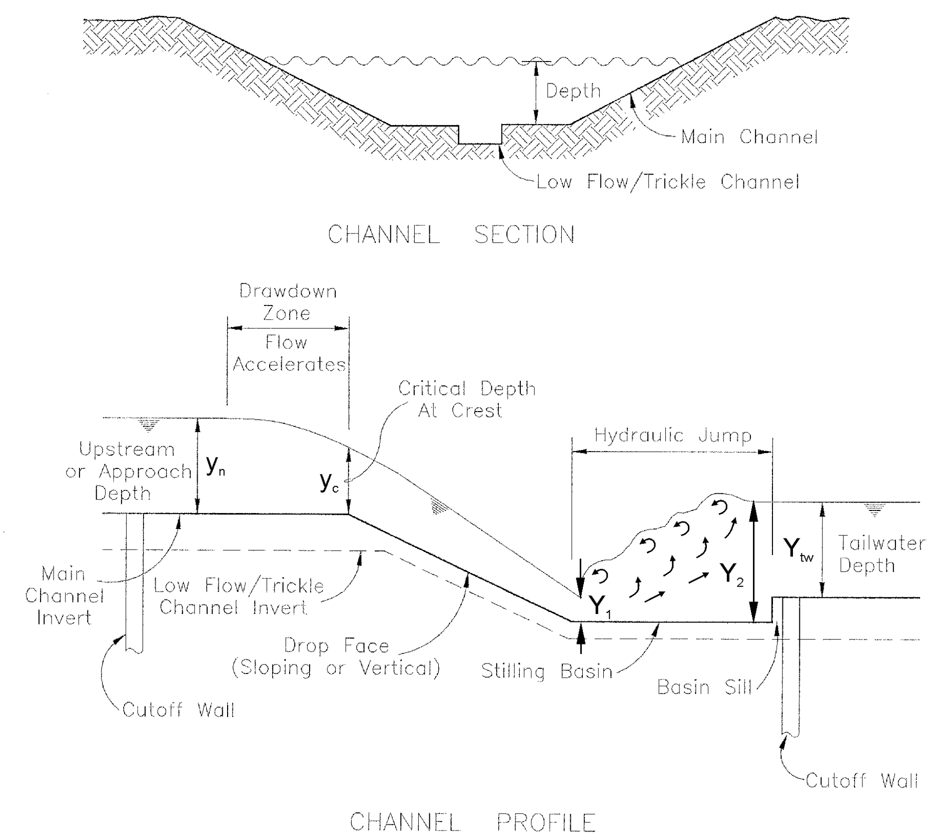
The term drop structure is broadly defined. Included are structures built to restore previously damaged channels, those constructed during new urban development to prevent accelerated erosion caused by increased runoff, and applications in which other specialized hydraulic conditions are created in the flow channel.

The focus of this design guideline is on drop structures with design flows up to 10,000 cfs. Flows less than 500 cfs are in the usual range for grade control structures.

Basic Components of a Drop Structure

Figure 8.1 shows a typical channel drop structure with its various components. Once a particular structure type is selected for design, analyses are conducted to determine the optimal sizing or extent of the various components.

FIGURE 8.1
TYPICAL DROP STRUCTURE COMPONENTS
(ADAPTED FROM McLaughlin Water Engineers, Ltd. 1986)



Design Considerations

In addition to hydraulic performance (discussed in Section 8.3.2), a number of other considerations affect the selection of an appropriate drop structure for a particular application.

Soil and Foundation Condition - Geotechnical investigations should be completed to identify the characteristics of the on-site soils. Silty and sandy soils require detailed analyses for seepage control. Expansive soils require special design techniques to minimize differential movement. Structure design for foundation, walls and slabs must consider soil and hydrostatic pressures, seepage and potential scour.

Construction Concerns - The selection of a drop and its foundation may also be tempered by construction difficulty, access, material availability, etc. Quality control through conscientious inspection is an important consideration.

Maintenance Concerns - Issues to be considered in the design include, ease of access to the crest and stilling basin areas, vandal resistance, eliminate trapped (ponded) water, sediment accumulation, and landscaped or grassed slopes that are easily maintained.

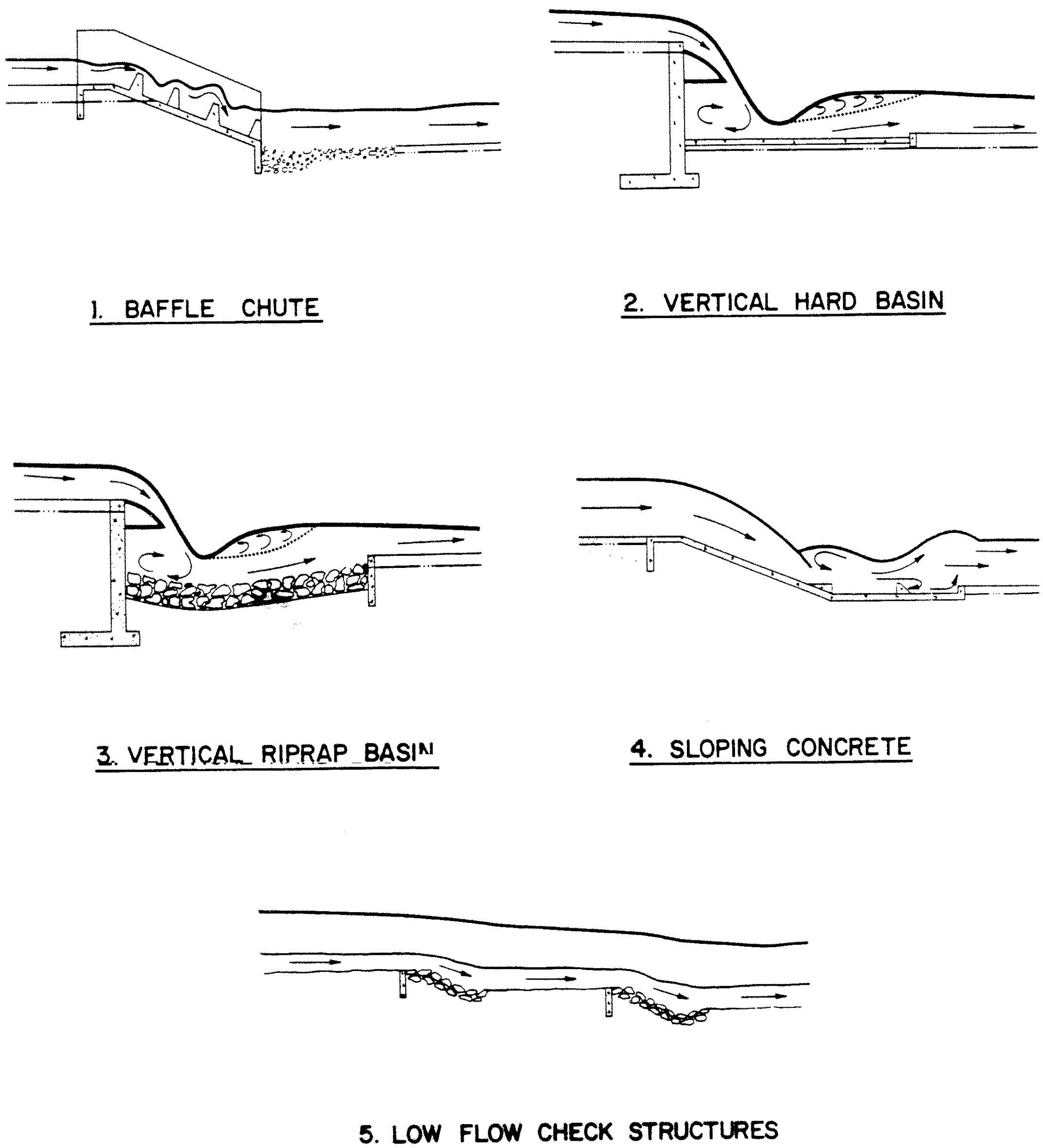
Sociological Considerations - These include public acceptability issues such as safety (Section 8.7), visual appearance (Section 8.8), mosquito breeding in ponded water, etc.

Drop Structure Types - Design guidance is presented in this section for the following drop structures:

- Baffle Chute Drops
- Vertical Hard Basin Drops
- Vertical Riprap Basin Drops
- Sloping Concrete Drops
- Grade Control Structures

Figure 8.2 shows schematic profiles of each type.

FIGURE 8.2
DROP STRUCTURE TYPES
(McLaughlin Water Engineers, Ltd. 1986)



Due to a high failure rate and excessive maintenance costs, drop structures having loose riprap on a sloping face are discouraged. Refer to Sloped Drop Structure/Rock Chute for design guidelines.

All drop structures should be inspected on a regular basis during construction in regard to construction quality and integrity. In addition, drop structures must be monitored on a periodic basis after construction.

Additional bank and bottom protection may be needed if secondary erosional tendencies are revealed. Thus, it is advisable to establish construction contracts and budgets with this in mind. Use of standardized design methods for the types of drops described herein can reduce the need for secondary design refinements. Section 8.3.3 presents considerations for the selection of the appropriate type of drop structure for particular application or site conditions.

8.3.2 Hydraulic Analysis Considerations

General Procedures

These design procedures are generalized. Use them to identify the most suitable approach, with the understanding that detailed analytical methods and design specifications may vary as a function of site conditions and hydraulic performance. A standard drop structure design approach would include at least the following steps:

1. Define the maximum design discharge (usually the 100-year) and other discharges appropriate for analysis (selected discharge(s) expected to occur on a more frequent basis, which may behave differently at the drop).

2. Select possible drop structure alternatives to be considered (Section 8.3.3).

3. Determine the required longitudinal channel slope and the total drop height required to produce the desired hydraulic conditions.

4. Establish the channel hydraulic parameters, reviewing drop structure and channel combinations that may be most effective.

5. Conduct hydraulic analyses for the structure. Where appropriate, apply separate hydraulic analyses to the main channel and the low flow zones of the drop to determine the extent of protection required, as well as the potential problems/solutions for each. (See discussion later in this section.)

6. Perform soils and seepage analyses to obtain foundation and structural design information. Combine seepage and hydraulic analysis data to determine forces on the structure. Evaluate uplifting, overturning, and sliding.

7. Evaluate alternative structures in terms of their estimated capital and maintenance costs, and identify comparable risks and problems for each alternative. Review alternatives with client and jurisdictional agency to select final plan. (This task is not specifically a part of the hydraulic analysis criteria, but is mentioned to illustrate other factors which are involved in the analysis of alternatives.)

8. Use specific design criteria to determine the drop structure dimensions, material requirements and construction methods necessary to complete the design for the selected structures.

Crest and Upstream Hydraulics

Usually, the starting point of drop analysis and design is the designation of the crest section (or review of existing configuration) at the top of the drop. As flow passes through critical depth near the crest, upstream hydraulics are separated from downstream. The critical flow state must be calculated and compared with the downstream tailwater effect which may submerge the crest and effectively control the hydraulics at the crest.

With control at the drop crest, upstream water surface profile computations are used to estimate the distance that protection should be maintained upstream, that is, the distance to where localized velocities are reduced to acceptable values. Backwater computations also yield the maximum upstream flow depth used to set wall abutment and bank heights. The water surface profile computations should include a transition/contraction head loss, which should typically range from 0.3 (modest transitions) to 0.5 (more abrupt transitions) times the change in velocity head. The reader should refer to standard hydraulic references for guidance (i.e., Chow 1959). For a given discharge, there is a balance between the crest base width, upstream and downstream flow velocities, the Froude Number in the drop basin, and the location of the jump. These parameters must be selected for each specific application.

Two basic configurations of crests are assumed. Baffle chutes, vertical hard basin and vertical riprap basin drops frequently have vertical or nearly vertical abutments with nearly rectangular cross sections. Sloping concrete drops generally have sloping abutments, forming a trapezoidal crest cross section. All drop types would typically have a low flow channel which is extended through the drop crest section at the channel invert.

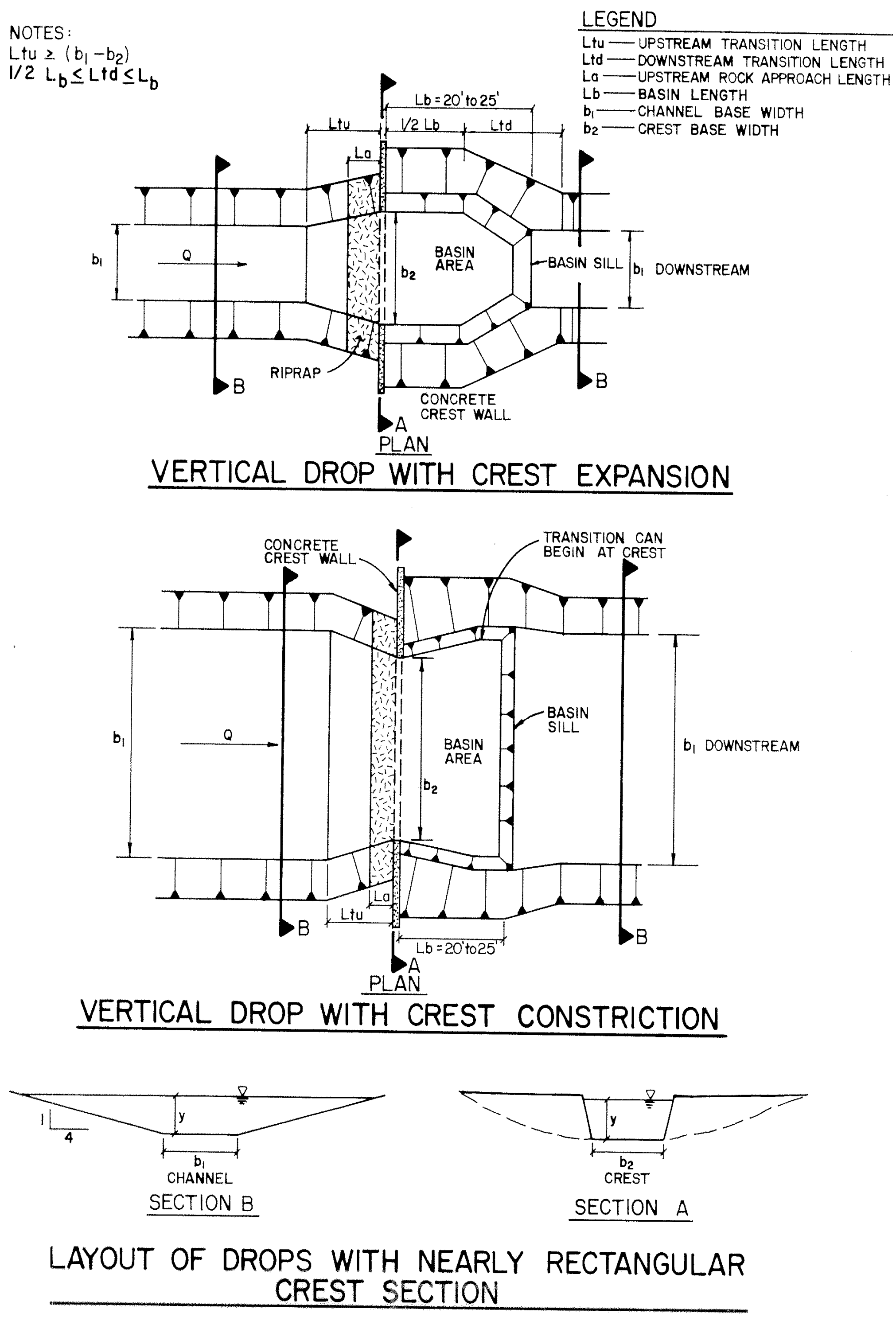
Vertical or Near Vertical Abutments at Drop Crest - Figure 8.3 presents alternative drop crests at a vertical drop structure. In general, the objectives of upstream hydraulics and crest design are:

1. To maintain freeboard in the approach channel,

2. To optimize crest and basin dimensions to achieve the most cost-effective structure, and

3. To prevent erosion in the transition zone, where flow accelerates approaching the crest.

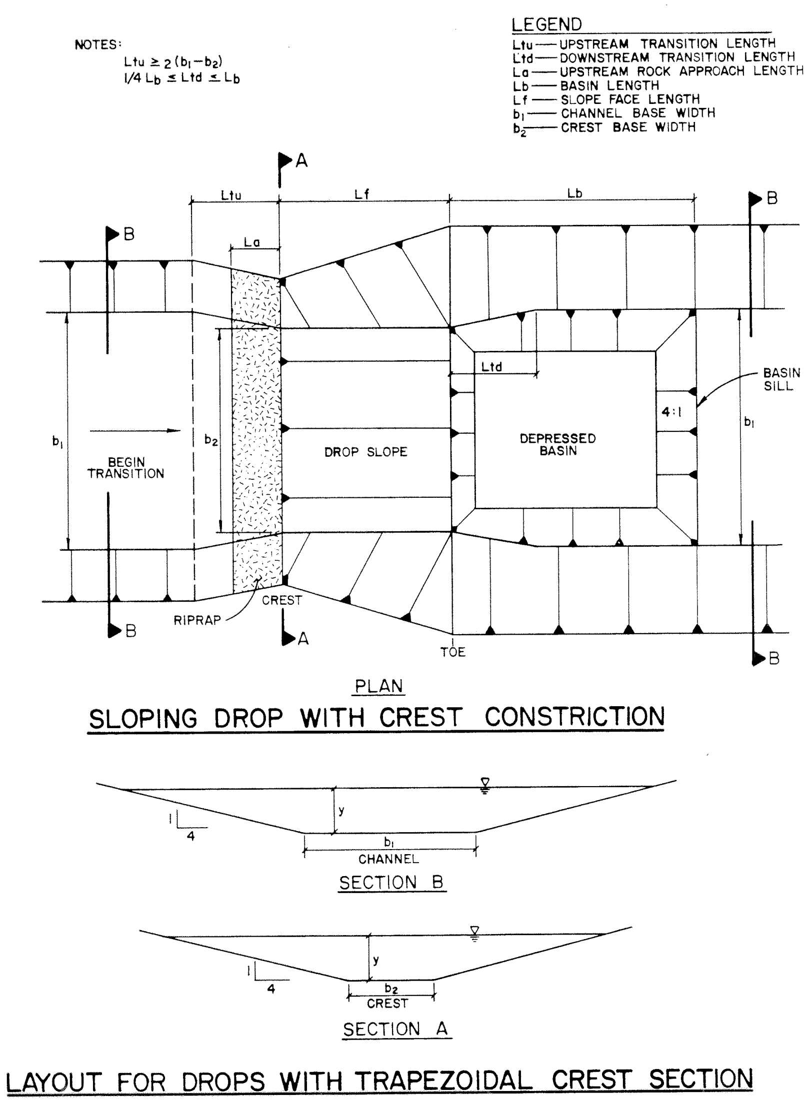
FIGURE 8.3
TYPICAL VERTICAL DROP CREST CONFIGURATION
(McLaughlin Water Engineers, Ltd. 1986)



A crest expansion may be necessary to maintain adequate freeboard in the upstream channel and reduce drawdown velocities just upstream of the crest. A crest constriction may be appropriate for wide channels to reduce the cost of the crest wall.

Sloping Abutments at Drop Crest - Figure 8.4 shows a schematic layout for the drop crest and upstream channel at a sloping drop structure. The design objectives discussed previously also apply here. Constricting the trapezoidal crest serves to economize the structure while maintaining upstream freeboard. The seepage cutoff wall is typically placed at or near the upstream end of the transition zone and the zone protected with concrete or grouted rock. This arrangement also provides better seepage control, as discussed later in this section.

FIGURE 8.4
TYPICAL SLOPING DROP CREST CONFIGURATION
(McLaughlin Water Engineers, Ltd. 1986)



Water Surface Profile Analysis

Backwater computations should be completed for the channel reaches upstream and downstream of the proposed drop structure to establish approach flow conditions and tailwater conditions for the range of design flows.

The next step is to determine the location of the hydraulic jump so that the stilling basin can be sized to adequately contain the zone of turbulence. The determination of the hydraulic jump's location is usually accomplished through the comparison of the unit specific force for the supercritical inflow and the downstream subcritical flow. For vertical drop structures, this requires analysis of the tailwater elevation to determine if it is sufficient to cause the jump to occur immediately, or if the jet will wash downstream until the specific force is sufficiently reduced to allow the jump to occur.

For sloping drop structures, water surfaces must be determined for the supercritical profiles down the face of the drop. The location of the hydraulic jump can be determined by using Equation (8.1) to compute the unit specific force F_s, above and below the toe of the drop. The hydraulic jump, in either the trickle channel or the main drop, will begin to form where the unit specific force of the downstream tailwater is greater than the specific force of the supercritical flow below the drop.

$$F_s = \frac{q^2}{gy} + \frac{y^2}{2} \tag{8.1}$$

The depth y, for downstream specific force determination, is the tailwater surface elevation minus the ground elevation at the point of interest, which is typically the main basin elevation or the trickle channel invert (if the jump is to occur in the basin). The depth for the upstream specific force (supercritical flow) is the supercritical flow depth at the point in question.

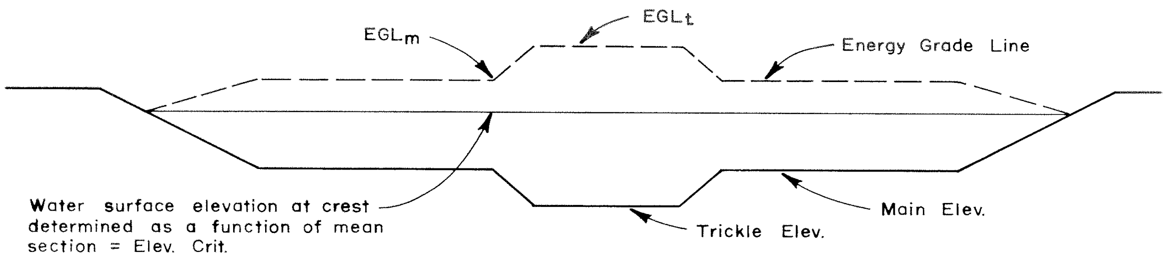
For jumps in vertical riprap basins, the user has to rely on the criteria derived from laboratory studies. The shaping or reshaping of riprap influences the jump stability and location. Nevertheless, the basic specific force equation provides some guidance.

Ideally, for economic considerations, the jump should begin no further downstream than the drop toe. This is generally accomplished in the main drop zone by depressing the basin to a depth nearly as low as the downstream trickle channel elevation.

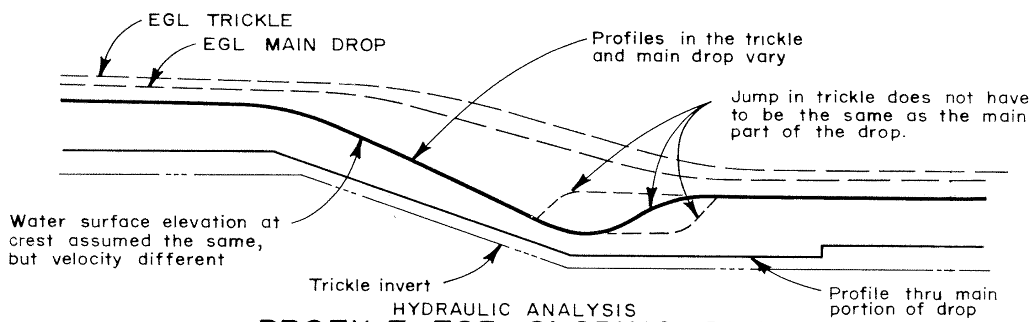
Analysis should be conducted for a range of flows, since flow characteristics at the drop can vary with discharge. For example, the 10-year flow may cascade down the face of a sloping drop and form a jump downstream of the toe, whereas the 100-year flow may totally submerge the drop.

Where a major channel incorporates a low flow channel, separate analyses should be completed for the low flow zone and the major channel overbank zone. This is because the deeper flow profile in the low flow channel zone has a higher energy grade line profile (Figure 8.5). Specific force analysis in this zone shows that the hydraulic jump will not occur in the same location as the rest of the flow over the drop, and in most cases the jump will occur further downstream. Separate analysis for this condition will determine if the stilling basin length is sufficient to contain the jump.

FIGURE 8.5
TYPICAL SECTION AND PROFILE FOR SLOPING DROP
(McLaughlin Water Engineers, Ltd. 1986)



HYDRAULIC ANALYSIS
SECTION AT CREST OF DROP



HYDRAULIC ANALYSIS
PROFILE FOR SLOPING DROP

Hydraulic Jump

With the exception of the baffle chute drop, all of the drop structures described herein use the formation of a hydraulic jump to dissipate energy. A discussion of this hydraulic phenomenon is presented as follows.

A hydraulic jump occurs when flow changes rapidly from low stage supercritical flow to high stage subcritical flow. Hydraulic jumps can occur: 1) when the slope of a channel abruptly changes from steep to mild; 2) at sudden expansions or contractions in the channel section; 3) at locations where a barrier, such as a culvert or bridge, occurs in a channel of steep slope; 4) at the downstream side of dip crossings or culverts; and 5) where a channel of steep slope discharges into other channels.

Hydraulic jumps are useful in dissipating energy, and consequently they are often used at drainage way outlet structures and drop structures as an efficient way to minimize the erosive potential of floodwaters. However, because of the high turbulence associated with hydraulic jumps, they must be contained within a well-protected area. Complete computations must be made to determine the height, length and other characteristics of the jump (including consideration of a range of flows) in order to adequately size the containment area.

The type of hydraulic jump that forms, and the amount of energy that it dissipates, is dependent upon the upstream Froude number (F_{r1}). The various types of hydraulic jumps that can occur are listed in Table 8.1.

TABLE 8.1
TYPES OF HYDRAULIC JUMPS
(USDOT, FHWA, HEC-14, 2006)

Upstream Froude Number	Type of Jump	Energy Loss, %
1.0 < $F_{r1} \leq$ 1.7	Undular Jump	Minimal
1.7 < $F_{r1} \leq$ 2.5	Weak Jump	20%
2.5 < $F_{r1} \leq$ 4.5	Oscillating Jump	20 to 45
4.5 < $F_{r1} \leq$ 9.0	Steady Jump	45 to 70
9.0 < F_{r1}	Strong Jump	70 to 85

Jump Height - The depth of flow immediately downstream of a hydraulic jump is referred to as the sequent depth (Y_2). The sequent depth in rectangular channels whose upstream Froude number is $\geq$ 1.7, can be computed by use of the following equation:

$$Y_2 = \frac{1}{2}Y_1[\sqrt{1 + 8F_{r1}^2} - 1] \tag{8.2}$$

The solution for sequent depth in trapezoidal channels can be obtained from a trial-and-error solution of Equation (8.3), which is derived from momentum equations. It is also acceptable for design purposes to determine the sequent depth in trapezoidal channels from Equation (8.2). Equation (8.2) is much simpler to solve and produces only slightly greater values for sequent depth than does Equation (8.3).

$$\frac{ZY_1^3}{3} + \frac{ZY_1^2}{2} + \frac{Q}{gA_1} = \frac{ZY_2^3}{3} + \frac{bY_2^2}{3} + \frac{Q}{gA_2} \tag{8.3}$$

Figure 8.6 and Figure 8.8 provide graphs of hydraulic jumps for a horizontal rectangular channel and a horizontal trapezoidal channel, respectively.

Undular Jump - An undular hydraulic jump is the type of jump which occurs where the upstream Froude number is between 1.0 and 1.7. This type of jump is characterized by a series of undular waves which form on the downstream side of the jump. Experiments have shown that the first wave of an undular jump is higher than the height given by Equation (8.3). Therefore, the height of this wave should be determined as follows:

$$\frac{Y_2 - Y_1}{Y_1} = F_{r1}^2 - 1 \tag{8.4}$$

Jump Length - The length of a hydraulic jump is defined as the distance from the front face of the jump to a point immediately downstream of the roller. Jump length can be determined from Figure 8.7 and Figure 8.9.

Surface Profile - The surface profile of a hydraulic jump may be needed to design the extra bank protection, or training walls for containment of the jump. The surface profile can be determined from Figure 8.10.

Jump Location - In most cases a hydraulic jump will occur at the location in a channel where the initial and sequent depths and initial Froude number satisfy Equation (8.3). This location can be found by performing direct-step calculations in either direction toward the suspected jump location until the terms of the equation are satisfied. Specific force analysis can then be used by employing Equation (8.1) to establish where a jump will occur. The hydraulic jump will begin to form where the unit specific force of the downstream tailwater is greater than the unit force of the supercritical approach flow.

<u>Design Charts and Figures</u>

Figure 8.6 to Figure 8.10 and Table 8.2 have been included as additional aids to the user of this manual.

FIGURE 8.6
HEIGHT OF A HYDRAULIC JUMP FOR A HORIZONTAL RECTANGULAR CHANNEL
(USDOT, FHWA, HEC-14, 2006)

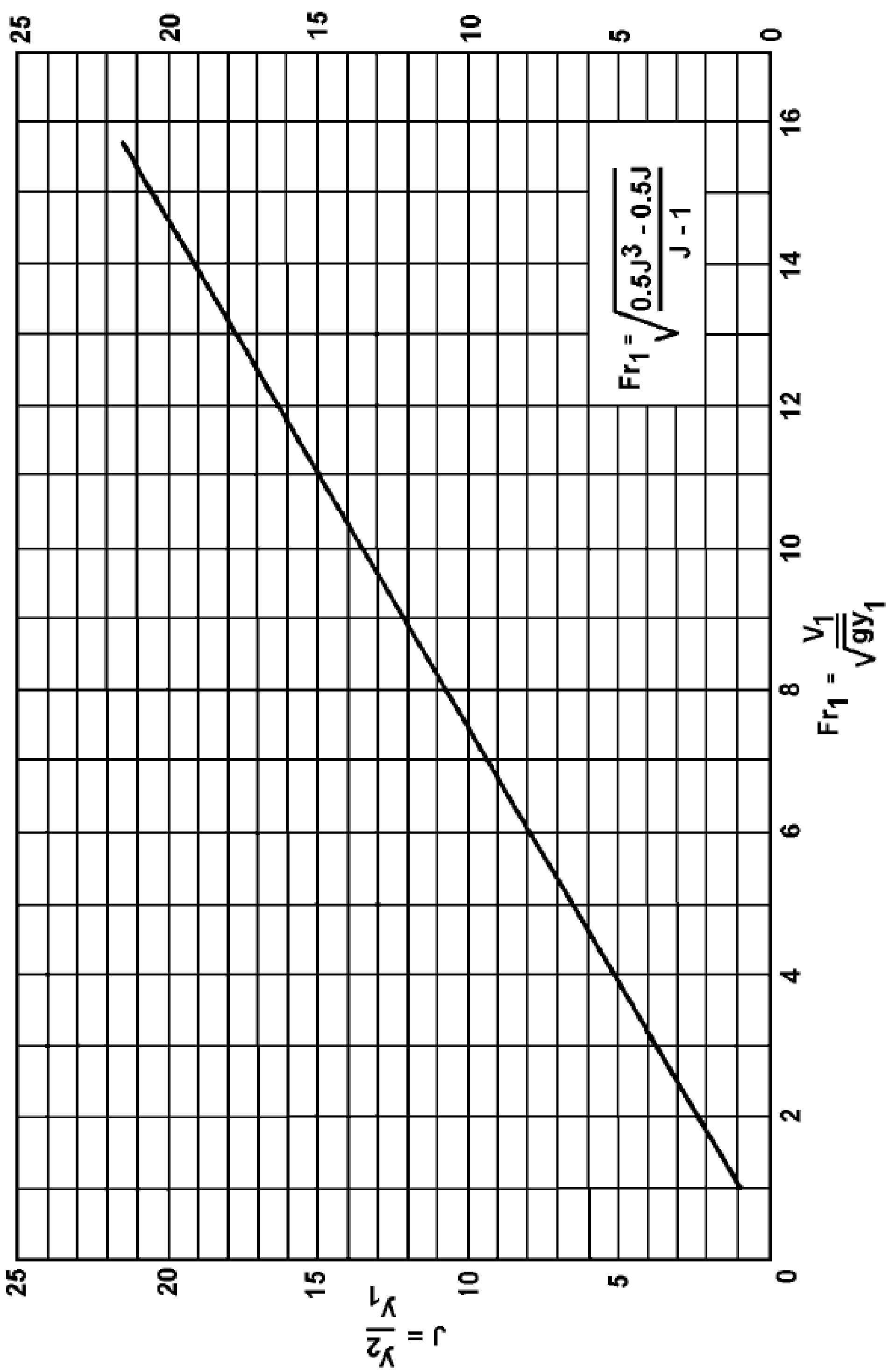


FIGURE 8.7
LENGTH OF HYDRAULIC JUMP FOR RECTANGULAR CHANNELS
(USDOT, FHWA, HEC-14, 2006)

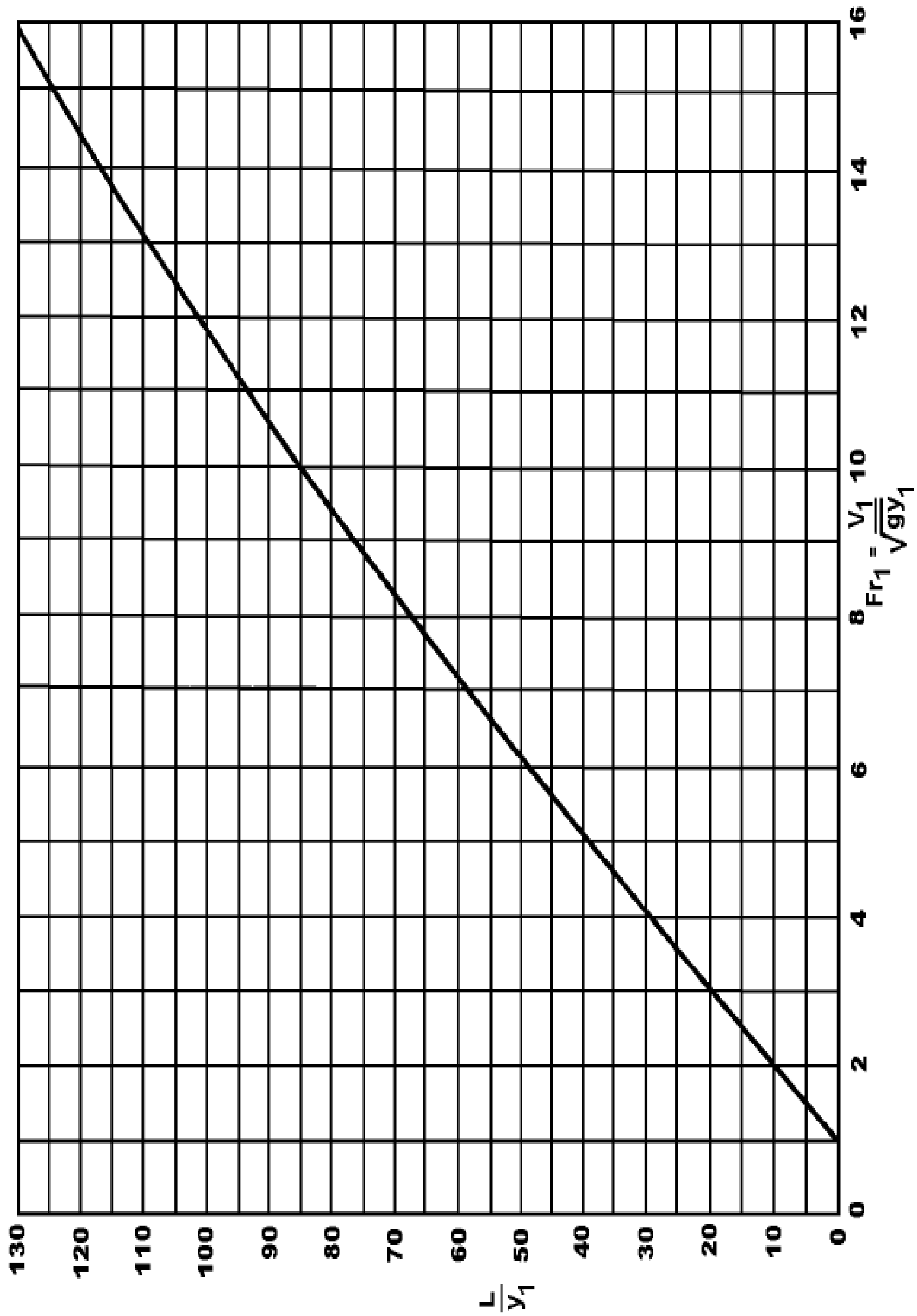


FIGURE 8.8
HEIGHT OF A HYDRAULIC JUMP FOR A HORIZONTAL TRAPEZOIDAL CHANNEL
(USING HYDRAULIC DEPTH) (USDOT, FHWA, HEC-14, 1983)

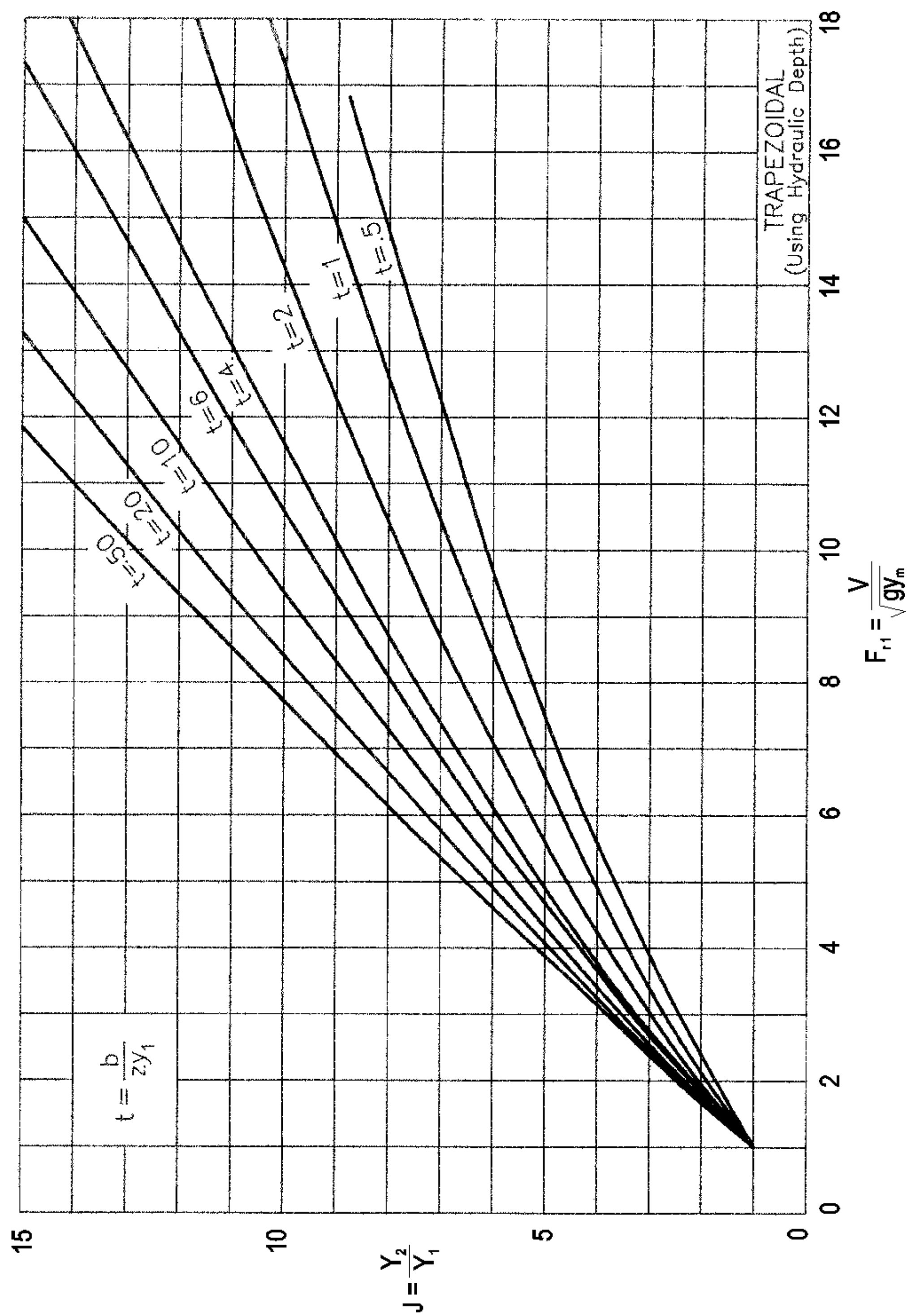


FIGURE 8.9
LENGTH OF A HYDRAULIC JUMP FOR NON-RECTANGULAR CHANNELS
(USDOT, FHWA, HEC-14, 1983)

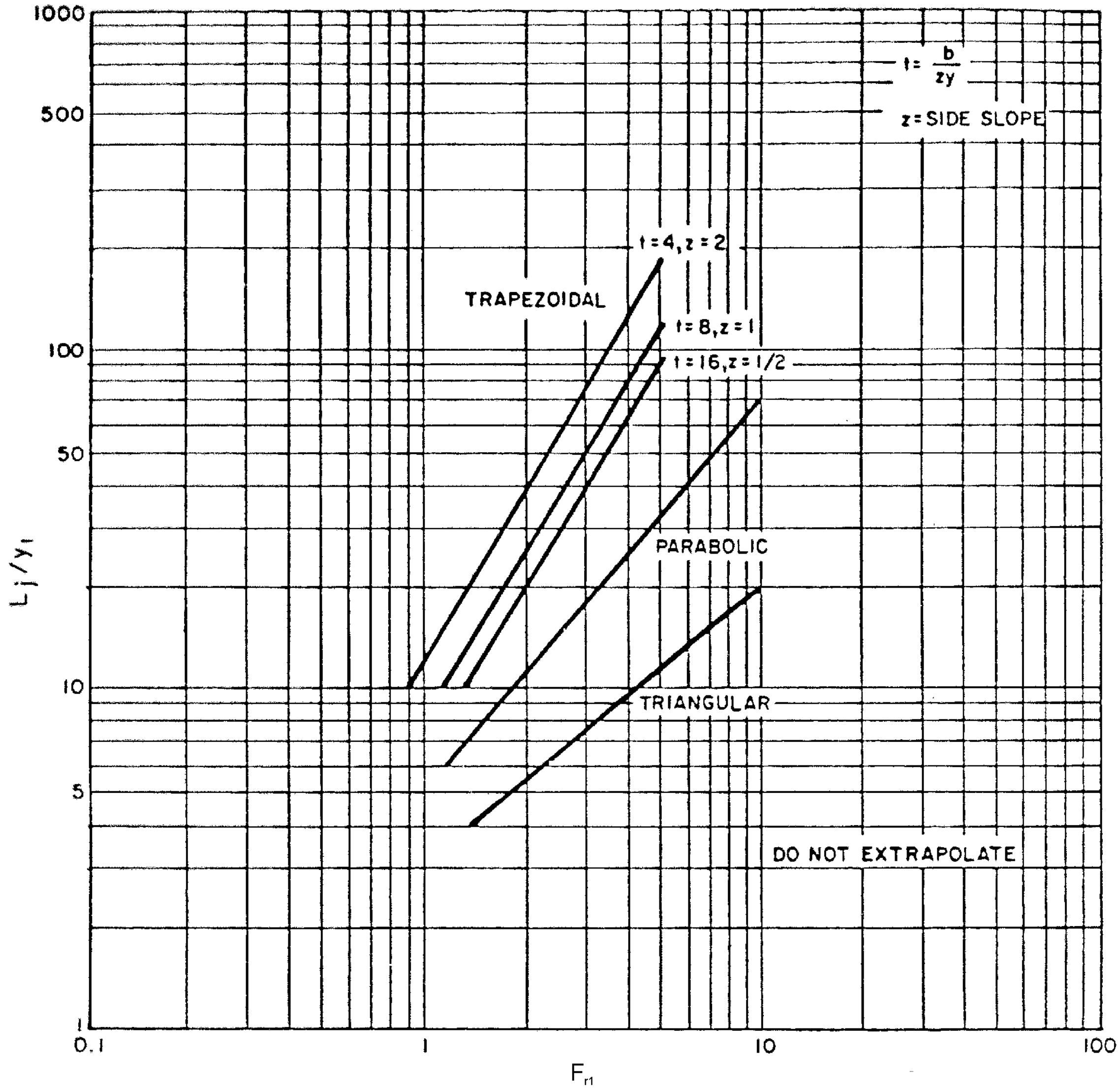


FIGURE 8.10
SURFACE PROFILE OF A HYDRAULIC JUMP IN A HORIZONTAL CHANNEL
(Chow, 1959)

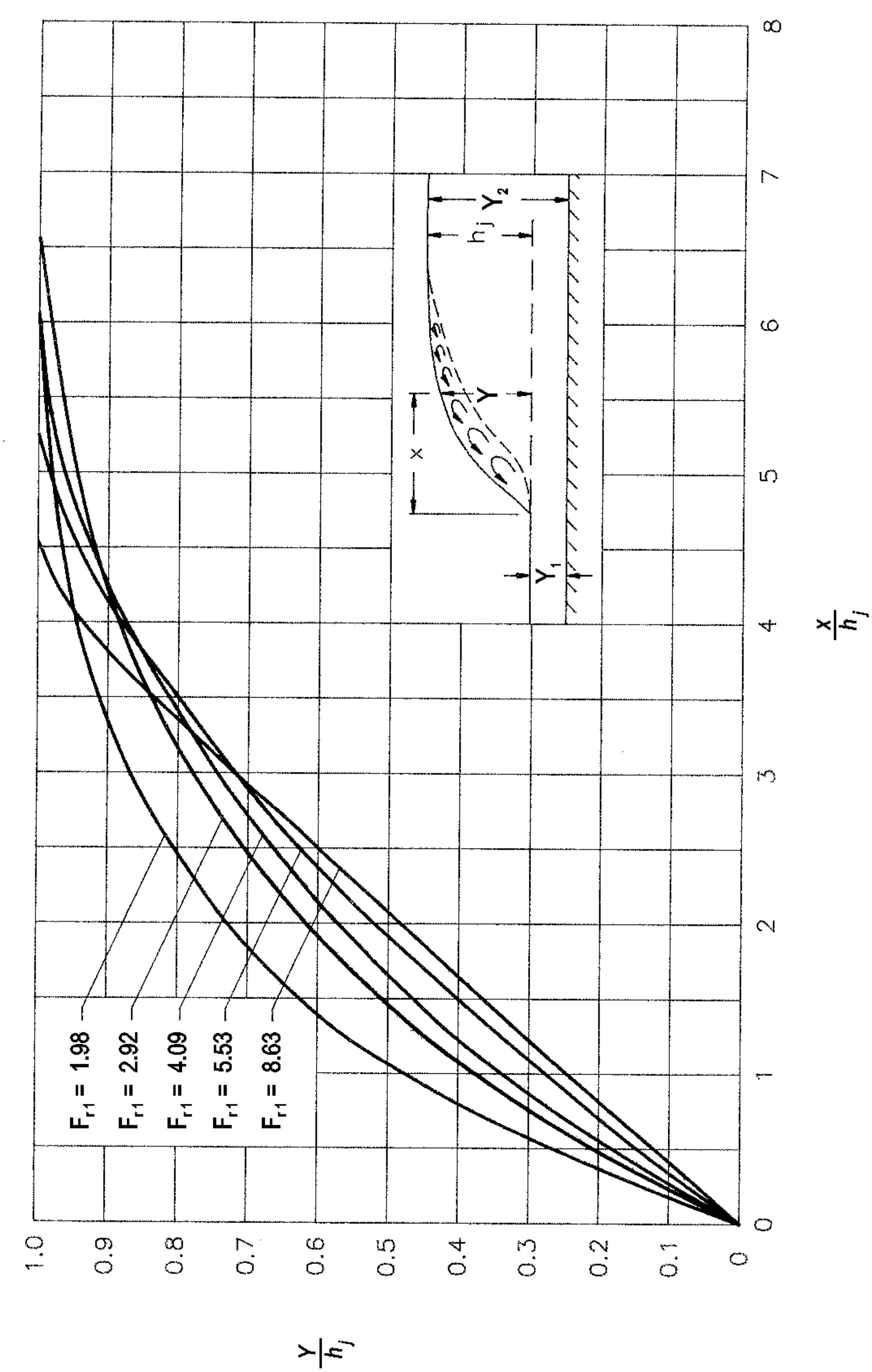


TABLE 8.2
UNIFORM FLOW IN CIRCULAR SECTIONS FLOWING PARTLY FULL
(USDOT, FHWA, HEC-14, 2006)

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

Seepage and Uplift Forces

The most common technique for seepage analysis is that proposed by E.W. Lane (1935), commonly referred to as "Lane's Weighted-Creep Method”. The essential elements of this method are paraphrased as follows:

1. The weighted-creep distance of a cross section of a drop structure is the sum of the vertical creep distances (along contact surfaces steeper than 45 degrees), L_V, plus one-third of the horizontal creep distances (along contact surfaces less than 45 degrees), L_H.

2. The weighted-creep head ratio is defined as:

$$C_w = \frac{(L_H + 3L_v)}{3H_{df}}$$

3. Lane's recommended weighted-creep ratios are given for various foundation materials in Table 8.3.

4. Reverse filter drains, weep holes, and pipe drains are aids to provide security from seepage, and recommended safe weighted-creep head ratios may be reduced as much as 10 percent, if used.

5. Care must be exercised that cutoff walls extend laterally into each bank so that flow will not outflank them.

6. The upward pressure to be used in design may be estimated by assuming that the drop in pressure from headwater to tailwater along the contact line of the drop structure and cutoff wall is proportional to the weighted-creep distance.

Seepage is controlled by increasing the seepage length such that C_w is raised to a conservative value. Soils tests must be taken during design and confirmed during construction. These tests are especially critical for reinforced concrete structures.

An example of this technique can be found in *Design of Small Dams* (USBR, 1977). An alternative approach is to use a flow net or computerized seepage analysis to estimate subsurface flows and uplift pressures under a structure. Seepage considerations should be included in the design of cutoff walls, wall footings, drains, filters, structural slabs, and grouted masses.

Locating a seepage cutoff wall upstream of the crest of a drop structure and using horizontal impervious blankets can be effective. It is also very important to control lateral seepage around the structure.

TABLE 8.3
LANE'S WEIGHTED-CREEP: RECOMMENDED RATIOS
(Lane, 1935)

Material	C_W Ratio
Very fine sand or silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Coarse gravel including cobbles	3.0
Boulders with some cobbles and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay or hardpan	1.6

8.3.3 Selection of Drop Structures

There are four major considerations for the selection of the type of drop structure for a particular application: 1) surface flow hydraulic performance, 2) foundation and seepage control, 3) economic considerations, and 4) construction considerations. Other factors which can affect selection are land uses, aesthetics, safety, maintenance, and anticipated downstream channel degradation.

Surface Flow Hydraulic System

The primary consideration for the selection of a drop structure should be functional hydraulic performance. The surface flow hydraulic system combines channel approach and crest hydraulics, sloping or vertical drop hydraulics and downstream tailwater conditions. Hydraulic analysis procedures are presented in Section 8.3.2. Additional guidelines are also contained in Section 8.3.4.

Foundation and Seepage Control Systems

Table 8.4 presents some typical foundation conditions and control systems typically used for various drop heights. Table 8.4 is presented only as a guide. The hydraulic engineer must calculate hydraulic loadings which can occur for a variety of conditions such as interim construction conditions, low flow, and flood flow. The soils/foundation engineer couples this information with the on-site soils information. Both work with a structural engineer to establish final loading diagrams, and selection and sizing of structural components. This section presents information relevant to hydraulics, but refer to geotechnical and structural books for related information.

TABLE 8.4
GENERAL SEEPAGE CUTOFF TECHNIQUE SUITABILITY
(UDFCD, 2008)

Soil Conditions	Drop Height (ft)			
	2	**4**	**8**	**12**
Sand and gravel over bedrock with sufficient depth of material to provide support - groundwater prevalent.	S*	S*	S/SwB*	S/SwB*
	CTc	CTc/ST	ST	ST
	CTf	CTf/CTI		
Sand and gravel with shallow depth to bedrock - groundwater prevalent.	CTc	CTc/ST	ST	ST
	CW	CW	CW	CW
	S**	S**	S**	SwB**
Sand and Gravel, great depths to bedrock - groundwater prevalent.	S	S	S	S/SwB
	CTc	CTc/ST	ST	ST
Sand and gravel, no groundwater, or water table normally below requirement (for variation caused by depth to bedrock, see first case).	S	S	S	S/SwB
	CTf/CTI	CTI	CTI	CTI
	CW	CW		
Clay (and silt) - medium to hard.	CTc	CTc	CTc	CTc
	CW	Reduce length for difficult backfill conditions		
	CTf/CTI	Only for local seepage zones/silts		
	ST	Expensive - for special problems		
Clay (and silt) - soft to medium with lenses of permeable material - groundwater present.	S	S	S	S/SwB
	CTc	CTc	CTc/ST	ST
Clay (and silt) - soft to medium with lenses of permeable material-may be moist but not significant groundwater source.	S	S	S	S/SwB
	CTc	CTc	CTc/ST	ST
	CTf	CTI	CTI	CTI
	CW	CW	CW	CW

* Consider Scour in sheet pile support

** Excavate onto bedrock and set into concrete

Legend

S	=	Sheet pile
SwB	=	Sheet pile with bracing and extra measures
CTc	=	Cutoff Trench backfilled with concrete
ST	=	Slurry Trench; similar to CTc; but trench walls are supported with slurry and then later replaced with concrete or additives that effect cutoff
CW	=	Cutoff Wall; conventional wall, possibly with footer, backfilled; note that the effective seepage length should generally be decreased because of backfill
CTl	=	Cutoff Trench with synthetic liner and fill
CTf	=	Cutoff Trench with clay fill

Economic Considerations

Evaluation of alternative drop structure costs should include consideration of construction costs and maintenance costs. Construction costs include site work specific to the structure, seepage control, excavation, reinforced concrete, riprap, boulders, grout and backfill. Maintenance costs include rock replacement, debris removal, erosion repair, structural repairs, graffiti and silt removal. A standard method of cost comparison is present-worth analysis by which estimated maintenance costs are converted to present worth amounts by applying an appropriate discount rate factor. The present worth maintenance cost is then added to the construction cost of each structure under consideration for comparison.

Other factors also affect the economics of alternative types of drop structures. In many cases, specific site requirements may dictate the direction of drop structure design. Depending on location, some construction materials, such as riprap or boulders, may not be readily available at reasonable cost. Analysis may include consideration of the cost of a single drop structure of height (H_d) versus the cost of two structures, each 1/2 H_d high.

Construction Considerations

The selection of a drop and its foundation may also be tempered by construction difficulty, location, access, and material availability/delivery. Table 8.5 lists construction considerations for key drop structure materials. Additional discussion of construction concerns is included with the design guidelines for each drop type in the following section.

TABLE 8.5
QUALITY CONTROL MEASURES AND CONCERNS OF DROP STRUCTURE COMPONENTS

Type	Quality Concerns	Quality Control Measures and Inspection
Concrete	The major concern is strength and ability to resist weathering. Aggregate strength and durability are important. Special architectural treatments include exposed aggregate, form liners and color additives.	Preconstruction items include review of shop drawings for reinforcing steel, formwork patterns and ties, concrete design mix and related tests, color additives or coatings and architectural treatments such as form liners, handrails and fences. Any architectural test samples should be completed and approved, along with all coatings, weather protection or other items which could affect appearance.
Reinforcing Steel	Usually not a problem unless the wrong grade of steel is brought to job, or site conditions are conducive to corrosion problems. Epoxy coated reinforcement can be specified for critical conditions.	During construction there are numerous items which require checking, including: rebar placement, formwork, tie placement, weep holes and drains, form release coatings and form cleaning before concrete placement, form removal, concrete placement and testing, weather protection, sealants, tie hole treatment, concrete finish work, and earthwork, especially that related to seepage control.
Riprap and Rock	Hardness is of concern because the rock is subject to rough handling and impact forces. Durability concerns are: Oxidation, weathering (freeze thaw tests), and leaching or dissolving by water. Fracturing, which leads to odd or undesirable shapes, is to be avoided. Seams or other discontinuities can lead to breakup or undesirable shapes and damage during handling. Geologic type is important; sedimentary rocks are undesirable. Volcanic rock often has low density. Density of the rock requires specific gravity tests.	A significant effort is needed in the area of rock quality control. Submittals should be required from suppliers to document quality. Rock should be durable, sound, and free of seams or fractures. The specific gravity should be a minimum of 2.40. Specifications should include requirements for orderly procedures and appropriate equipment, both for rock and grout placement. Gradation, durability and specific gravity tests of riprap at the quarry are needed, and should only be waived for small projects where the quarry can demonstrate recent tests. Handling that results in excessive breakage should result in changed methods and/or reexamination of rock quality. Subgrades should be dewatered and stabilized. Filters and bedding layers should be reviewed for compatibility to the on-site soil conditions. Rock handling and placement is critical. Riprap should be handled selectively so that the gradation is reestablished through any given vertical section. Areas where the thickness is comprised of all materials smaller than the d_{50}, or where excessive voids or radical surface variations occur should be reworked. Good placement techniques should result in a riprap layer with surface materials d_{50} size or greater, closely spaced with voids thoroughly chinked and locked between larger rock, top surfaces generally parallel to the plane of the overall riprap bank or surface, and no great departures in surface elevation from rock to rock. Graded riprap should not be used for grouting, as the smaller rock can prevent full penetration of the grout to the subgrade and can cause incomplete filling of the voids. Large rock or boulders should be placed with a gradall or multi-prong grapple device for ease of handling and to minimize disturbance of the subgrade. A minimum dimension should be specified for the rock to aid field inspection. On slopes, uphill boulders should be keyed in below the tops of downhill boulders for stability. A "stairstep" arrangement where the top surface of the rock is flat and horizontal is preferable for both aesthetic and hydraulic reasons.

TABLE 8.5 (CONTINUED)
CONSTRUCTION COMPONENTS CONCERNS & QUALITY CONTROL MEASURES OF DROP STRUCTURES

Type	Quality Concerns	Quality Control Measures and Inspection
Grout	Cement content and type, aggregate and water content are important considerations for strength and durability.	The key to success with grouting is to use rock that is no smaller in any dimension than the desired grout thickness (so that one can fully access and fill the voids), to pump and place the grout using a grout pumper with a nozzle that can penetrate to the subgrade, to vibrate using a "pencil vibrator" to assure complete filling of the voids, to have good control of the grout mix (too wet creates shrinkage cracks and stability problems on slope, too dry leads to poor penetration), and to place the grout to the desired thickness. A minimum grout thickness is needed to counteract uplift forces. However, placing too much is unattractive and reduces the roughness of the drop which is needed to prevent the jump from washing downstream. During grouting, it is important to protect the weep drains. With care, one can avoid getting grout on the top of the rock. Any spillage should be washed off immediately. A wood float leaves a smooth finish, and the "pencil vibrator", which is preferred, will generally leave a satisfactory appearance with some touch-up. Full time inspection is required during grouting, as is periodic inspection during the rock placement depending upon the performance of the contractor and the aesthetic appearance desired.
Sheetpile	Sheetpile comes in many configurations and, in particular, joint details. It requires geotechnical, structural and hydraulic expertise, as well as pile driving experience during construction.	Inspection is required to ensure that piling is driven to the design depth, or keyed into bedrock if required. Underground obstructions can create problems with driving. If piling becomes separated at the joints during installation, excessive subsurface flow can result.
Roller Compacted Concrete	Construction equipment limitations constrain drop structure dimensions.	The exposed horizontal portion of the step should be six feet at a minimum with the overall lift width at least nine feet. The designer should coordinate with prospective contractors during the design of the structure.
Soil Cement	Construction equipment limitations constrain drop structure dimensions.	The exposed horizontal portion of the step should be six feet at a minimum with the overall lift width at least nine feet. The designer should coordinate with prospective contractors during the design of the structure.
Synthetic Liners	Liners must be flexible and strong enough to allow adjustment to the actual subgrade, and to allow rock placement without significant damage to the liner material.	Subgrade must be well prepared to minimize voids and piping along the smooth surface of the liner. Certificates of conformance to the technical specifications should be provided by the manufacturer. Liners should be spliced only when necessary and placed in accordance with manufacturers instructions
Seepage Cut-off Soils	Important considerations are: classification and homogeneity of clay soils, placement and compaction techniques.	The subgrade should be inspected and sloped to achieve compaction of the cutoff soils and the adjacent subgrade. In order to use this type of drop structure, the subgrade soil needs to be a clay (CL), as classified by a qualified soils engineer.
Drains	Permeability and gradation of media, reverse filter characteristics and compatibility with in situ materials, pipe and other hydraulic components.	Gradation analysis of in situ materials and proposed filter media are advisable. Fabric materials should be used with caution to insure that plugging will not occur. Piping and valving components should comply with specifications and be double checked for suitability for the particular application. The toe drain and other drains should be placed and protected from contamination, particularly if grout or concrete is placed later.

TABLE 8.5 (CONTINUED)
CONSTRUCTION COMPONENTS CONCERNS & QUALITY CONTROL MEASURES OF DROP STRUCTURES

Type	Quality Concerns	Quality Control Measures and Inspection
Cutoffs using Slurry Trench	The homogeneity and stability of the slurry cutoff is critical. The construction techniques to achieve a cutoff to the desired depth and width are also critical.	Practically, cutoffs using slurry trench techniques are more exotic applications and require intensive geotechnical engineering and custom specifications for individual applications. Measures can involve intensive soil testing, density testing of slurry mixtures, tests related to special chemicals and admixtures, and standard concrete and grout testing methods. Besides inspections related to all of the above, site environmental controls are required for slurry mixing and placement, and for disposal of materials displaced during the process.
Architectural and Landscape Items	Coatings are always subject to quality concerns, which are compounded by substrate conditions. Plantings are subject to a wide variety of quality and size.	Landscape and architectural treatments can make a big difference in appearance; take care to work with experienced professionals.

8.3.4 Design Guidelines for Drop Structures

Baffle Chute Drops

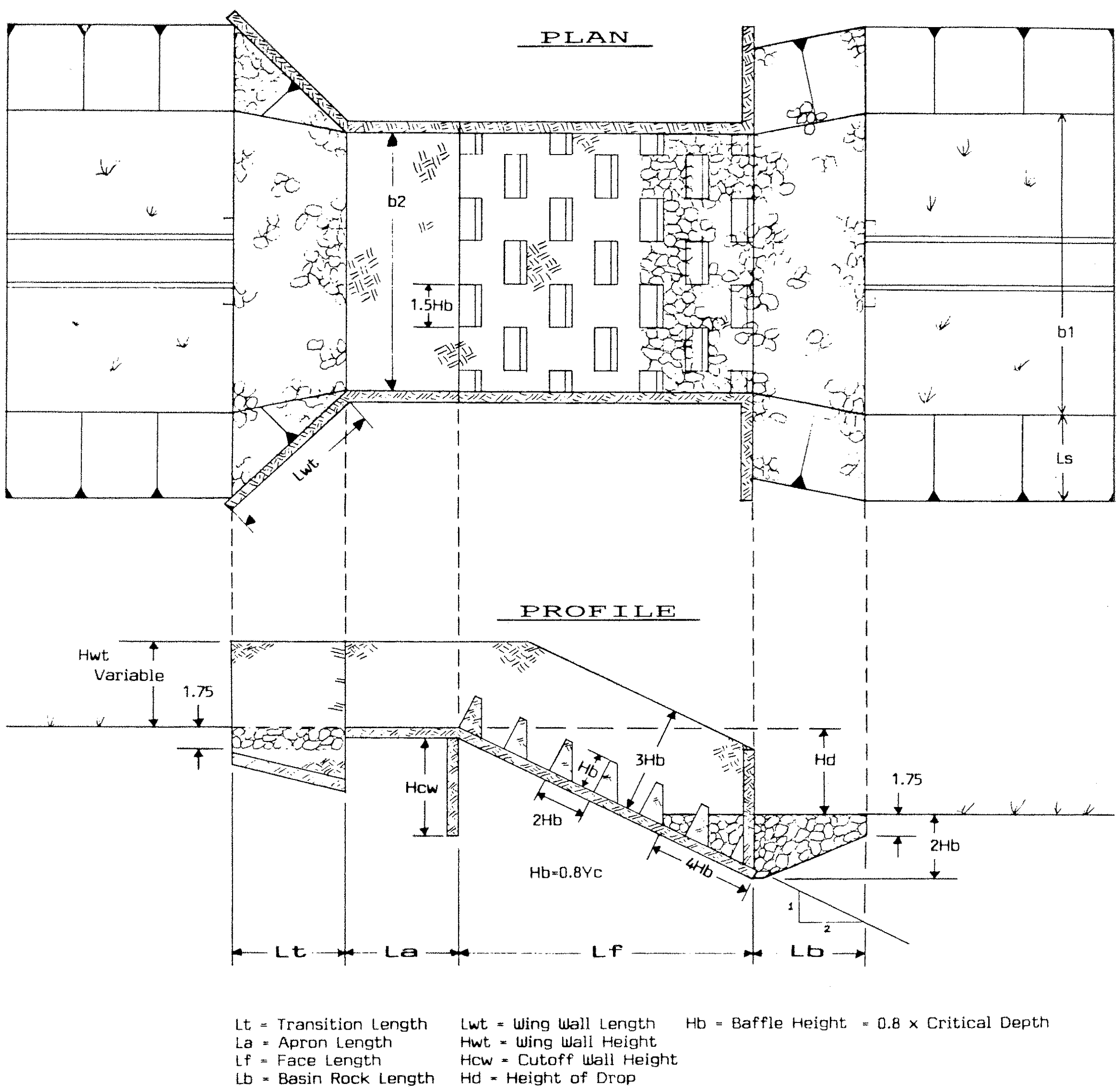
The USBR has developed design standards for a reinforced concrete chute with baffle blocks on the sloping face of the drop, which is commonly referred to as baffled apron or baffle chute drops. There are excellent references that should be used for the design of these structures: *Hydraulic Design of Stilling Basins and Energy Dissipators* (Peterka, 1984), and *Design of Small Canal Structures* (USBR, 1974). Another reference is *Baffled Apron as Spillway Energy Dissipator* (Rhone, 1977), which evaluates higher design discharges, and entrance modifications to reduce the backwater effect caused by the baffles.

The optimal performance occurs for a unit flow (q) at the chute width of 35 to 60 cfs/ft. Model testing has evaluated discharges up to 300 cfs/ft, and there have been structures built with up to 120 cfs/ft. The USBR states that the recommended design flow of 60 cfs/ft for baffle chute drops has been exceeded at several locations without causing significant problems.

The hydraulic concept involves flow repeatedly encountering obstructions (baffle piers) that are of a nominal height equivalent to critical depth. The excess energy through the drop is dissipated by the momentum loss associated with the reorientation of flow. A minimum of four rows of baffle piers are recommended to achieve control of the flow and maximum dissipation of energy. Guidelines are given for sizing and spacing the blocks. Designing for proper approach velocities is critical to structure performance. One advantage of the baffle chute drop is that it does not require tailwater control.

Typical design consists of upstream transition walls, a rectangular approach chute, a sloping apron of 2:1, or flatter, slope with multiple rows of baffle piers (see Figure 8.11). The toe of the chute extends below grade and is backfilled with loose rock to prevent undermining the structure by eddy currents or minor degradation of the downstream channel. This rock will rearrange to establish a stable bed condition and produce additional stilling action. The structure is effective without tailwater; however, higher tailwater reduces scour at the toe. Grouted and concrete basins have also been used to prevent a standing pool from forming at the transitions to the downstream trickle and main channels. The structure also lends itself to a variety of soils and foundation conditions.

FIGURE 8.11
BAFFLE CHUTE DROP
(McLaughlin Water Engineers, Ltd., 1986)



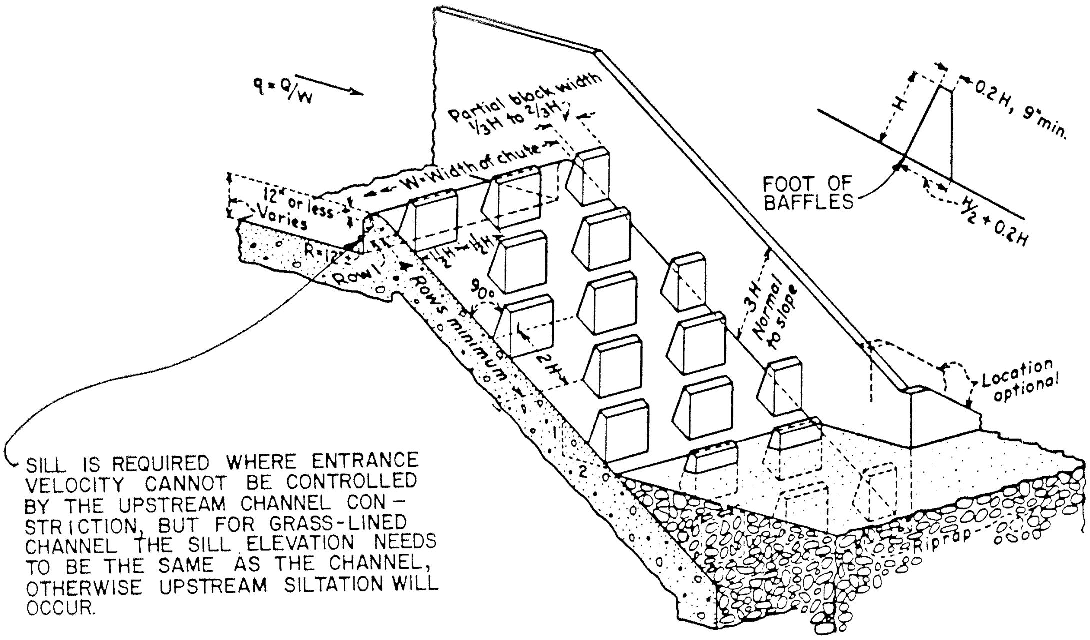
There are fixed costs associated with the upstream wing walls, crest approach section, downstream transition walls and a minimum length of sloping apron (for four baffle rows). Consequently, the baffle chute becomes more economical with increasing drop height.

This design is quite flexible in adaption, once the hydraulic principles are understood. For example, the design has been modified for low drops by locating two rows of baffles on the slope and two rows on a horizontal extension of the chute. Another approach has been to use a flatter chute slope than the usual 2 horizontal to 1 vertical. There are examples where sloping abutments have been used. Other examples include the use of sloping abutments at the crest and chute sides. These drops can be extended at a later date if downstream bed degradation occurs beyond that initially anticipated.

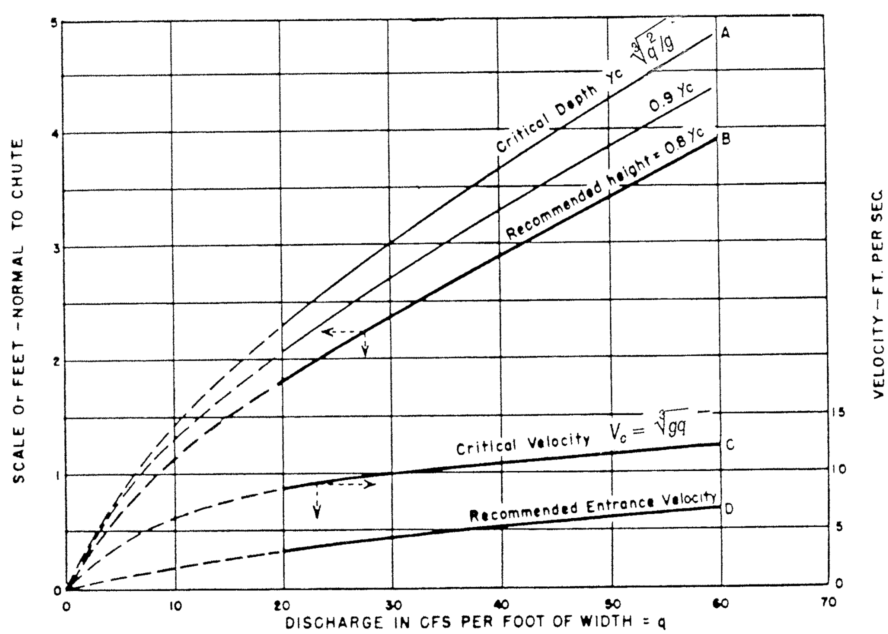
The potential for debris flow must also be considered. Use caution when conditions include streams with heavy debris flow, because the baffles can become clogged between the interstices, resulting in overflow, low energy dissipation, and direct impingement of the erosive stream jet on the downstream channel.

The design performance has been documented for numerous baffled apron drops (USBR, 1974). The resulting design precautions generally relate to relatively minor problems, such as erosion protection in adjacent channels, spray above the chute walls, and debris problems. The basic design criteria and modification details are given in Figure 8.12 and Figure 8.13. Remaining structural design parameters must be determined for specific site conditions. The recommended design procedures are discussed on the following pages.

FIGURE 8.12
BAFFLE CHUTE DESIGN CRITERIA
(ADAPTED FROM: Peterka 1984)

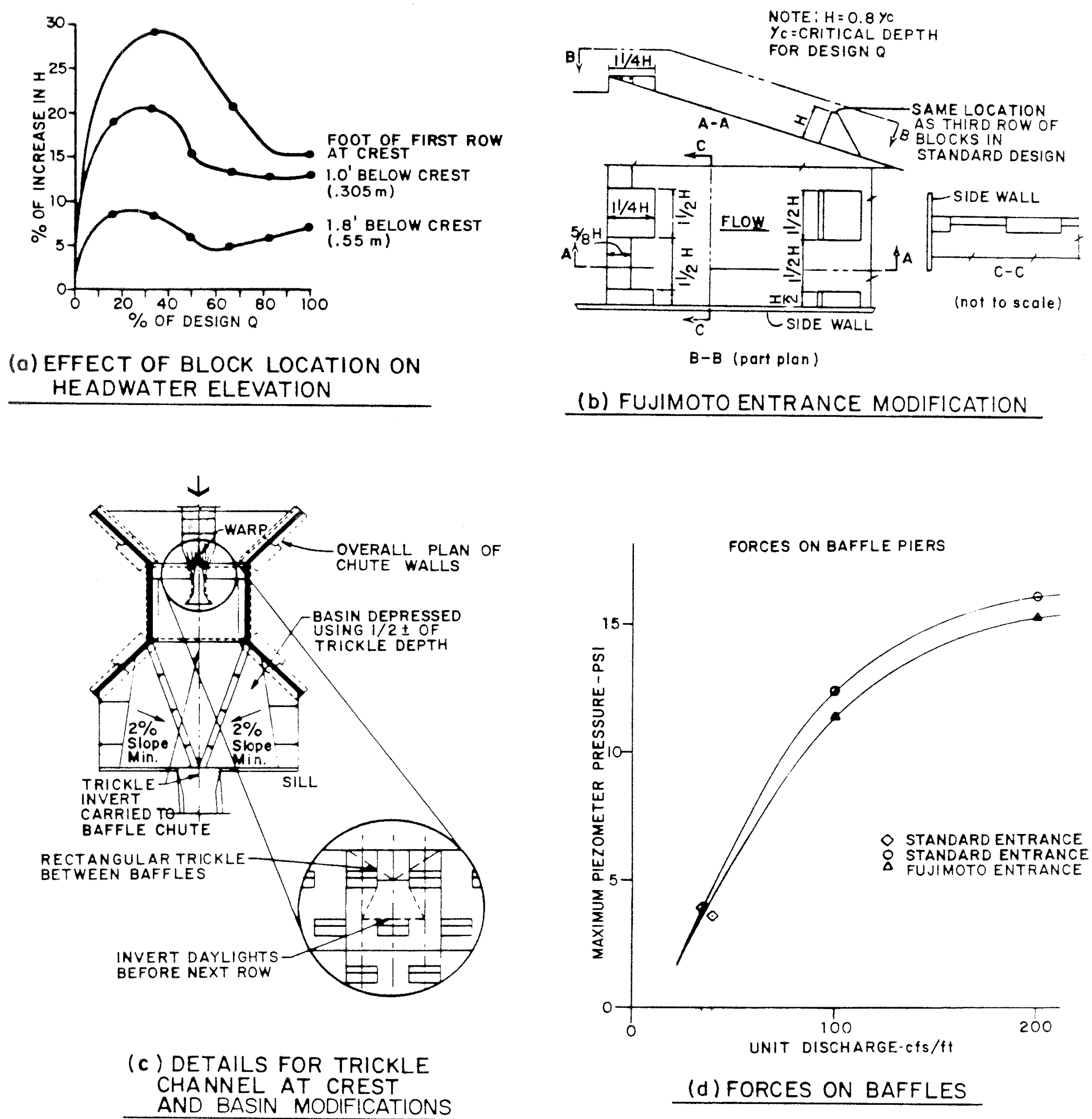


(A) USBR ISOMETRIC



(B) DESIGN CRITERIA

FIGURE 8.13
BAFFLE CHUTE CREST MODIFICATIONS AND FORCES
(ADAPTED FROM: Peterka 1984)



General Hydraulic Design Procedure:

1. Determine the maximum inflow rate and the design discharge per unit width:

$$q = Q/W \tag{8.5}$$

The chute width, W, may depend on the upstream or downstream channel width, the upstream hydraulic control, economy, or local site topography. Generally, a unit discharge between 35 to 60 cfs/ft is most economical.

2. An upstream channel transition section with vertical wing walls, constructed 45 degrees to the flow direction, causes flow approaching the rectangular chute section to constrict. It is also feasible to use walls constructed at 90 degrees to the flow direction. In either configuration, it is important to analyze the approach hydraulics and water surface profile. Often, the effective flow width at the critical cross section is narrower than the width of the chute opening due to flow separation at the corners of the abutment. To compensate for flow separation, it is recommended that the actual width constructed be 1 foot wider than the design analysis width if the constricted crest width is less than 90 percent of the upstream channel flow width. In any case, the design should carefully consider the approach hydraulics and contraction/separation effects. Depth and approach velocities should be evaluated through the transition to determine freeboard, scour, and sedimentation zones.

3. The entrance transition is followed by a rectangular flow alignment apron, typically 5 feet in length. The upstream approach channel velocity, V, should be as low as practical and less than critical velocity at the control section of the crest. Figure 8.12(b) gives the USBR recommended entrance (channel) velocity. In a typical grass-lined channel, the entrance transition to the rectangular chute section will produce the desired upstream channel velocity reduction. The elevated chute crest above the channel elevation, as shown in Figure 8.12(a), should only be used when approach velocities cannot be controlled by the transition. Special measures to prevent aggradation upstream would be necessary with the raised crest configuration.

Entrance Modification:

1. The trickle flow (or low flow) channel should be maintained through the apron, approach, and crest sections. It may be routed between the first row of baffle piers. The trickle channel should start again at the basin rock zone which should be slightly depressed and then graded up to transition to the downstream channel. Figure 8.13(c) illustrates one method of designing the low flow channel through the crest.

2. The conventional design shown in Figure 8.12(a) results in the top elevation of the baffles being higher than the crest, which causes a higher backwater surface effect upstream. Figure 8.12(b) may be used to estimate the extent of the effect and to determine corrective measures, such as increasing the upstream freeboard or widening the chute. Note that baffles projecting above the crest will tend to produce upstream sediment aggradation. Channel aggradation can be minimized by the low flow treatment suggested in the previous paragraph.

Another means of alleviating these problems is the Fujimoto entrance, developed by the USBR and illustrated in Figure 8.13(b). The upper rows of baffles are moved one row increment downstream. The important advantage of this entrance is that there is no back-water effect of the baffles. The serrated treatment of the modified crest begins disrupting the flow entering the chute without increasing the headwater. More importantly, this configuration provides a level crest control. The designer may either bring the invert of the upstream low flow channel into this crest elevation, widening the low flow channel as it approaches the crest, or the designer may have a lower trickle channel and bring it through the serrated crest similar to 1, above. These treatments will have to be observed until more application experience shows what may work best.

Structural Design Dimensions: (see Figure 8.11)

1. Assume critical flow at the crest and determine critical depth for both peak flow and for 2/3 of peak flow. For unit discharge exceeding 60 cfs/ft, Figure 8.12(b) may be extrapolated:

$$y_c = (q^2/g)^{0.33} \tag{8.6}$$

2. The chute section (baffled apron) is concrete with baffles of height, H_b, equal to 0.8 times critical depth. The chute face slope is 2:1 for most cases, but may be reduced for low drops or where a flatter slope is desirable. For unit discharge applications greater than 60 cfs/ft, the baffle height may be based on 2/3 of the peak flow; however, the chute side walls should be designed for peak flow (see number 4).

   Baffle pier widths and spaces should equal, preferably, about 1.5 H_b but not less than H_b. Other baffle block dimensions are not critical hydraulically. The spacing between the rows of baffle blocks should be H_b times the slope. For example, a 2:1 slope makes the row spacing equal to $2H_b$ parallel to the chute floor. The baffle piers are usually constructed with the upstream face normal to the chute floor surface.

3. Four rows of baffle piers are required to establish full control of the flow, although fewer rows have operated successfully. At least one row of baffles are buried in riprap where the chute extends below the downstream channel grade. Riprap protection continues from the chute outlet to a distance of approximately $4H_b$, or as necessary to prevent eddy currents from undermining the walls. Additional rows of baffles may be buried below grade to allow for downstream channel degradation.

4. The baffle chute side wall height (measured normal to the floor slope) should be 2.4 times the critical depth based on peak discharge (or $3H_b$). The wall height will contain the main flow and most of the splash. The design of the area behind the wall should consider that

some splash may occur, but extensive protection measures are not required.

5. Determine upstream transition and apron side wall height as required by backwater analysis. Lower basin wing walls are generally constructed normal to the chute side walls at the chute outlet to prevent eddy current erosion at the drop toe. These transition walls are of a height equal to the channel normal depth plus 1 foot, and length sufficient to inhibit eddy current erosion.

6. All concrete walls and footer dimensions are determined by conventional structural methods. Cutoff walls and underdrain requirements are determined by seepage analysis (see Section 8.3.2).

7. The most troublesome aspect of the design is the determination of the hydraulic impact forces on the baffles to allow the structural engineer to size adequate reinforcing steel. Figure 8.13(d) may be used as a guideline. The structural engineer should apply a conservative safety factor, as this curve is based on relatively sparse information.

Construction Considerations*:*

There are numerous steps necessary in the construction of a baffle chute, but they are usually easily controlled by a contractor. For quality control and inspection, there are consistent, measurable, and repeatable standards to apply.

Potential areas of concern include foundation problems, riprap quality control and placement, and finish work with regard to architectural and landscape treatments. Formwork, form ties, and seal coatings can leave a poor appearance, if not handled properly. Poor concrete vibration can result in surface defects (honeycombing) or more serious conditions, such as exposed rebar.

In summary, baffle chute drop structures are the most successful as far as hydraulic performance is concerned and are straightforward to construct. Steel, formwork, concrete placement and finish, and backfill require periodic inspection.

Vertical Hard Basin Drops

The vertical hard basin is a generalized category which can include a wide variety of structure design modifications and adaptations. A variety of components can be used for both the hard basin and the wall, various contraction effects can be implemented to reduce approach velocities, and different trickle channel options can be selected. The maximum vertical drop height from crest to basin for a vertical hard basin drop should be limited to 2.5 feet for safety considerations subject to the local jurisdiction’s standards. Similarly, a 6-foot apron should be employed for each 2.5 feet of vertical drop. For drops greater than 2.5 feet, a stair step configuration is required.

The hydraulic phenomenon provided by this type of drop is a jet of water which overflows the crest wall into the basin below. The jet hits the hard basin and is redirected horizontally. With sufficient tailwater, a hydraulic jump is initiated. Otherwise, the flow continues horizontally in a supercritical mode until the specific force of the tailwater is sufficient to force the jump. Energy is dissipated in the turbulence through the hydraulic jump; therefore, the basin is sized to contain the supercritical flow and the erosive turbulent zone.

Generally, a rough basin is advantageous since increased roughness will result in a shorter, more economical basin. Figure 8.14 shows a vertical drop with a grouted boulder basin (concrete may also be used), and illustrates several important design considerations.

General Hydraulic Design Procedure:

1. The design approach uses the unit discharge in the main channel and the trickle channel to determine the separate water surface profiles and jump locations in these zones. The basin is sized to adequately contain the hydraulic jump and associated turbulent flows.

2. The rock lined approach length ends abruptly at a structural retaining crestwall which has a nearly rectangular cross section and trickle channel section. (Refer to Section 8.3.2)

3. Crest wall and footer dimensions are determined by conventional structural methods. Underdrain requirements are determined from seepage analysis.

4. *Open Channel Hydraulics* (Chow, 1959), makes a brief presentation for the "Straight Drop Spillway," which applies here. Separate analysis would need to be undertaken for the trickle channel area and the main channel area as discussed in Section 8.3.2. In the following equations add the subscript t for the trickle channel area, and the subscript m for the main channel area.

   Refer to Figure 8.15 to identify the following parameters. L_b is the design basin length which includes L_d and the distance to the jump, D_j, which is measured from the downstream end of L_d. The jump length, L_j, is approximated as six times the sequent depth, Y_2. As a safety factor, to assure a sufficient length for L_b, 0.6 L_j is added in the design of L_b, such that:

$$L_b \geq L_d + D_j + 9.6Y_2 \tag{8.7}$$

FIGURE 8.14
VERTICAL HARD BASIN DROP
(McLaughlin Water Engineers, Ltd., 1986)

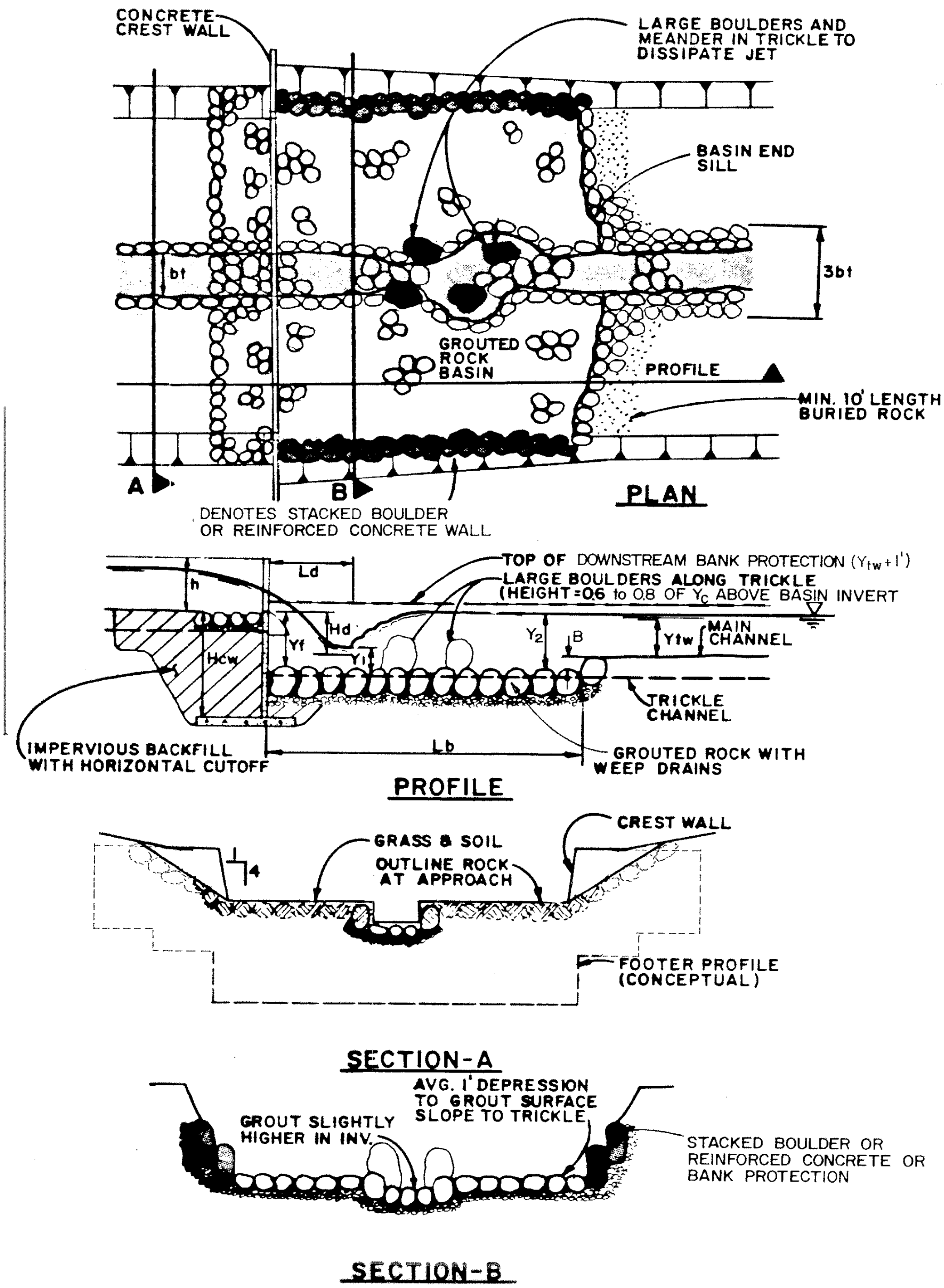
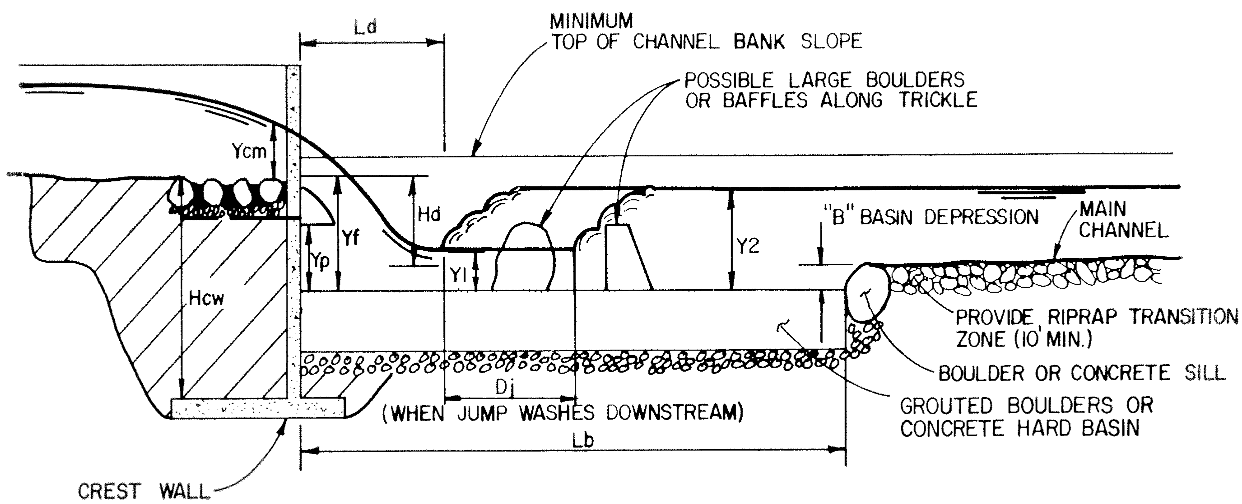


FIGURE 8.15
VERTICAL DROP HYDRAULIC SYSTEM
(McLaughlin Water Engineers, Ltd., 1986)



When a hydraulic jump occurs immediately where the nappe hits the basin floor, the following variables are defined:

$$L_d / Y_f = 4.3 D_n^{0.27} \tag{8.8}$$

where:

$$D_n = q_c^2 / (g Y_f^3) \tag{8.9}$$

$$Y_p / Y_f = 1.0 D_n^{0.22} \tag{8.10}$$

$$Y_1 / Y_f = 0.54 D_n^{0.425} \tag{8.11}$$

$$Y_2 / Y_f = 1.66 D_n^{0.27} \tag{8.12}$$

5. In the case where the tailwater does not provide a depth equivalent to or greater than Y_2, the jet will wash downstream as supercritical flow until its specific force is sufficiently reduced to allow the jump to occur. Determination of the distance to the hydraulic jump, D_j, requires a separate water surface profile analysis for the main and low flow zones. Any change in tailwater affects the stability of the jump in both locations.

6. Caution is advised regarding the higher unit flow condition in the low flow zone. Large boulders and meanders in the trickle zone of the basin are shown to help dissipate the jet,

and rock is extended downstream along the low flow channel. This results in three possible basin length design conditions:

a. At the main channel zone:

$$L_{bm} = L_{dm} + D_{jm} + 0.60(6)(Y_2)_m \quad (8.13)$$

b. At the trickle zone, standard design:

$$L_{bt} = L_{dt} + D_{jt} + 0.60(6)(Y_2)_t \quad (8.14)$$

c. When large boulders or baffles are used to confine the jump to the impingement area of the low flow zone, the low flow basin length may be reduced:

$$L_{bt} = L_{dt} + 0.60(6)(Y_2)_t \quad (8.15)$$

7. The basin floor elevation is depressed at depth B, variable with drop height and practical for trickle flow drainage. Note that the basin depth adds to the effective tailwater depth. The basin is constructed of concrete or grouted rock. Either material must be evaluated for the hydraulic forces and seepage uplift.

8. There is a sill at the basin end to bring the invert elevation to that of the downstream channel and side walls extending from the crestwall to the sill. The sill is important in causing the hydraulic jump to form in the basin. Buried riprap should be used downstream of the sill to minimize any local scour caused by the lift over the sill.

9. Water surface profile analyses have proven that base widths of the rectangular crest which are less than that of the channel will result in high unit discharges and velocities, thereby requiring unreasonable extensions of both the basin length and upstream rock protection. Roughness in the basin area can reduce the basin length required to contain the hydraulic jump. This is the primary advantage of the use of grouted rock in the drop basin.

Construction Considerations:

Foundation and seepage concerns are very critical with regard to the vertical wall, as poor control can result in sudden failure. The use of caissons or pile can mitigate this effect. Put in comparative terms with the baffle chute, seepage problems can result in displacement of the vertical wall with no warning, where the box-like structure of the baffle chute may evidence some movement or cracking, but not total failure, and thus allow time for repairs.

The quality control concerns and measures for reinforced concrete are described under baffle chutes. The foundation concerns for the wall are critical as described above. The subsoil conditions for the basin are also important so that the basin concrete or grouted riprap is stable against uplift pressures.

A grouted boulder stilling basin provides roughness, which is useful in shortening the basin length. As the name implies, the basin should be constructed of individual boulders placed on a prepared subgrade. Boulders should be a minimum dimension that exceeds the grout layer thickness, so that the contractor and the inspector can see and have grout placed directly to the subgrade and completely filling the voids. Graded riprap should not be used for grouting, as the smaller rock prevents the voids from being completely filled with grout. The result is a direct piping route for water beneath the grout, and a structural slab with insufficient mass. The completed combination of boulders and grout should have an overall weight sufficient to offset uplift forces. A minimum dimension of 18 inches is recommended for boulders, and 12 inches for the grout layer. By maintaining the finished surface of the grout below the top of the boulder, both appearance and roughness characteristics are enhanced. Seepage relief for the basin slab should be provided.

This type of structure has a moderate level of construction difficulty. The wall, once foundation conditions are addressed, is conventional construction. It is very possible for the construction of the seepage control and earthwork to go awry and problems to go undetected until the time of failure. The flat concrete or grouted rock placement is easier for the contractor than graded rock placement/quality control, but again poor placement and undetected subsoil, bedding or rock problems can result in failure. Thus, it is easier than many other types to construct, but susceptible to some hidden risks and problems.

Vertical Riprap Basin Drops

As shown in Figure 8.16, this structure is essentially a plunge pool drop that incorporates a reinforced concrete crest wall with a riprap lined dissipation pool below. A nearly rectangular crest section is recommended to reduce the width of the plunge pool. Maximum drop depth for a vertical riprap basin should be limited to 2.5 feet due to safety considerations and the practicality of obtaining the larger riprap needed for higher drops. This height limitation is subject to the standards evoked by the jurisdictional entity. Submergence by high tailwater can limit the dissipation efficiency.

The hydraulic design was developed through model testing by Smith and Strang in 1967 (*Scour in Stone Beds*) and design procedures were further developed by Stevens in 1981 (*Hydraulic Design Criteria for Riprapped Chutes and Vertical Drop Structures*).

In this structure, flow passing over the vertical crest wall plunges into a riprap basin area. Energy is dissipated by turbulence in the plunge pool. Loose riprap is placed in the basin according to the initial design specifications. The rock is successively rearranged by inflows until a more stabilized basin plunge pool is formed. The depth of the scour hole, d_s, and the nominal rock size are inversely related.

Structural design for the vertical crest wall is complicated by the lack of downstream support, seepage, soil saturation and hydraulic loading on the upstream side. In sandy or erosive soils, it is common to use sheet pile for the crest wall construction, while caissons may be an acceptable foundation for certain other applications. A concrete retaining wall is frequently selected for ease of construction, seepage control and low maintenance.

General Hydraulic Design Procedure:

The hydraulic analysis of this type of drop is generally similar to that presented previously in this section for crest hydraulics. The design of the flexible plunge pool basin is described below.

The desired drop across the structure is the difference in the bed elevations of the approach channel at the weir and the downstream channel at the end of the structure. Let this difference be H_d. It follows from Figure 8.16 that:

$$H_d = D - 0.67 d_s \tag{8.16}$$

The designer must find the combination of rock size and jet plunge height D that gives a depth of scour which balances Equation (8.16). The relation between rock size d_{50}, jet plunge height D, head on the weir, H, ($H = 1.5 y_c$) and depth of scour d_s is given in Figure 8.17. As these values will be different in the main drop and the trickle, the design d_{50} and/or d_s will vary.

To obtain an adequate cutoff, the depth of the vertical wall that forms the weir crest must extend below the bottom of the excavation for the riprap. Thus, it usually becomes uneconomical to design a scour depth d_s, any greater than 0.3 D. To meet this limitation in the field it is necessary to: increase the rock size d_{50}; decrease the jet plunge height D (by using more drops); decrease H (by using a wider structure); or, to use another type of drop structure.

The side slopes in the basin must be riprapped also as there are strong back currents in the basin. Granular filter material is required under this riprap. The side slopes in the basin should be the same slope as for the downstream channel.

FIGURE 8.16
VERTICAL RIPRAP BASIN DROP
(Stevens, 1981)

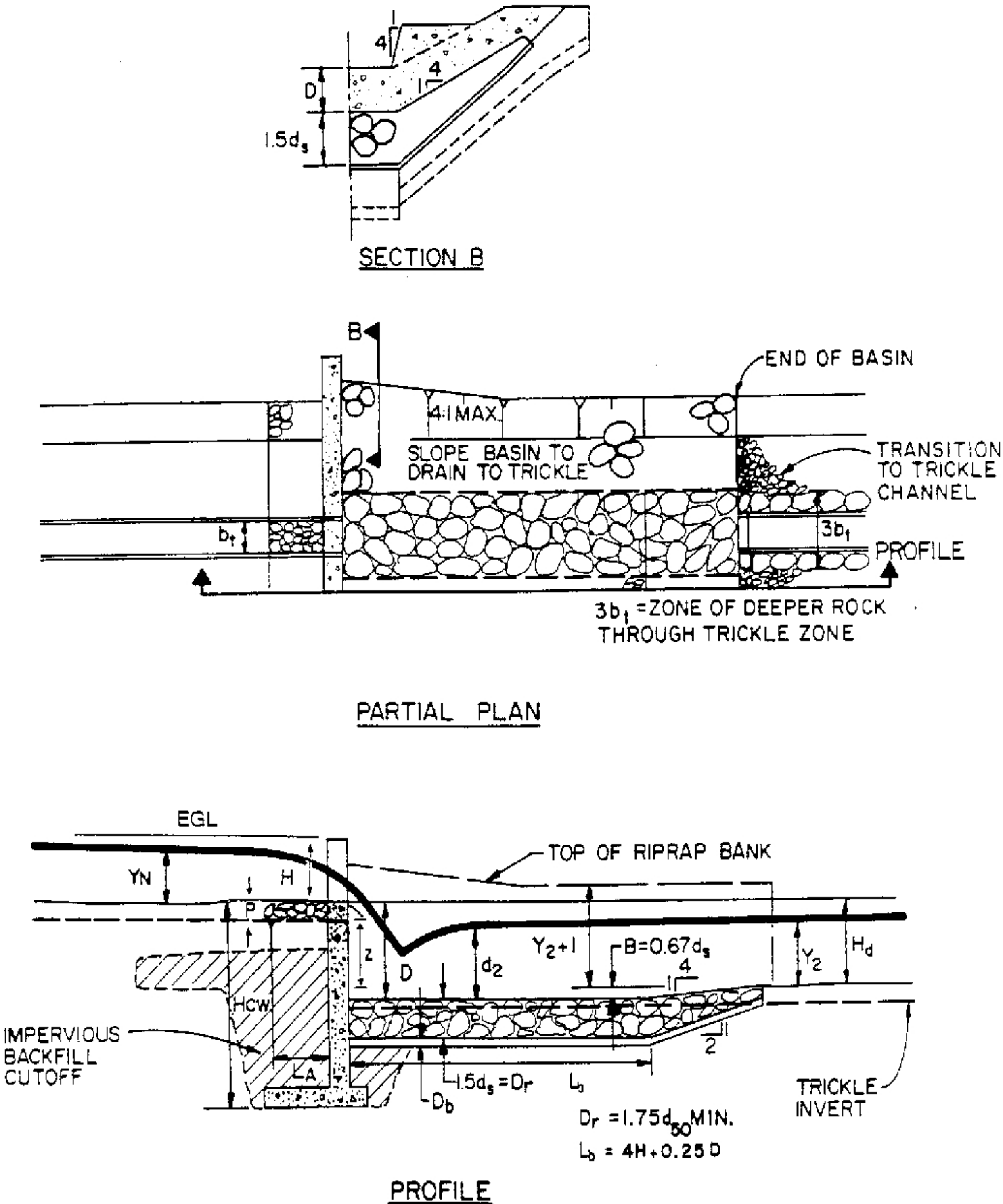
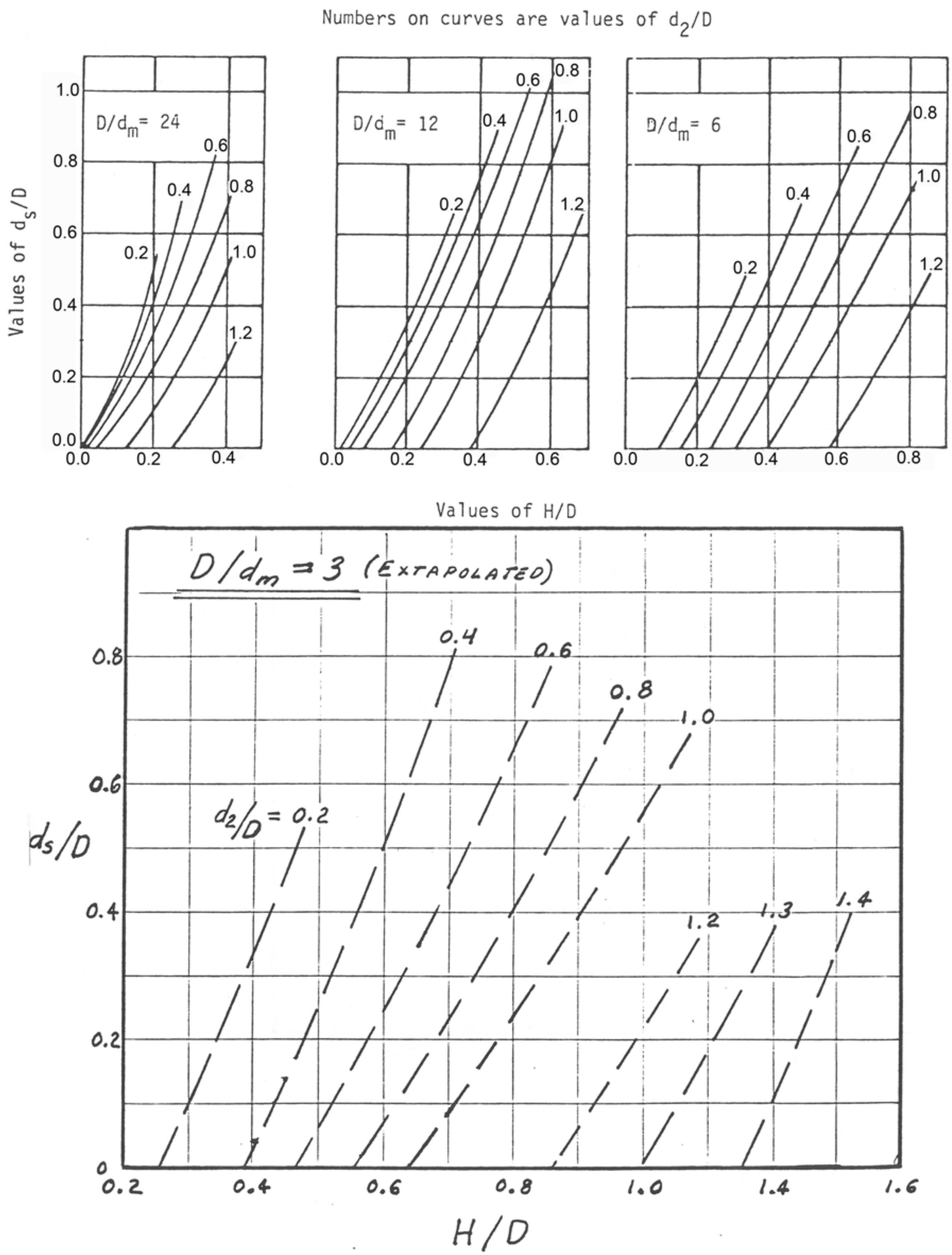


FIGURE 8.17
CURVES FOR SCOUR DEPTH AT VERTICAL DROP
(Stevens, 1981)



Construction Considerations:

Foundation and seepage concerns are critical with regard to the vertical wall in this type of drop. They are also generally more critical than with an equivalent vertical drop into a hard basin because the riprap basin may scour and reshape, leaving less supporting material on the downstream side. Thus, if seepage is worse than anticipated, backfill is poor, or if seepage control measures are not functioning, an immediate and severe structure stability problem can occur. The use of caissons or piles can mitigate this effect. Seepage problems can result in displacement of the vertical wall with no cracking as an advance warning. Seepage can also cause piping failure where the water will actually flow under the vertical wall. Problems can result from rock that does not meet specifications for durability, specific gravity or gradation. Quality control of rock installation can be difficult in regard to measuring performance and maintaining consistency. Undersized rock in the plunge pool basin can cause the basin to reshape differently than designed and result in stability problems for the wall, the basin, and the downstream channel.

This type of structure has a moderate level of construction difficulty. The wall, once foundation conditions are addressed, is straightforward. It is very possible for the construction of the seepage control and earthwork to go awry and for problems to go undetected until the time of failure. The flat riprap placement is easier than sloping, but again poor placement and undetected subsoil, bedding, or rock problems can all contribute to failure.

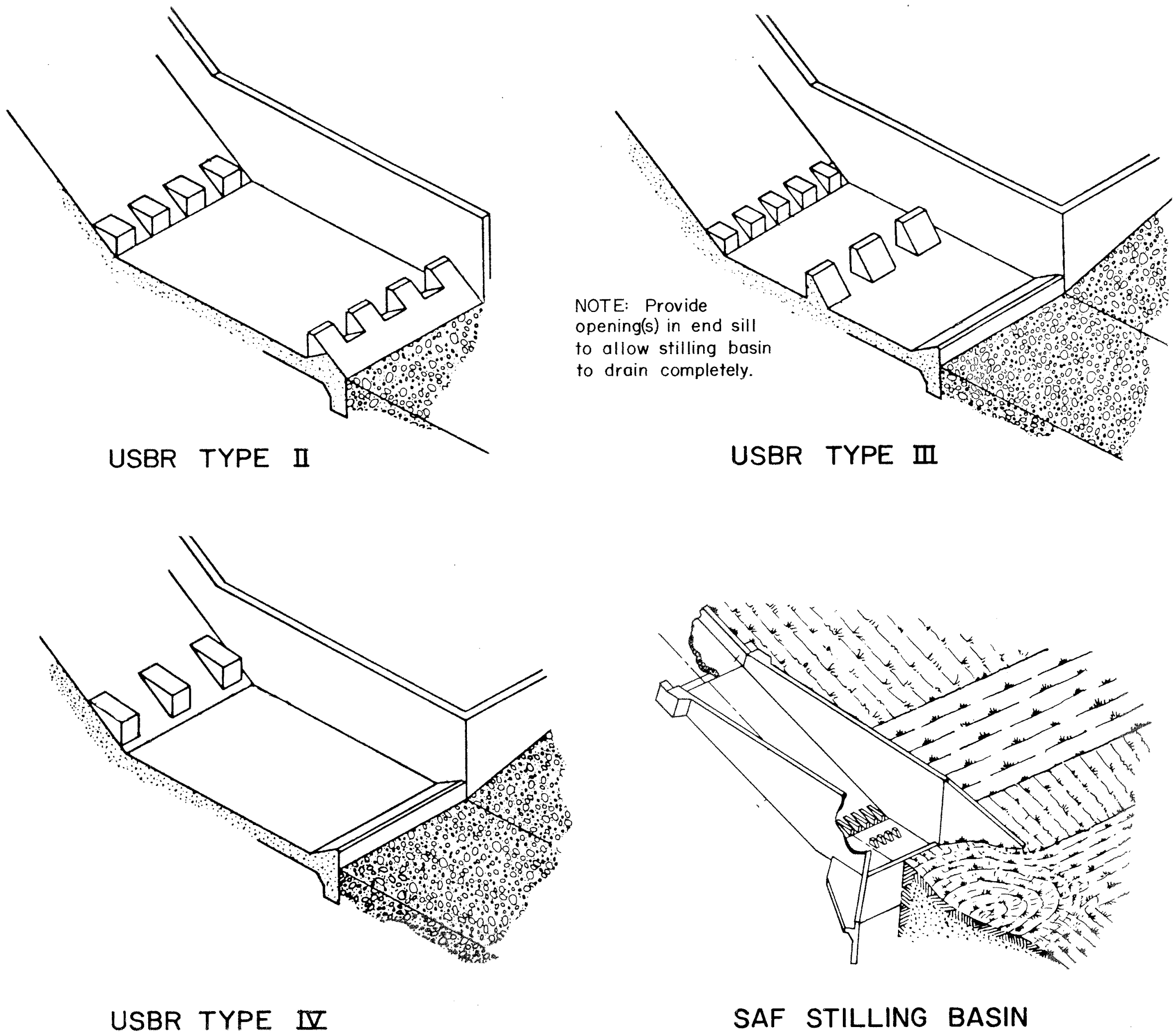
Sloping Concrete Drops

The hydraulic concept of these structures is to dissipate energy by formation of a conventional hydraulic jump, usually associated with a reverse current surface flow as the supercritical flow down the face converts to subcritical flow downstream.

Numerous concepts have been investigated. Among them are the Saint Anthony Falls (SAF) Stilling Basin, and the USBR Basins I, II, III, and IV (USDOT, 1983; and Peterka 1984). These drops and associated basins are suited for different kinds of situations.

The SAF and the USBR Basins (with the exception of Type I) all work at techniques to shorten the basin length. In the USBR Basin I, no special measures are provided. On the smooth concrete basin it can take considerable basin length to "burn off" enough energy to dissipate the supercritical flow of where a jump will begin, and then more length to allow for the turbulence of the jump. Basin I is relatively expensive because of its length. The other basins require a certain amount of tailwater, which requires depressing the basin, and the use of baffles or other shapes to allow shorter basins, related dissipation, and control of troublesome wave patterns. Figure 8.18 illustrates the various types of stilling basins for use with sloping concrete drops.

FIGURE 8.18
STILLING BASINS FOR SLOPING CONCRETE DROPS
(ADAPTED FROM: USDOT, FHWA, HEC-14, 1983)



General Hydraulic Design Procedure:

Design procedures for USBR Basins II, III, and IV and the SAF Stilling Basin are presented in *Hydraulic Design of Energy Dissipators for Culverts and Channels* (USDOT, 1983) and *Hydraulic Design of Stilling Basins and Energy Dissipators* (Peterka, 1984).

Analysis of channel approach and crest hydraulics generally follows the guidelines presented in Section 8.3.2. Once water surface profiles have been determined, including tailwater determination and supercritical water surfaces down the sloping face, seepage uplift forces must be evaluated. Net uplift forces vary as a function of location along the drop, cutoff measures, drain gallery locations and water surface profiles through the basin.

For a stable structure, net uplift force from seepage must be countered by net forces in the downward direction. For a smooth concrete chute, downward forces are the buoyant weight of the concrete structure and the weight of water (a function of the depth of flow). Significant pressure differentials can occur with a combination of high seepage forces and shallow supercritical flow. Seepage analyses should be conducted using Lane's weighted creep methodology (Section 8.3.2), and suitable countermeasures designed. Such measures include cutoff walls, weep drain galleries and concrete slab thickness design. A range of flood discharges should be evaluated, since differential pressure relationships can vary with flow depth and location of hydraulic jump.

Construction Considerations:

There may be applications where sloping concrete drops are advantageous, but generally other drops such as baffle chutes or vertical drops are more appropriate for a wider range of applications. The design guidance provided by the literature is clear and relatively easy to use, but the implementation is often difficult or impractical. This basically has to do with providing basin depth without creating a maintenance problem and less flexibility in adapting to varying bed conditions.

The integrity of the cutoff is important as seepage and resultant uplift forces are key concerns. Uncontrolled underflow could easily lift a major concrete slab.

The stilling basin should be designed to drain completely, to eliminate nuisances related to ponded water, such as mosquito breeding and sediment/debris accumulation.

Considerations relating to general concrete construction are the same as discussed previously for baffle chute drops. Public acceptability is likely to be low in urban areas, as the sloping concrete face is inviting for bicyclists, roller skaters, and skateboard enthusiasts.

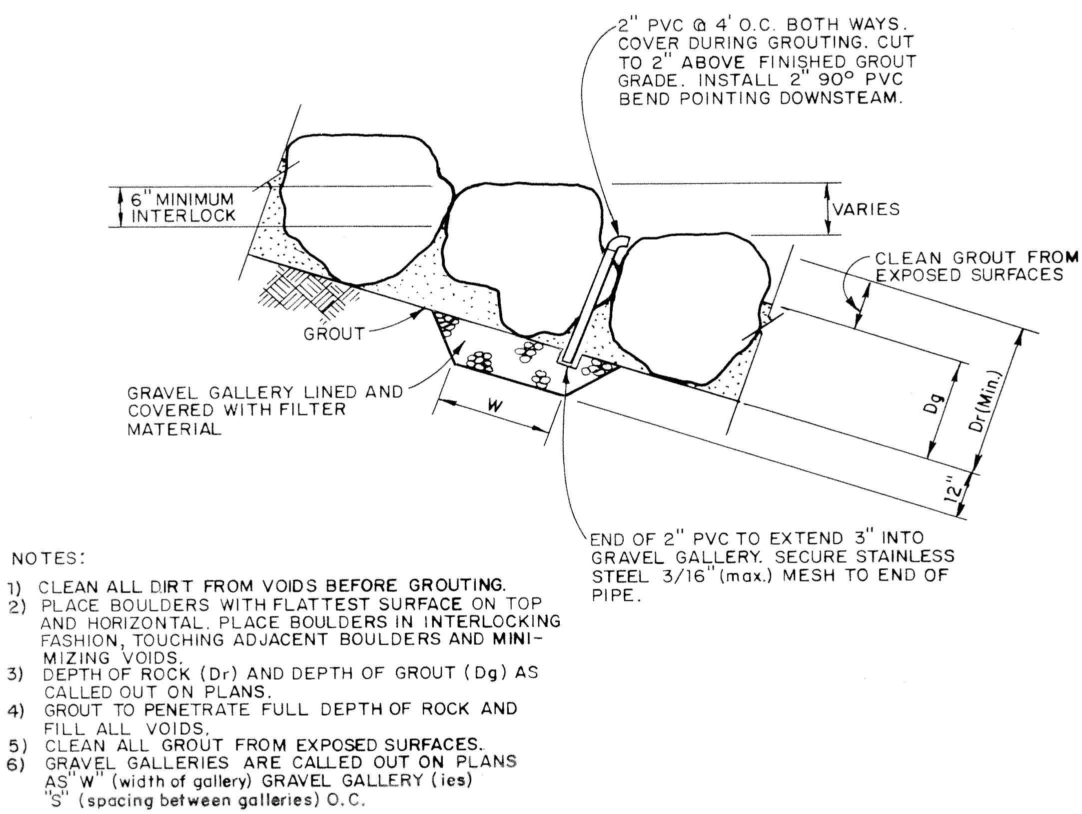
Other Types of Drop Structures

There are numerous other types of drop structures for specific applications in drainage design. The four types of structures presented above are appropriate for the majority of situations to be encountered in Maricopa County. Some possible variations or modifications are presented below along with a few specialized types.

Sloping Drop Variations - The use of soil cement, roller compacted concrete, and grouted boulders are possible variations in sloping drop design. The primary concern with soil cement is its ability to resist the high abrasive action of turbulent flow associated with a drop structure. Adequate countermeasures would be required to demonstrate the suitability of soil cement prior to its approval for use on drop structures.

Addition of roughness elements on the face of a sloping concrete drop can provide increased energy dissipation. "Stepped" concrete has been successfully applied at spillways and drop structures. Roller compacted concrete is a methodology that can achieve the stairstep geometry on the face of a sloping drop. Reinforced concrete steps can be constructed by standard construction methods on small structures. Stepped drop structures have been found to be effective in dissipating the energy associated with low flows but fail to effectively dissipate energy of higher flows. Thus, stilling basin length for a stepped drop structure will be based upon the conventional length calculations for a sloping drop presented herein. Stepped drop structures will be no steeper than 2H:1V with a step height no greater than 2.5 feet and a step apron length of 6 feet. Construction of a drop with grouted boulders is another means of creating desirable roughness on the sloping face and in the stilling basin (see Figure 8.19).

FIGURE 8.19
GROUTED BOULDER PLACEMENT
(McLaughlin Water Engineers, Ltd., 1986)



However, because the structure is comprised of a structural slab with two components (boulders and grout), great care must be taken to design the structure to withstand uplift and to specify boulder and grout material to assure full quality control in the field. Seepage analysis is required to determine a compatible combination of cutoff depth, location of the toe drain and/or other drains, and the thickness of rock and grout. Problems with rock specific gravity, durability and hardness are of concern. Gradation problems are largely eliminated because the boulders are specified to meet minimum physical dimensions and/or weights, which is much easier to observe and enforce in the field than with graded riprap.

The handling of the large boulders requires skilled work force and specialized equipment. Equipment similar to logging tongs, and specially modified buckets with hydraulically powered "thumbs" have been used in recent years and have greatly improved quality and placement rates. The careful placement of stacked boulders, so that the upstream rock is keyed in behind the downstream rock, and placed with a large flat surface horizontally, has been shown to be successful.

The greatest danger lies with a "sugar coated" grout job, where the grout does not penetrate the voids between the rock and the subgrade, leaving a direct piping route for water under the grout. This can easily occur when attempting to grout graded riprap, thus the need to use individual boulders that are larger in diameter than the grout layer so that the contractor and the inspector can see and have grout placed directly to the subgrade. The best balance appears to be boulders 33 to 50 percent greater in size than the grout thickness, but of an overall weight sufficient to offset uplift. Also, when holding grout to this level, the appearance will be much better.

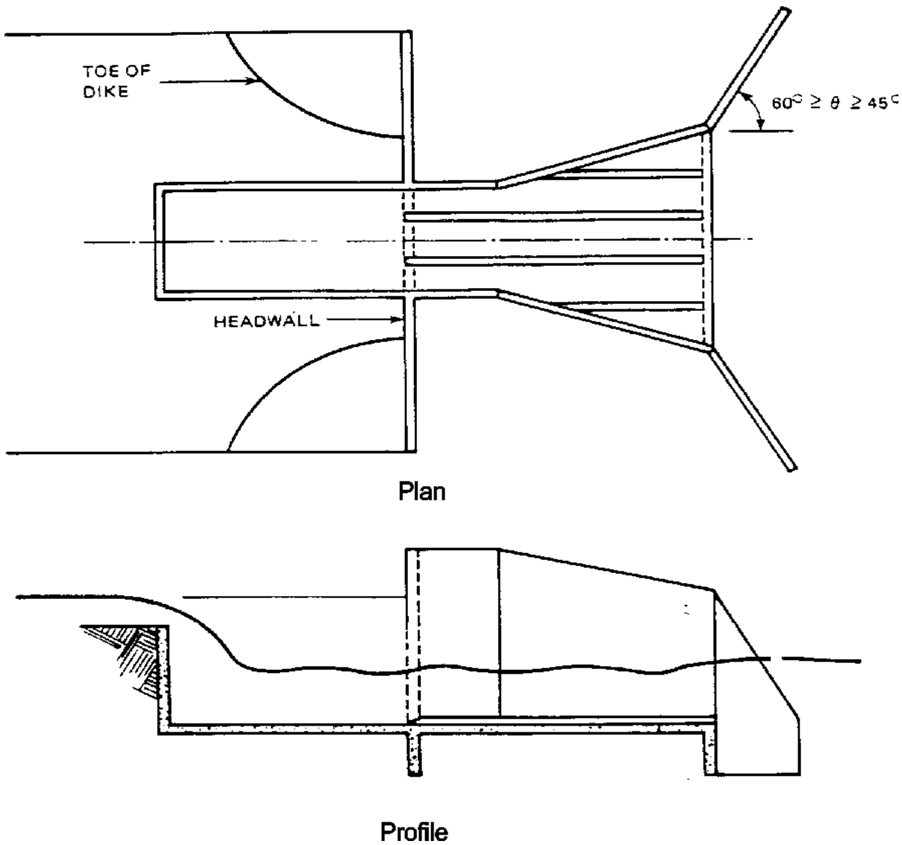
The grout should have a minimum 4,000 psi compressive strength at 28 days, stone aggregate with a maximum dimension of one-half inch and a slump within a range of 4 to 7 inches. The water/cement ratio should not exceed 0.48.

Other USBR Basins - Some other stilling basins developed by the USBR (Peterka, 1984) have limited application. For example, Basin I is basically a horizontal concrete apron downstream of a sloping or vertical drop. This type of basin is applicable only to a concrete lined channel, and, as the USBR states, has wave problems that are difficult to overcome. Maintenance of sufficient tailwater depth is important to cause a hydraulic jump within a practical zone close to the toe of the drop. Generally, other types of USBR basins are better alternatives to Basin I.

USBR Basin V is a stilling basin with sloping apron, and provides dissipation as effective as that which occurs in the basin with a horizontal apron. Again, adequate tailwater is a must. This type of structure would have an application as a spillway into a pond with a permanent pool, so that minimum tailwater is essentially guaranteed.

Box Inlet Drop Structure - The box inlet drop structure may be described as a rectangular box open at the top and downstream end (Figure 8.20). Water is directed to the crest of the box inlet by earth dikes and headwalls. Flow enters over the upstream end and two sides and leaves the structure through the open downstream end. The long crest of the box inlet permits large flows to pass at relatively low heads. The width of the structure does not need to be greater than the downstream channel. It is applicable for drops from 2 feet to 12 feet. Designers of box inlet drop structures should review permissible drop heights allowed by governing jurisdictions as safety issues need to be considered.

FIGURE 8.20
BOX INLET DROP STRUCTURE
(ADAPTED FROM: USDOT, FHWA, HEC-14, 1983)



The outlet structure can be adjusted to fit a wide variety of field conditions. It is possible to lengthen the straight section and cover it to form a highway culvert. The sidewalls of the stilling basin section can be flared if desired, thus permitting use with narrow channels or wide floodplains. Flaring the sidewalls also makes it possible to adjust the outlet depth to match the natural channel.

Design guidelines are presented in *Hydraulic Design of Energy Dissipators for Culvert and Channels* (USDOT, 1983).

Grade Control Structures

Grade control structures can be effective in stabilizing natural channels and other unlined channels. These structures are designed to provide control points to maintain stable bed slopes. They do not stabilize channel side slopes. Set at grade across the channel/floodplain, these structures do not serve to change the velocity profile of the flow regime, but rather, serve as a barrier to headcutting. Here, headcutting is defined as the scouring of the channel bed proceeding from a downstream to upstream direction. Local soils, bed materials, and sediment gradation must be considered along with channel hydrology and hydraulics for the effective design of a grade control structure (See Chapter 5 and Chapter 11 for further discussion on sediment transport and estimating scour depth). The longevity of the structure is dependent upon the depth of toe down (among other things), which must exceed the depth of scour in order to stabilize the channel slope upstream of the structure. The potential for seepage cutoff must be assessed for hydrostatic pressure and the potential failure of the structure foundation due to "piping" of the underlying soils. If an issue, the appropriate engineered solutions should be employed in the design. These solutions include the use of geotextile filter fabrics to prevent soil loss and small diameter PVC pipes to relieve hydrostatic pressure. In any case, appropriate access to grade control structures is necessary to permit intermittent maintenance.

8.4 ENERGY DISSIPATION STRUCTURES AT CULVERT OUTLETS

8.4.1 General

This section is applicable to both culvert and storm drain outlets. Outlet structures can provide a high degree of energy dissipation and are generally effective even with relatively low tail water control. Rock protection at culvert outlets is appropriate where moderate outlet conditions exist; however, there are many situations where rock aprons or basins are impractical even at low to moderate flow conditions. Concrete energy dissipation or stilling basin structures are then required to prevent scour damages caused by high exit velocities and flow expansion turbulence at culvert outlets. Concrete outlet structures can be designed easily and are suitable for a wide variety of site conditions. In some cases, they are more economical than large rock basins, particularly where long term costs are considered. Covered in this section are both riprap and concrete energy dissipators.

8.4.2 Riprap at Culvert Outlets

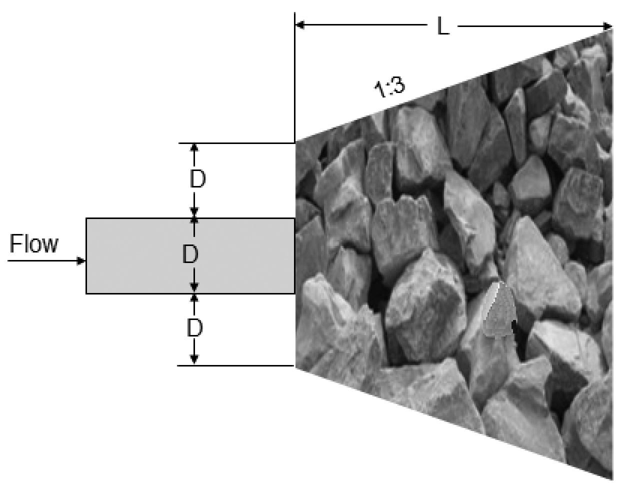
Two types of riprap protection for culvert outlets are recommended: 1) the riprap apron and 2) the riprap basin. In general, when the diameter for a circular culvert or equivalent diameter for a non-circular culvert is equal to or less than 60 inches, a riprap apron at the outlet may be used. The riprap basin approach shall be used for pipes larger than 60-inch diameter and may also be used for pipes smaller than 60 inches in diameter or equivalent. The equivalent diameter for a non-circular culvert can be computed by $\sqrt{4A/\pi}$ where A is the cross section area for a non-circular culvert. The threshold of 60-in diameter is based on HEC-14 (USDOT, 2006). It is a general guideline but not an absolute dividing line between riprap apron and riprap basin. The area for a 60-in diameter pipe is about 20 square feet.

The following design procedures are directly applicable to culverts that are installed inside a channel for a roadway crossing. The procedures are also applicable to culverts that outfall to a detention basin or a channel where the velocity is so small that it will not move the riprap. If culverts outfall to a channel as a side drainage system, the riprap must be large enough to withstand the flow in the channel. The riprap sizing equation for channel flow can be found in Channel Bed on Straight Reach or Channel Bed on Curved Reach in Section 6.6.3. The larger riprap size between the riprap apron/basin approach and the channel flow riprap sizing method should be used. When the riprap for channel flow is larger than that for the riprap basin approach, the riprap basin parameters should be adjusted based on the larger median riprap

Riprap Apron

The material for a typical riprap apron is loose angular riprap. The apron should be level wherever possible, or set at the streambed slope. The median diameter for riprap (d_{50}) can be estimated by the loose riprap sizing equation for Channel Bed on Straight Reach in Section 6.6.3. When a highly turbulent flow is expected at the culvert outlet, the loose riprap sizing equation for Downstream of Grade Control/Drop Structure in Section 6.6.3 should be used to compute d_{50}. When riprap size becomes too large to be practical, the culvert should be re-designed to reduce the culvert exit velocity. The other riprap gradation limits (d_{100}, d_{85}, and d_{15}) can be found in Table 6.4. The apron has a fan-shape as shown in Figure 8.21 with a 3:1 expansion ratio. "D" in Figure 8.21 represents the diameter for a circular pipe or equivalent diameter for a non-circular culvert.

FIGURE 8.21
RIPRAP APRON PLAN VIEW



The apron length and thickness can be estimated based on Table 8.6 (USDOT, 2006). For a d_{50} size not listed in Table 8.6, a linear interpolation can be performed to determine the apron length and apron thickness.

TABLE 8.6
APRON LENGTH AND THICKNESS
(USDOT, 2006)

d_{50} (in)	Apron Length (L, ft)	Apron Thickness (ft)
5	4D	3.5 d_{50}
6	4D	3.3 d_{50}
10	5D	2.4 d_{50}
14	6D	2.2 d_{50}
20	7D	2.0 d_{50}
22	8D	2.0 d_{50}

The apron width at the outlet should be 3 times the diameter for a circular culvert or equivalent diameter for a non-circular culvert. The apron width at the apron downstream end can be estimated by $3D + 2L/3$ where D is the diameter or equivalent diameter and L is the apron length.

The apron shall have a filter beneath the riprap. The filter can be a fabric filter or granular filter. Refer to Riprap Layer Characteristics in Section 6.6.3 for the filter design procedure.

Grouted riprap may also be used for the riprap apron for side inlet outfall erosion protection. The grouted riprap apron bed should have a zero slope. The apron should also be fan-shaped as shown in Figure 8.21 using the same expansion ratio. The apron length should be at least $8D$ where D is the diameter for a circular culvert or equivalent diameter for a non-circular culvert. The width at the beginning of the apron is at least $3D$. The width at the downstream end of the apron is at least $3D + 2L/3$ where L is the apron length. Riprap sizing and gradation limits can be found in Section 6.6.5 Grouted Riprap Lined Channels. Concrete turndowns to total scour depth and 1 foot wide shall be constructed around the entire perimeter of the grouted riprap apron. If the total scour depth is large, a 2.5-ft deep concrete turndown may be used with loose angular riprap around the perimeter of the concrete turndown. The volume of riprap should be sufficient to launch to the total scour depth. Concrete grout mix shall be an 8 sack mix per MAG Specification 220.6 (MAG, 1998). A filter blanket underneath the grouted riprap apron is not necessary. Additional loose angular riprap 3 feet wide may be needed around the turndown to ensure that the turndown will be protected from channel flow if the total scour depth is more than 2.5 feet. The depth of the additional loose angular riprap should be based on the channel total scour depth computed using the procedures in Section 11.8.2. The loose angular riprap design can be based on toe protection requirements in Section 6.6.4.

Riprap Basin

The following design procedures for a riprap basin are adapted from Chapter 10, USDOT, 2006. The procedures are applicable for both circular and box culverts. The typical riprap basin is shown on Figure 8.22 and Figure 8.23. Either angular or rounded riprap may be used. A fabric or granular filter blanket shall be installed beneath the riprap. The filter design procedure is defined in Riprap Layer Characteristics in Section 6.6.3.

FIGURE 8.22
PROFILE OF RIPRAP BASIN
(USDOT, 2006)

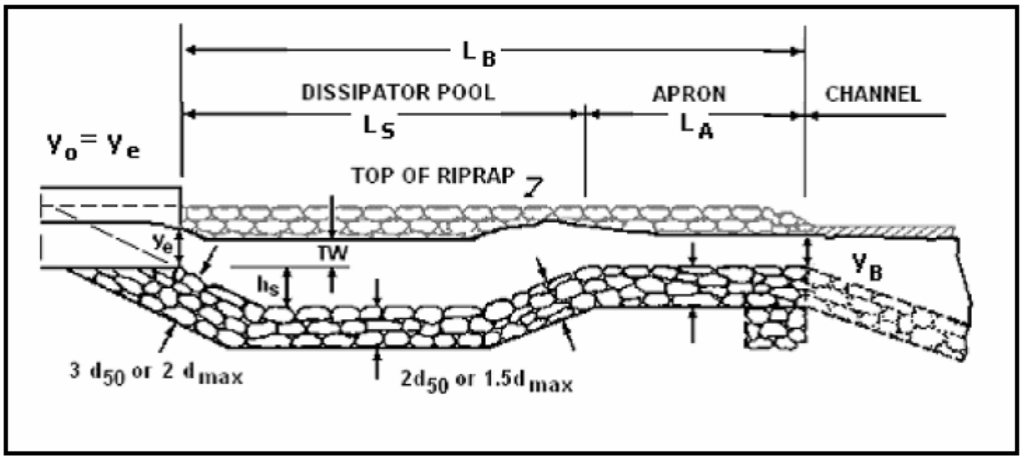
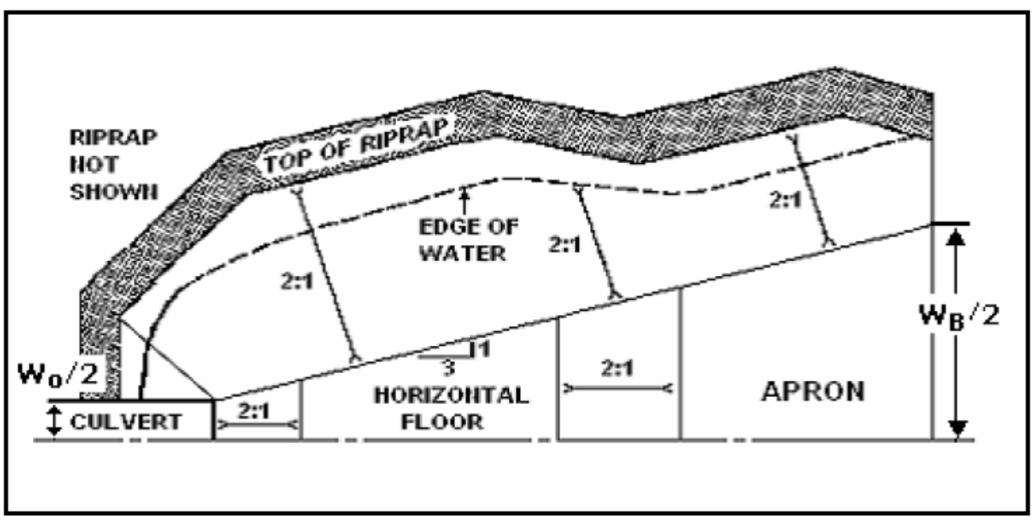


FIGURE 8.23
HALF PLAN OF RIPRAP BASIN
(USDOT, 2006)



Step 1. Compute the culvert outlet flow depth (y_o) and velocity (V_o).

For subcritical flow (culvert on a mild or horizontal slope), use Figure 8.24 for rectangular culverts or Figure 8.25 for circular culverts to obtain y_o/D and then compute y_o. TW stands for tailwater depth and can be estimated by using Manning's equation for the channel section downstream of the culvert. V_o is then computed by dividing Q by the wetted area associated with y_o. D is the height of a box culvert or diameter of a circular culvert.

For supercritical flow (culvert on a steep slope), y_o and V_o can be found by solving the Manning's equation inside the culvert. Compute the Froude number to verify it is a supercritical flow condition by $Fr = V_o/(gy_e)^{1/2}$, for brink conditions using brink depth for box culverts ($y_e = y_o$) and equivalent depth ($y_e = (A/2)^{1/2}$) for non-rectangular sections where A is the wetted cross section area inside the culvert at y_e.

Step 2. Estimate the median riprap size (d_{50}) and Dissipator Pool Depth (h_s).

d_{50} and h_s can be estimated by the trial and success method on d_{50} based on the following equation until the $h_s/d_{50} \ge 2$ and $d_{50}/y_e \ge 0.1$ conditions are both met.

$$\frac{h_s}{y_e} = 0.86\left(\frac{d_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_0 \qquad (8.17)$$

where:

h_s = dissipator pool depth, ft,

y_e = equivalent brink (outlet) depth, ft, which can be estimated by $y_e = (A/2)^{0.5}$ for non-rectangular culverts,

A = wetted cross sectional area inside the culvert at y_e, ft^2,

d_{50} = median rock size by weight, ft, and

C_0 = the tailwater parameter.

When the consequences of riprap basin failure are severe, the following tailwater parameter C_0 for the envelope design relationship should be used, which is defined as:

$C_0 = 1.4$ for $TW/y_e < 0.75$

$C_0 = 4(TW/y_e) - 1.6$ for $0.75 < TW/y_e < 1.0$

$C_0 = 2.4$ for $1.0 < TW/y_e$

where TW is the tailwater depth (ft), which can be estimated by using Manning's equation for the channel section and y_e is the equivalent outlet depth (ft).

FIGURE 8.24
DIMENSIONLESS RATING CURVES FOR THE OUTLETS OF RECTANGULAR CULVERTS
ON HORIZONTAL AND MILD SLOPES
(Simons, et al., 1970)

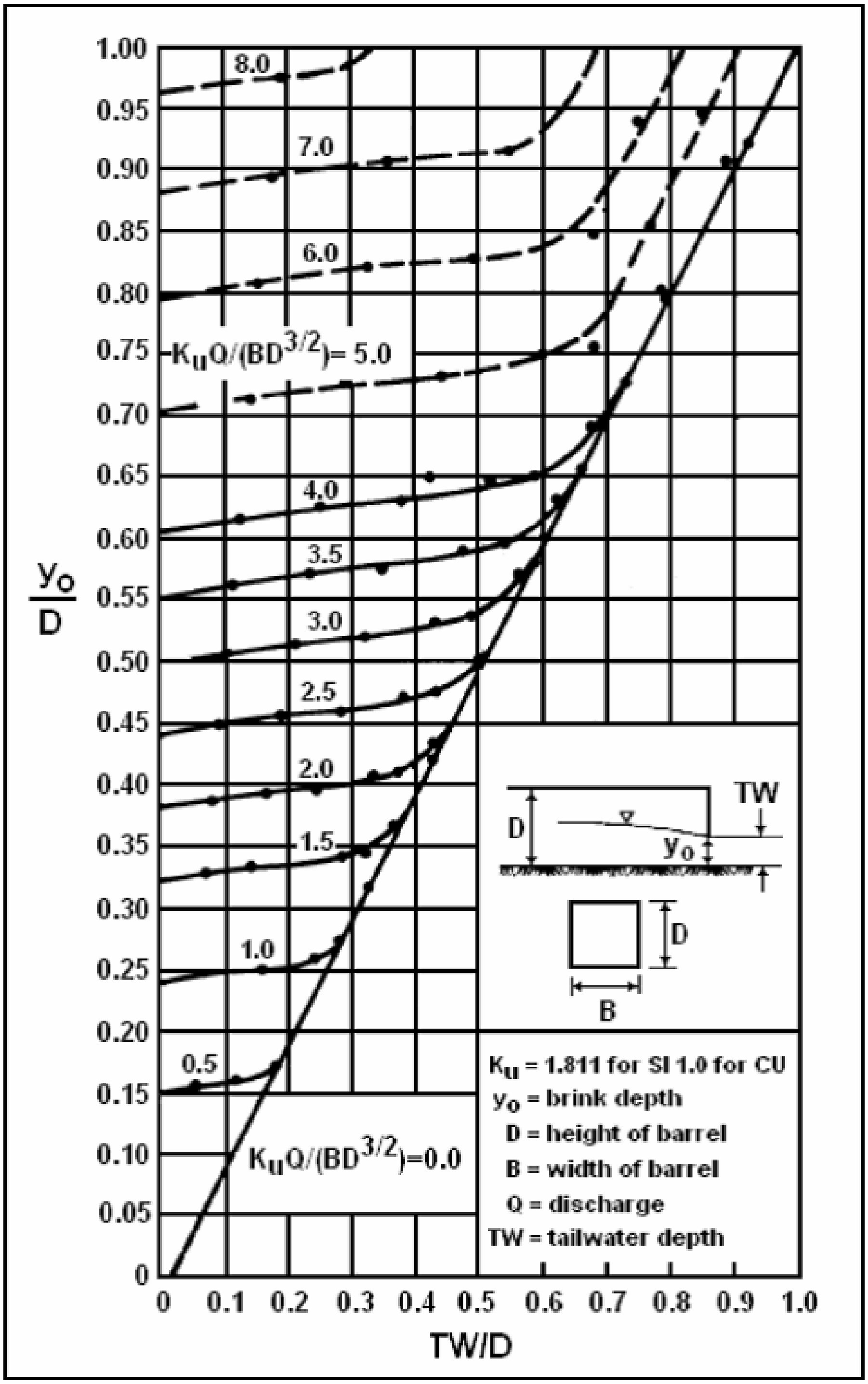
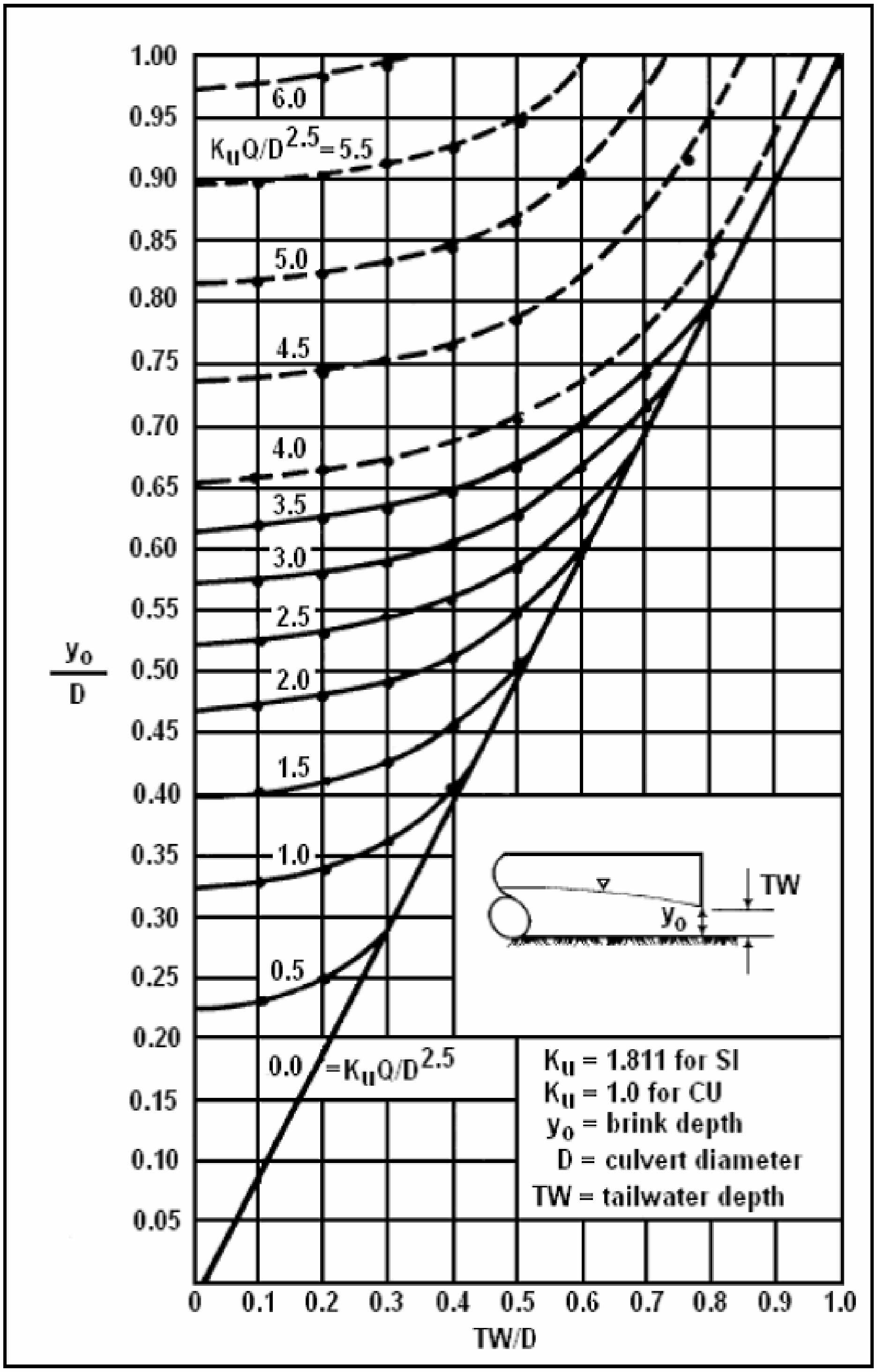


FIGURE 8.25
DIMENSIONLESS RATING CURVES FOR THE OUTLETS OF CIRCULAR CULVERTS
ON HORIZONTAL AND MILD SLOPES
(Simons, et al., 1970)



When a riprap basin failure can be easily addressed as part of routine maintenance, the tailwater parameter for the best fit design relationship should be used, which is defined as:

$$C_0 = 2.0 \qquad \text{for} \quad TW/y_e < 0.75$$

$$C_0 = 4(TW/y_e) - 1.0 \qquad \text{for} \quad 0.75 < TW/y_e < 1.0$$

$$C_0 = 3.0 \qquad \text{for} \quad 1.0 < TW/y_e$$

Basins sized where $h_s/d_{50} \geq 2$ is greater than, but close to, 2 are often the most economical choice (USDOT, 2006). Either angular or rounded riprap can be used (USDOT, 2006). The recommended riprap gradation limits (d_{100}, d_{85}, and d_{15}) can be found in Table 6.4. The riprap basin shall have a filter beneath the riprap. The filter can be a fabric filter or granular filter. The riprap gradation limits and the filter design information are presented in Riprap Layer Characteristics in Section 6.6.3.

Step 3. Determine riprap basin dimensions.

Dissipator pool length L_s is equal to the larger value of 10h_s or 3W_o where W_o is the culvert width at the culvert outlet. The length of apron L_A is the larger value of 5h_s or W_o. The overall length of the basin L_B is the larger value of 15h_s or 4W_o. The riprap layer thickness at the pool shall be at least 2d_{50} or 1.5d_{100} thick. The riprap layer immediately downstream of the culvert outlet shall have a minimum thickness of 3d_{50}. The basin width at the basin exit (W_B) can be computed using the 1:3 expansion ratio from the culvert outlet to the basin exit, which is $W_o + 2(L_B/3)$. When there are multiple culverts, the estimation of L_s, L_A, and L_B may be based on a single culvert because the equations were developed based on single culvert experiments. However, W_B should be based on the multiple culvert total width. The other basin parameters such as slopes can be found in Figure 8.22 and Figure 8.23. 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.

Step 4. Determine the basin exit depth (y_B) and exit velocity (V_B).

It is assumed that the flow at the basin exit is critical flow. Therefore, basin exit depth (y_B) is critical depth ($y_B = y_c$) and exit velocity ($V_B = V_c$). The critical depth at the basin exit can be determined by solving the critical flow condition equation:

$$Q^2/g = (A_c)^3/T_c \tag{8.18}$$

where

$$A_c = y_c(W_B + zy_c),$$

$$T_c = W_B + 2zy_c,$$

$$V_c = Q / A_c.$$

where:

z = basin side slope, $z:1(H:V)$

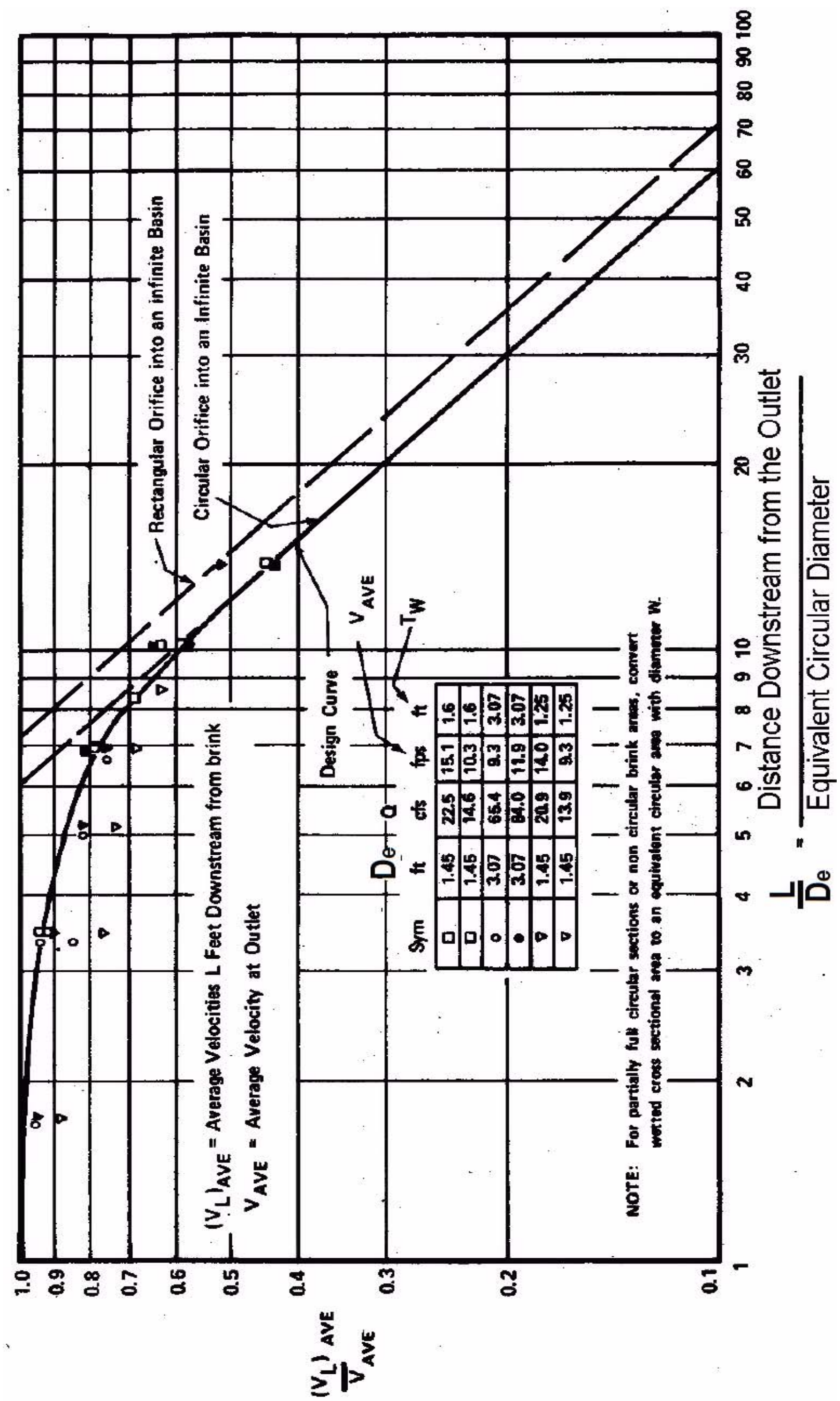
After V_c is computed, it is compared with V_{allow} where V_{allow} is the allowable exit velocity which can be taken as the estimated normal velocity in the tailwater channel or a velocity specified based on stability criteria, whichever is larger. If V_c is less than V_{allow}, then the basin design at above Step 3 is acceptable. If not, the above steps are repeated to evaluate alternative dissipator designs. The normal velocity is computed by Manning's equation. HEC-RAS may be used to estimate channel normal velocity. The stability criteria may be the permissible velocity for the soils downstream of the basin. Permissible velocity standards are set forth in Section 6.5.3.

Step 5. Assess need for additional riprap downstream of the dissipator exit.

If the outlet hydraulics constitute a low tailwater condition defined by $TW / y_o \le 0.75$, no additional riprap is needed. If the outlet hydraulics constitute a high tailwater condition defined by $TW / y_o > 0.75$, the high velocity core of water passes through the basin and diffuses downstream. As a result, the scour hole is longer and narrower. Therefore, if is the outlet protection hydraulics constitute a high tailwater condition, additional riprap is required downstream of the dissipator exit.

The required total distance measured from the dissipator outlet exit can be estimated using Figure 8.26. Figure 8.26 shows a decreasing relationship for L/D_e and $(V_L)_{ave}/V_{ave}$ where L is the distance downstream from the culvert outlet, D_e is the equivalent circular diameter $(4A / \pi)^{0.5}$ for a non-circular culvert, V_{ave} (or V_o) is the average velocity at the culvert outlet and $(V_L)_{ave}$ is the average velocity for the distance L. Various values for L can be tried until $(V_L)_{ave}$ is less than V_{allow}. Once the value of L is found, the additional length of riprap can be found by subtracting L_B from L. The riprap median size for the additional riprap downstream of the riprap basin can be computed by the riprap sizing equation for Downstream of Stilling Basin in Section 6.6.3. The other gradation limits for the riprap can be found in Table 6.4.

FIGURE 8.26
DISTRIBUTION OF CENTERLINE VELOCITY FOR FLOW FROM SUBMERGED OUTLETS
(USDOT, 2006)



Step 6. Determine the toe-down depth for sloping apron or cutoff wall.

The total scour depth should be estimated based on scour estimation procedures in Chapter 11. The toe-down depth for a sloping apron should, as a minimum, be set equal to the total scour depth. The toe-down depth for vertical cutoff walls can be estimated based on the toe protection procedure in Section 6.6.4. This procedure is also sometimes referred to as the “launching of riprap procedure”.

8.4.3 Riprap Downstream of Energy Dissipators

The exit flow conditions at some energy dissipators are at or near critical flow conditions. This flow condition rapidly adjusts to the downstream or natural channel regime; however, critical velocity may be sufficient to cause erosion problems requiring protection adjacent to the energy dissipator. The riprap median size equation for riprap downstream of energy dissipators can be found in Downstream of Stilling Basin in Section 6.6.3. The other gradation limits for the riprap can be found in Table 6.4 in Section 6.6.3.

The length of protection can be judged based on the magnitude of the exit velocity compared with the natural channel velocity. The greater this difference, the longer will be the length required for the exit flow to adjust to the natural channel condition. A filter blanket should also be provided as described in Riprap Layer Characteristics in Section 6.6.3.

8.4.4 Stilling Basins

Stilling basins (see Figure 8.27) are external energy dissipators placed at the outlet of a culvert, chute, or rundown (USDOT, 2006). Stilling basins are some combination of chute blocks, baffle blocks, and sills designed to trigger a hydraulic jump in combination with a required tailwater condition. With the required tailwater, velocity leaving a properly designed stilling basin is equal to the velocity in the receiving channel. There are three types of stilling basins approved for use in Maricopa County; the USBR Type III, the USBR Type IV, and the Saint Anthony Falls (SAF). The ranges for the applicable approaching Froude Number for USBR Type III, USBR Type IV, and SAF are 4.5 - 17, 2.5 - 4.5, and 1.7 - 17, respectively. The design procedures and examples for these stilling basins can be found in (USDOT, 2006). Figure 8.27 illustrates a typical stilling basin. Typical sketches for these three stilling basins can be found in Figure 8.28, Figure 8.29, and Figure 8.30.

FIGURE 8.27
TYPICAL STILLING BASIN
(USDOT, 2006)

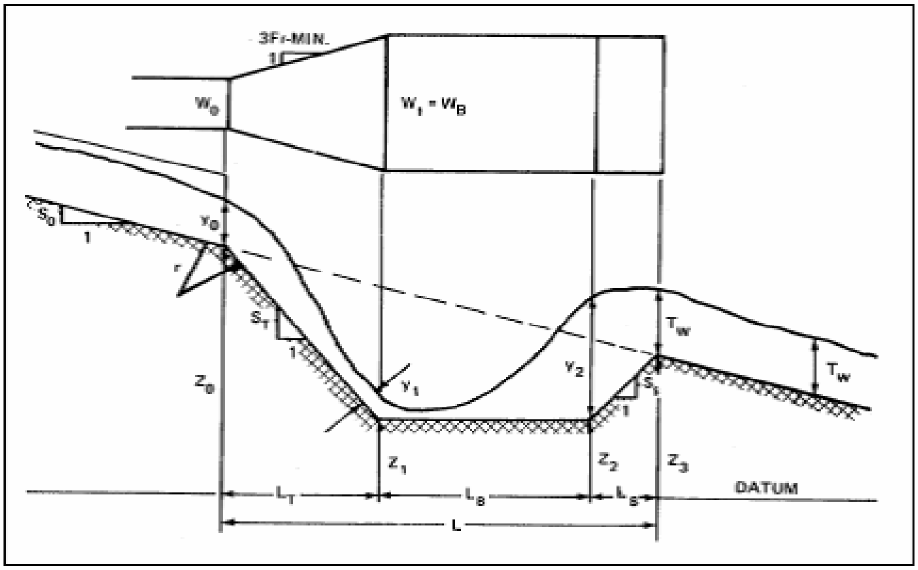


FIGURE 8.28
USBR TYPE III STILLING BASIN
(USDOT, 2006)

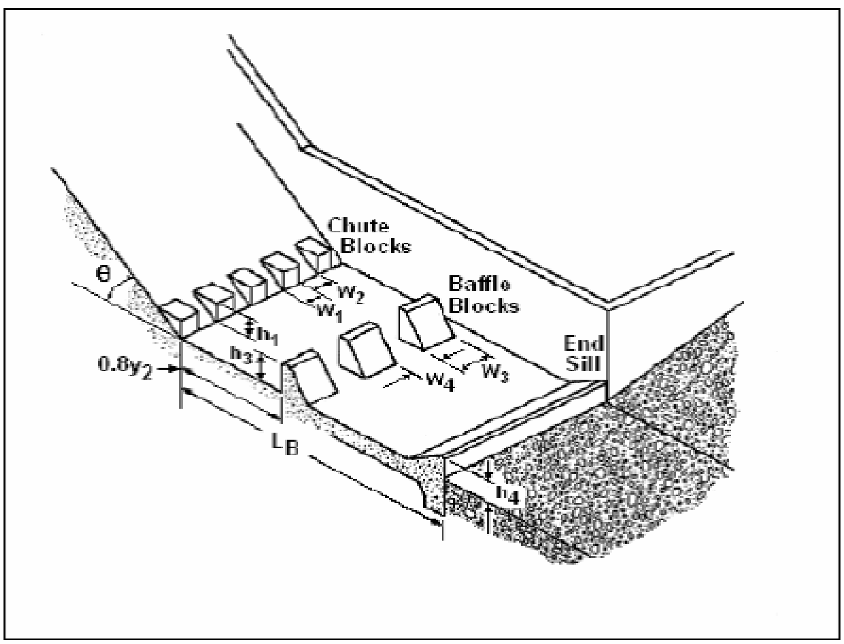


FIGURE 8.29
USBR TYPE IV STILLING BASIN
(USDOT, 2006)

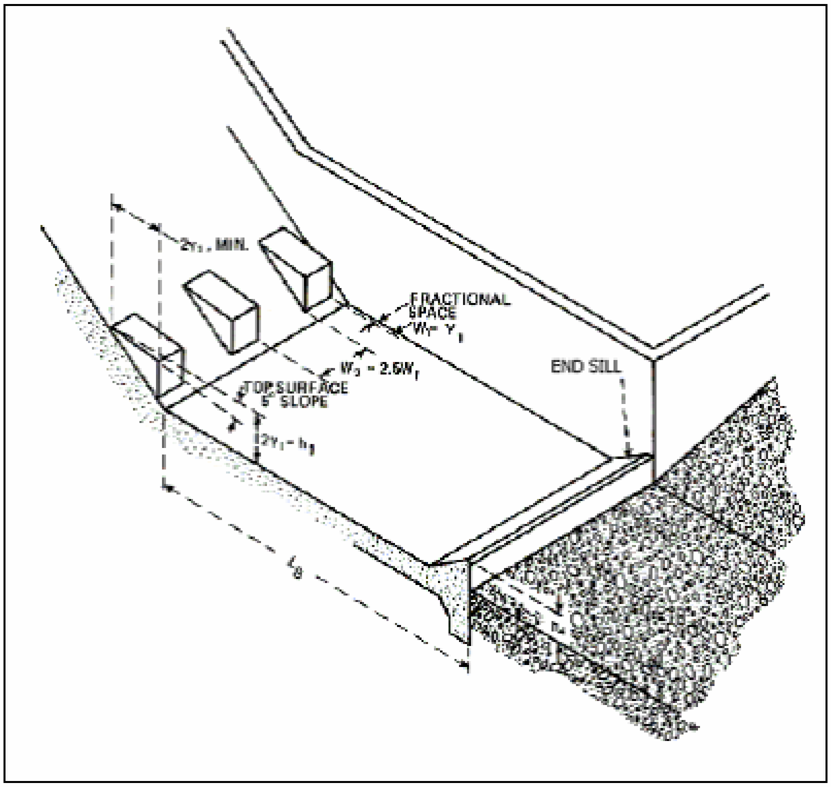
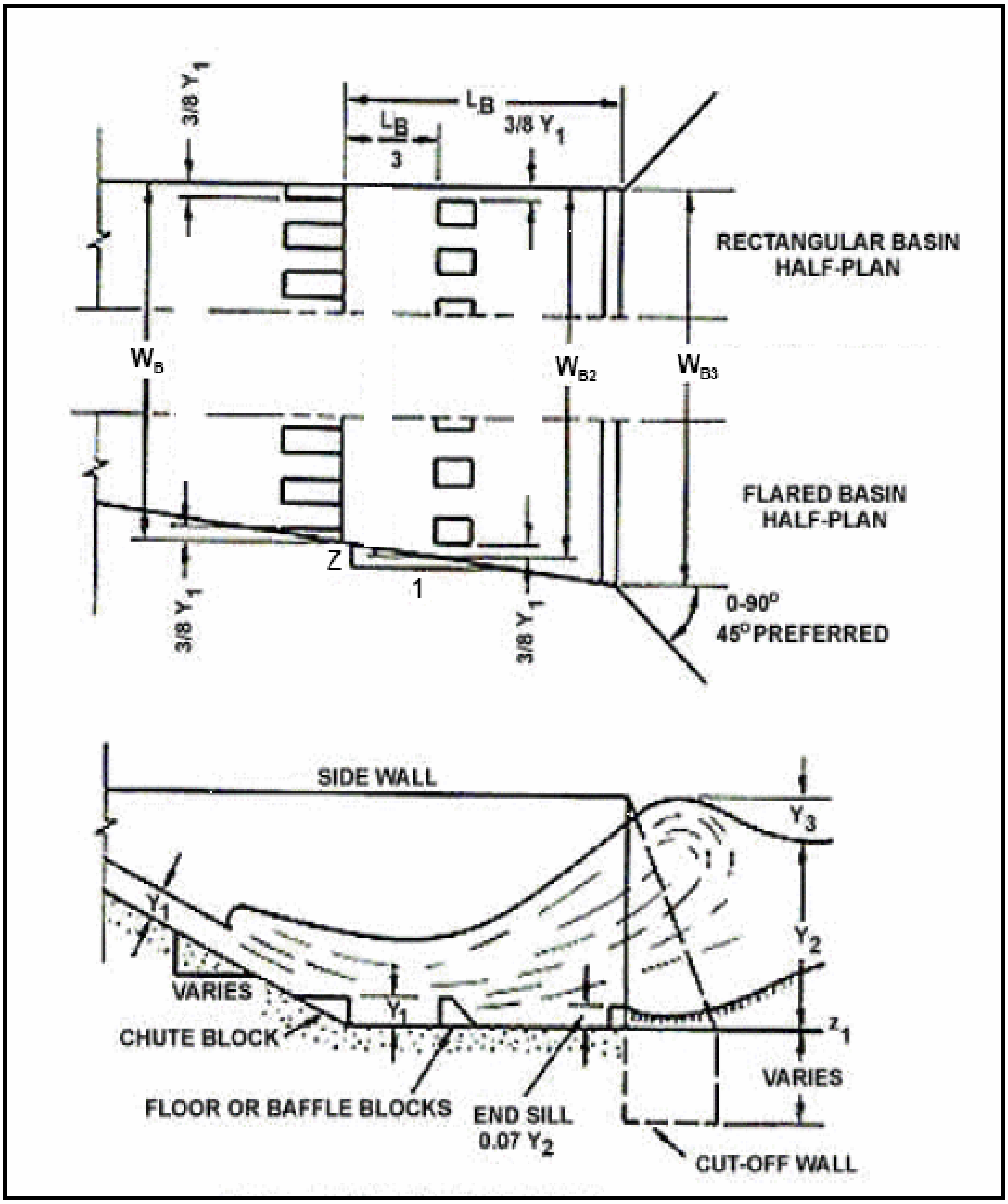


FIGURE 8.30
ST. ANTHONY FALLS (SAF) STILLING BASIN
(USDOT, 2006)



8.4.5 Streambed Level Dissipator Basins

The energy dissipator basins at streambed level for culvert outlets approved for use in Maricopa County are the CSU rigid boundary basin, the Contra Costa basin, the Hook basin, and the

USBR Type VI impact basin (USDOT, 2006). The CSU rigid boundary basin uses staggered rows of roughness elements to initiate a hydraulic jump. The Contra Costa energy dissipator basin is intended for use primarily in urban areas with defined tailwater channels. There are two types of Hook basins. One uses warped wingwalls, another a uniform trapezoidal channel. The Hook basin roughness elements consist of upstream facing hook-shaped dissipators. The USBR Type VI impact basin is contained in a relatively small box-like structure that requires no tailwater for successful performance. This structure can also be used in open channels. The design procedures and examples for these basins can be found in USDOT, (2006). Figure 8.31, Figure 8.32, Figure 8.33, Figure 8.34, and Figure 8.35 illustrate these streambed level energy dissipator basins.

FIGURE 8.31
CSU RIGID BOUNDARY BASIN
(USDOT, 2006)

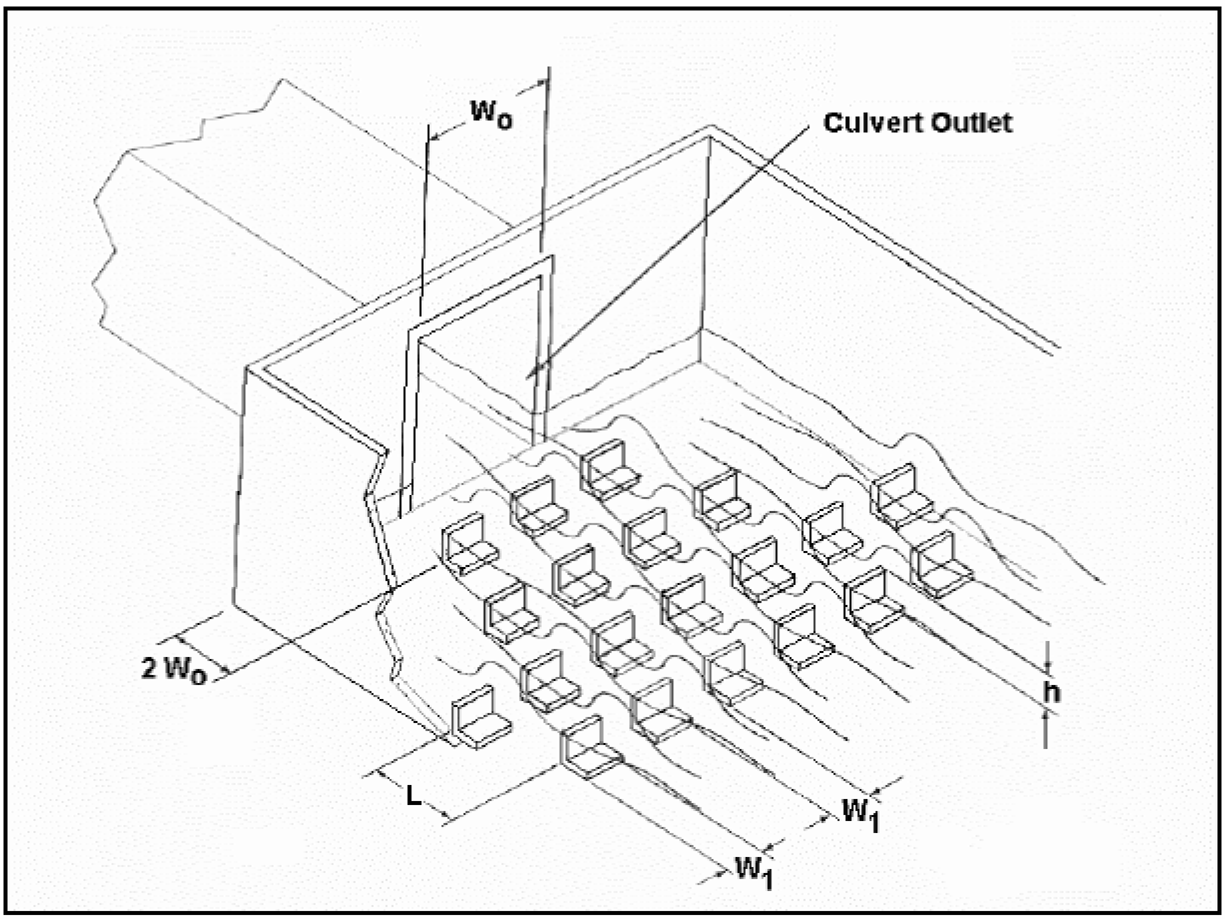


FIGURE 8.32
CONTRA COSTA BASIN
(USDOT, 2006)

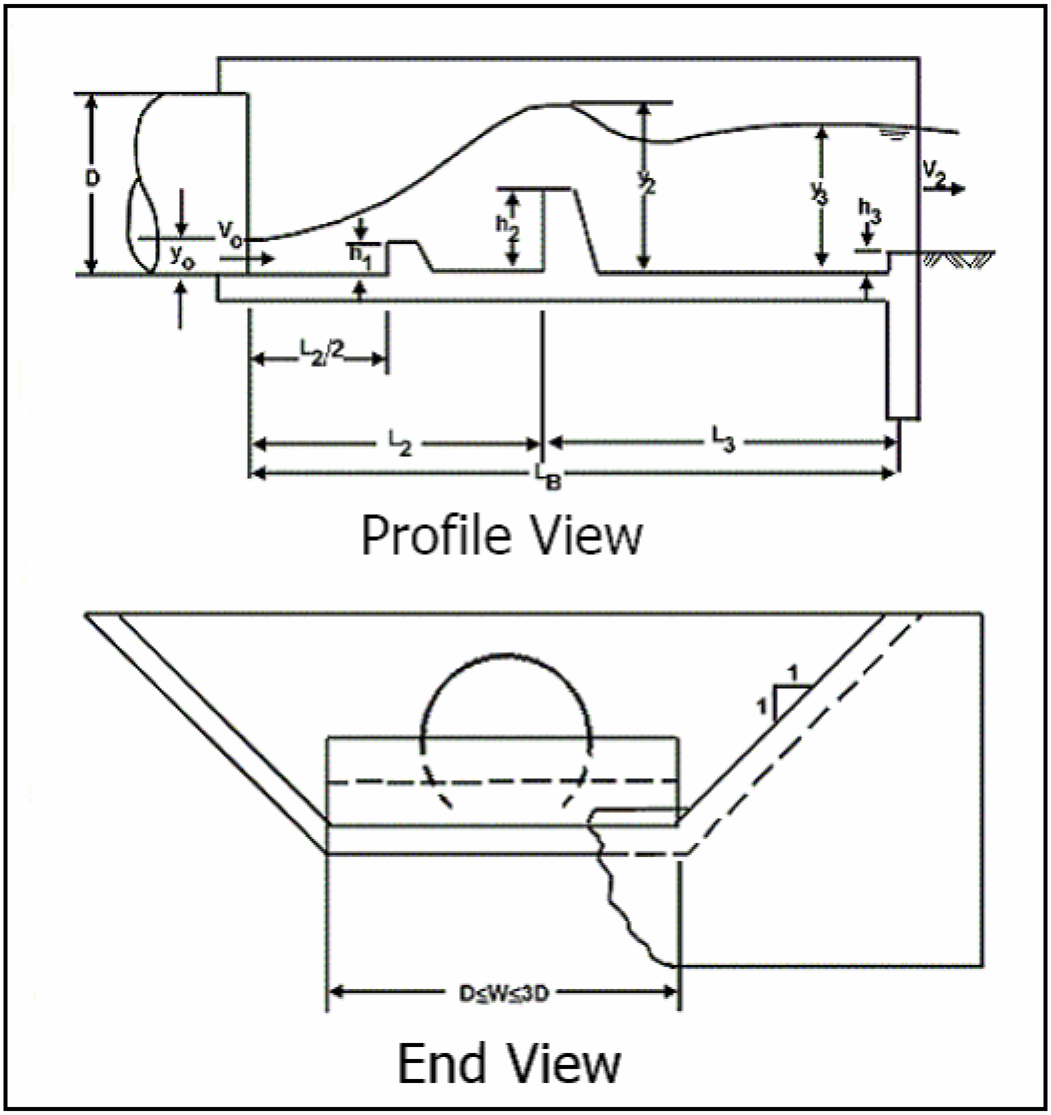


FIGURE 8.33
HOOK BASIN WITH WARPED WINGWALLS
(USDOT, 2006)

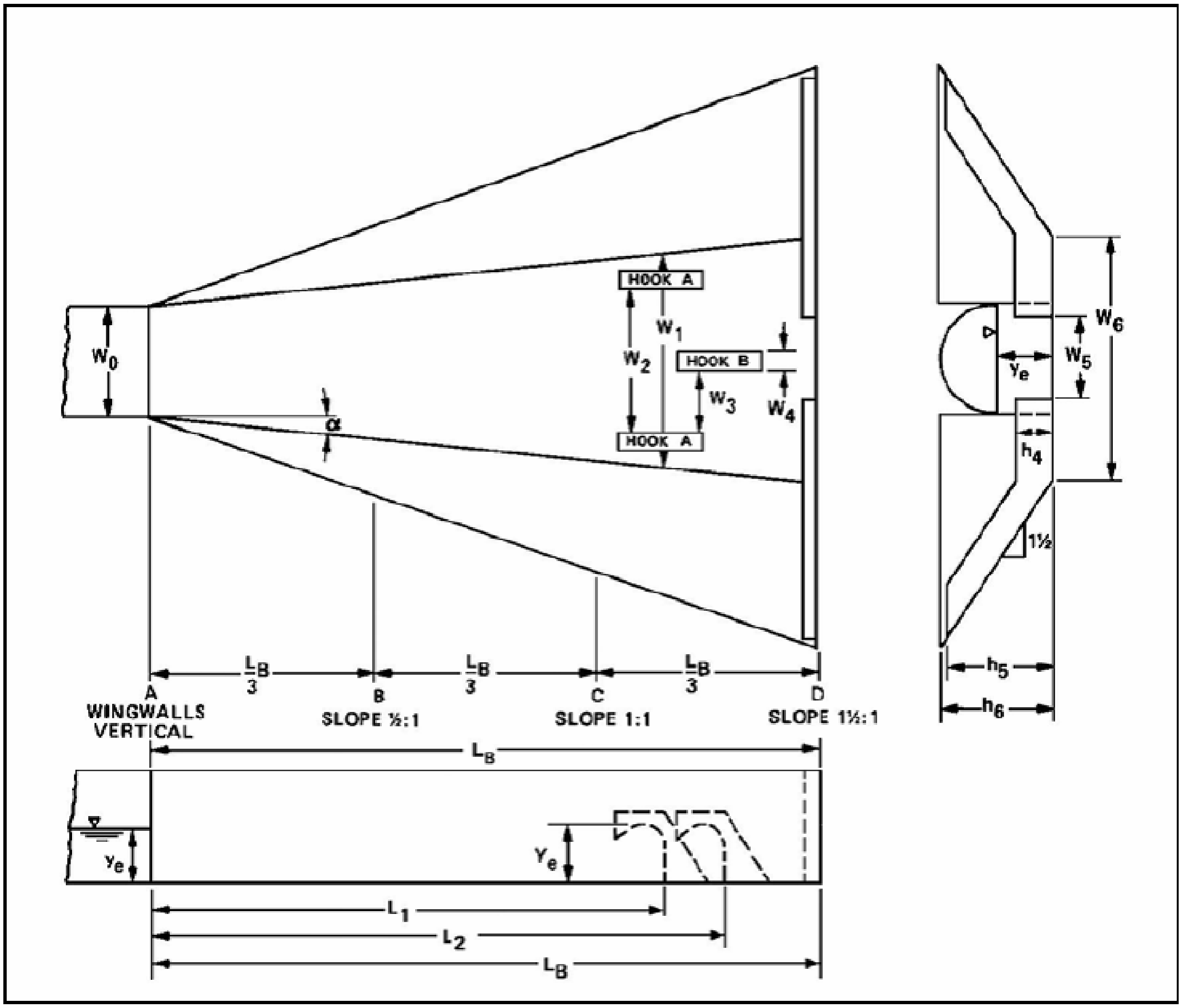


FIGURE 8.34
HOOK BASIN WITH UNIFORM TRAPEZOIDAL CHANNEL
(USDOT, 2006)

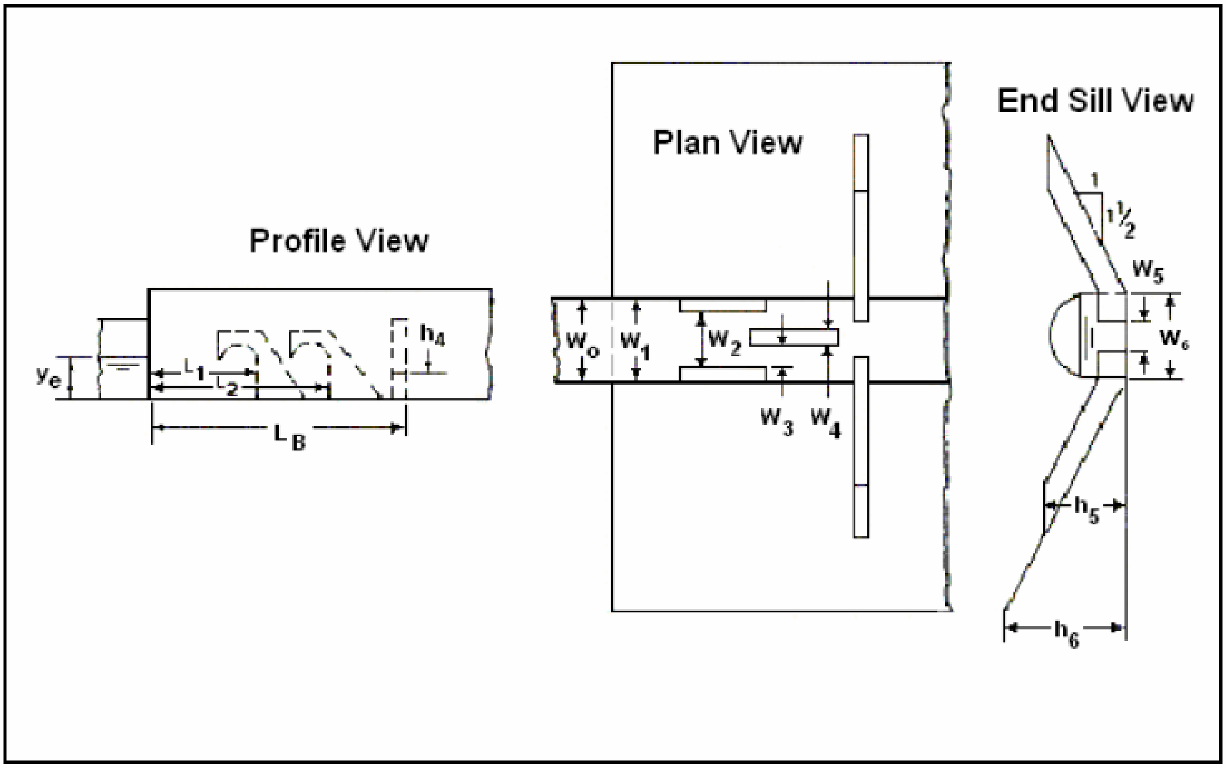
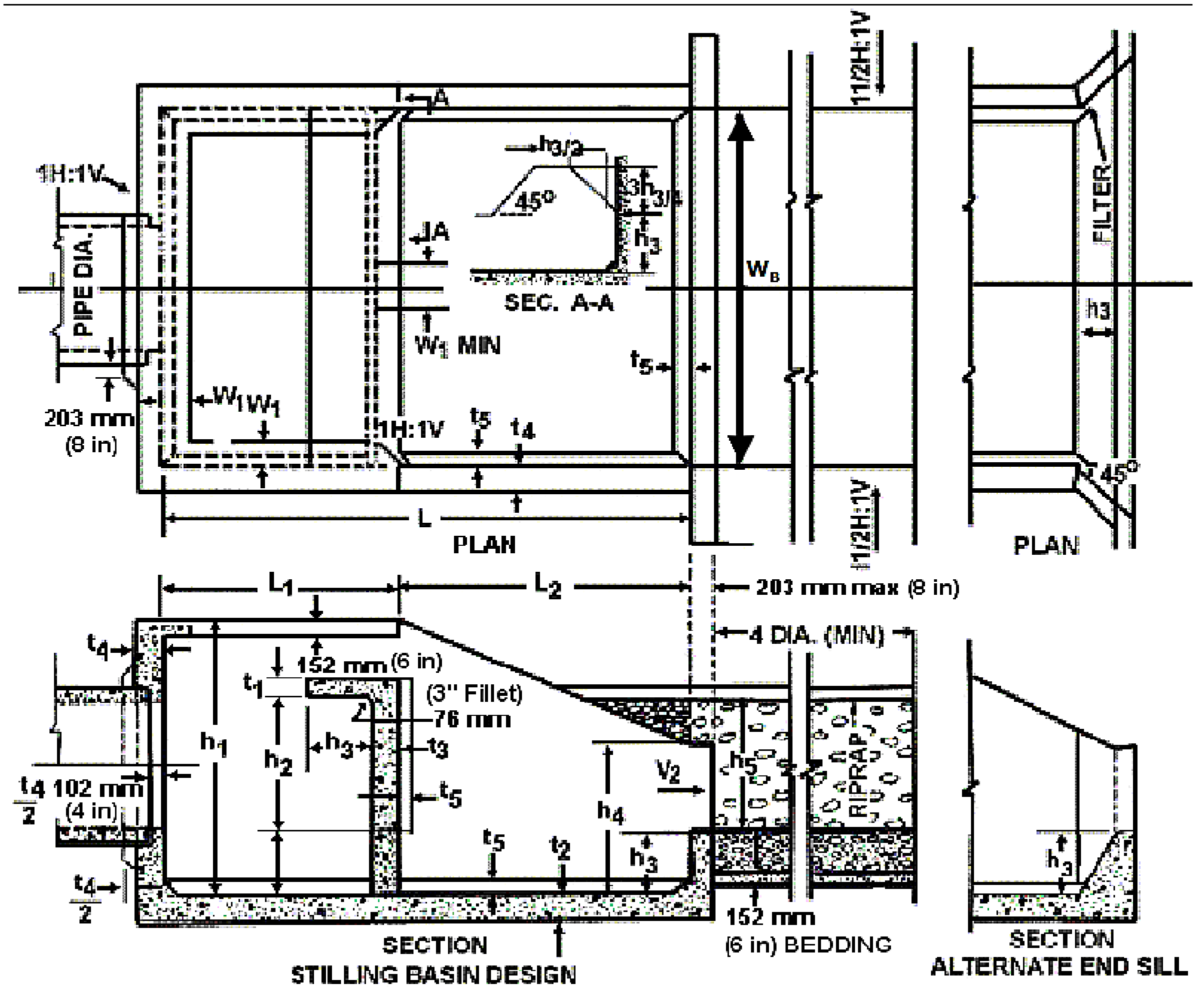


FIGURE 8.35
USBR TYPE VI IMPACT BASIN
(USDOT, 2006)

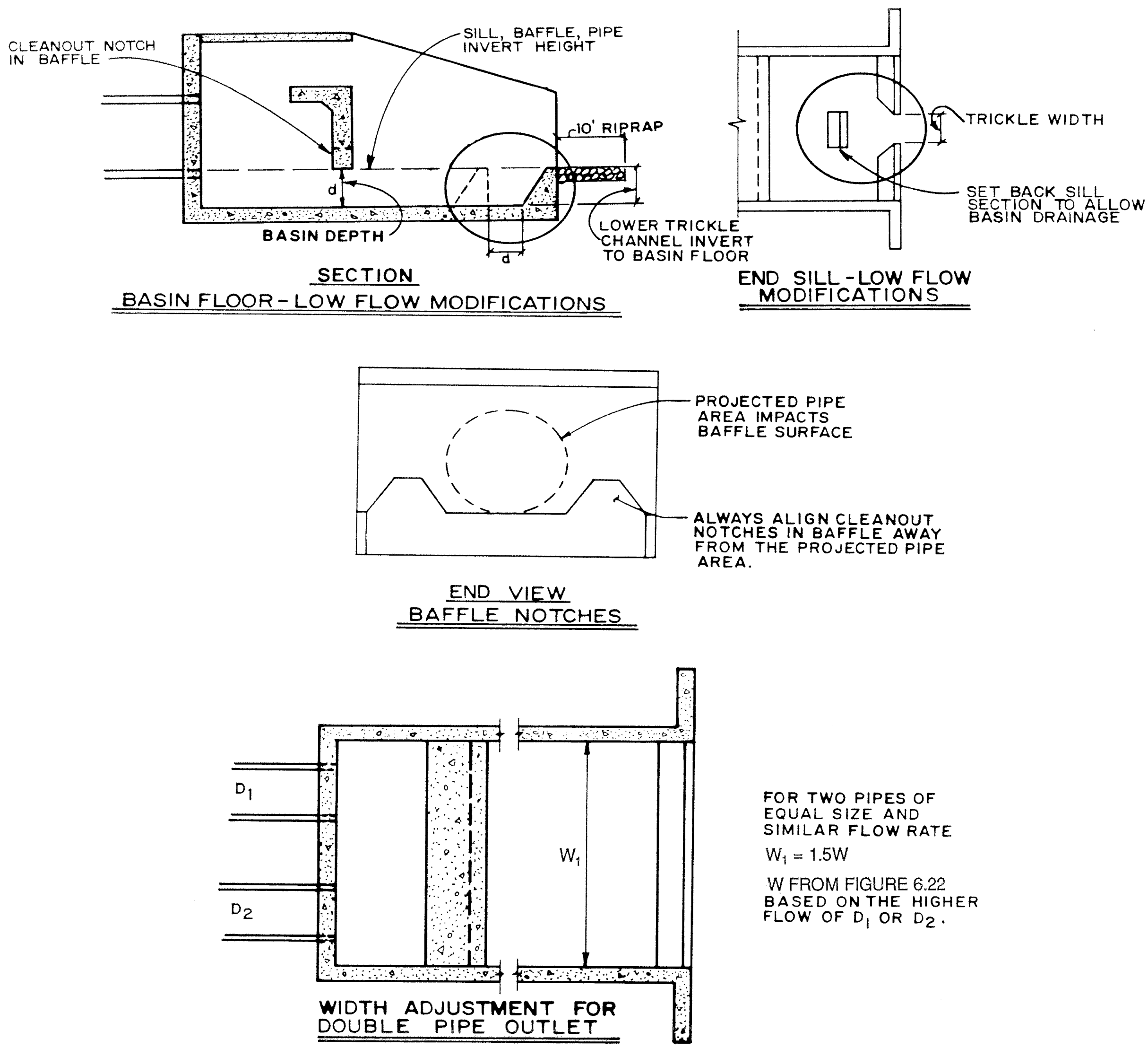


8.4.6 Modifications to USBR TYPE VI Basin

Low Flow Modifications

The standard design will retain a standing pool of water in the basin bottom which is generally undesirable from a safety and maintenance standpoint. This situation should be alleviated where practical by matching the receiving channel low flow depth to the basin depth, see Figure 8.36.

FIGURE 8.36
MODIFICATIONS TO IMPACT STILLING BASIN
(TO ALLOW BASIN DRAINAGE FOR URBAN APPLICATIONS)
(McLaughlin Water Engineers, Ltd., 1986)



A low flow gap is extended through the basin end sill wall. The gap in the sill should be as narrow as possible to minimize effects on the sill hydraulics. This implies that a narrow and deeper (1.5 to 2-foot) low flow channel will work better than a wider gap section. The low flow width should not exceed 60 percent of the pipe diameter to prevent the jet from short-circuiting through the cleanout notches.

Low flow modifications have not been fully tested to date. Caution is advised to avoid compromising the overall hydraulic performance of the structure. Other ideas are possible including locating the low flow gap at one side (off center) to prevent a high velocity jet from flowing from the pipe straight down the low flow channel.

The optimal configuration results in continuous drainage of the basin area and helps to reduce the amount of sediment entrapment.

1. For large basins where the sill height is greater than 2.0 feet, the depth dimension, d, (in Figure 8.36) may be reduced to avoid a secondary drop from the sill to the main channel. The low flow invert thereby matches the floor invert at the basin end and the main channel elevation is equal to the sill. Dimension d should not be reduced by more than one-third and not less than 2 feet. This implies that a deeper low flow channel (1.5 to 2.0 feet) will be advantageous for these installations.

   Note that dimension d is also reduced at the minimum pipe invert height and at the bottom of the baffle wall.

2. A sill section should be constructed directly in front of the low flow notch to break up bottom flow velocities. The length of this sill section should overlap the width of the low flow by about 1 foot. The general layout for the low flow modifications is shown in Figure 8.36.

<u>Multiple Conduit Installations</u>

Where more than one conduit of different sizes has outlets in close proximity, a composite structure can be constructed to take advantage of common walls. This can be somewhat awkward since each basin "cell" must be designed as an individual basin with different dimensions. Where two conduits of the same size have close outlets, the structures may be combined into a single basin as shown in Figure 8.36.

The total width of a combined dual inlet basin can be reduced to three-fourths of the total width for separate basins. For example, if the design width for each pipe is W, the combined basin width would be $1.5W$.

The effect of mixing and turbulence of the combined flows in the basin has not been model tested to date. It is suggested that no wall be constructed to separate flow behind the baffle, thereby allowing greater turbulence in the combined basin.

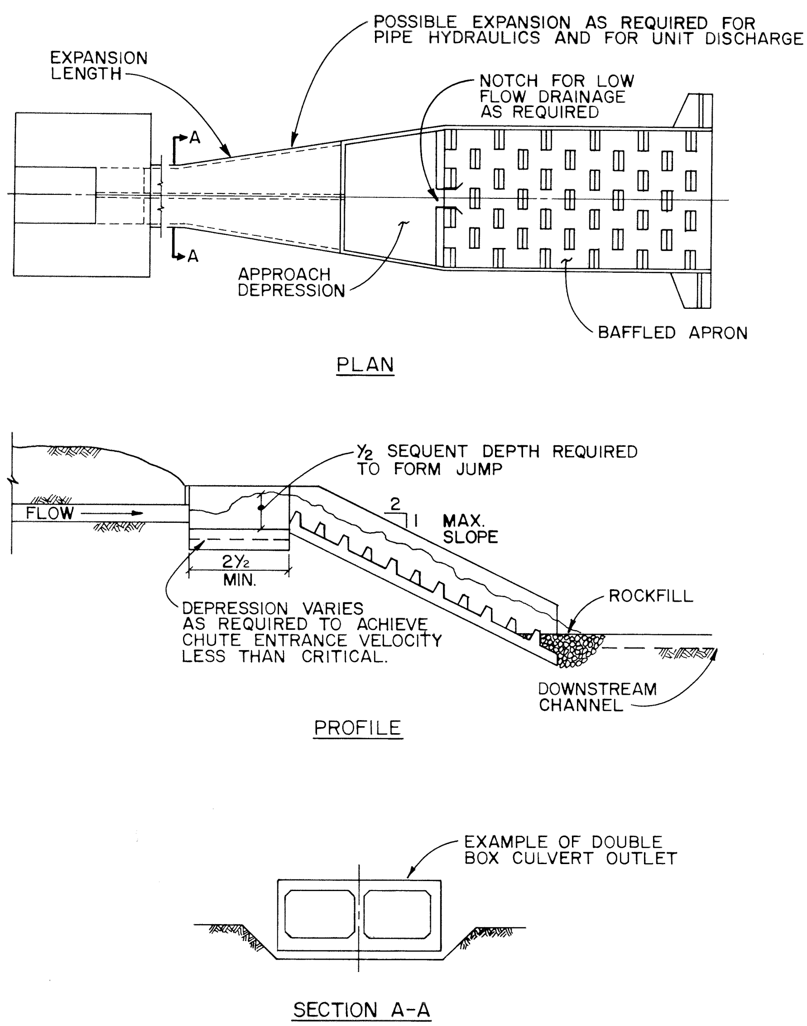
Remaining structure dimensions are based on the design width of a separate basin W. If the two pipes have different flows, the combined structure should be based on the higher Froude number flows.

8.4.7 Baffle Chute Energy Dissipator

The baffle chute developed by Peterka (1984) has also been adapted to use at pipe outlets. This structure is particularly well suited to situations with very large conduit outfalls and at outfalls to channels in which some future degradation is anticipated. As mentioned previously, the apron can be extended at a later time to account for channel subsidence. Generally, this type of structure is only cost effective if a grade drop is necessary below the outfall elevation and a hydraulic backwater can be tolerated in the culvert design.

Figure 8.37 illustrates a general configuration for baffled outlet for a double box culvert outlet. In this case, an expansion zone occurs just upstream of the approach depression. The depression depth is designed as required to achieve the flow velocity at the chute entrance as described in Section 8.3.4. The remaining hydraulic design is the same as for a standard baffle chute. The same crest modifications are applicable to allow drainage of the approach depression, to reduce the upstream backwater effects of the baffles, and to reduce the problems of debris accumulation at the upstream row of baffles.

FIGURE 8.37
BAFFLE CHUTE AT CONDUIT OUTLET
(ADAPTED FROM: Peterka, 1984)



An effective means of controlling velocities within the culvert is the use of reinforced concrete pipe (RCP) velocity control rings. The culvert velocity reduction by internal energy dissipators (velocity control rings or roughness elements) force the hydraulic jump to occur within the culvert, thus eliminating costly outlet structures. The design procedures can be found in *Concrete Pipe Handbook* (ACPA, 1988) and *HEC-14* (USDOT, 2006).

8.4.8 Use of the HY-8 Software for Energy Dissipator Design

The HY-8 software program is a national standard for culvert design. HY-8 version 7.2 includes energy dissipator options for riprap basins, stilling basins, and streambed level dissipators design. The riprap basin design menu is located under the streambed level dissipators menu inside the software. HY-8 is a public domain package, the latest version of which can be downloaded from http://www.fhwa.dot.gov/engineering/hydraulics/software/hy8/. Although the software can be used for design, it is highly recommended to verify the results with hand-calculations based on the procedures and examples presented in HEC-14 (USDOT, 2006) and Peterka (1984).

8.5 SPILLWAYS

Hydraulic Analysis of Spillways

Spillways can take a variety of forms. Some of those, such as morning glory and fuse-plug, are beyond the scope of this manual. In application of the more complex spillways, the appropriate hydraulic analyses must be performed by an experienced hydraulic engineer with due consideration of all aspects of the flow hydraulics. The most common spillways for use in typical drainage structures are of the weir or orifice type. In those cases, the weir equation (Equation (8.19)) and the orifice equation (Equation (8.20)) are the commonly used analytic methods.

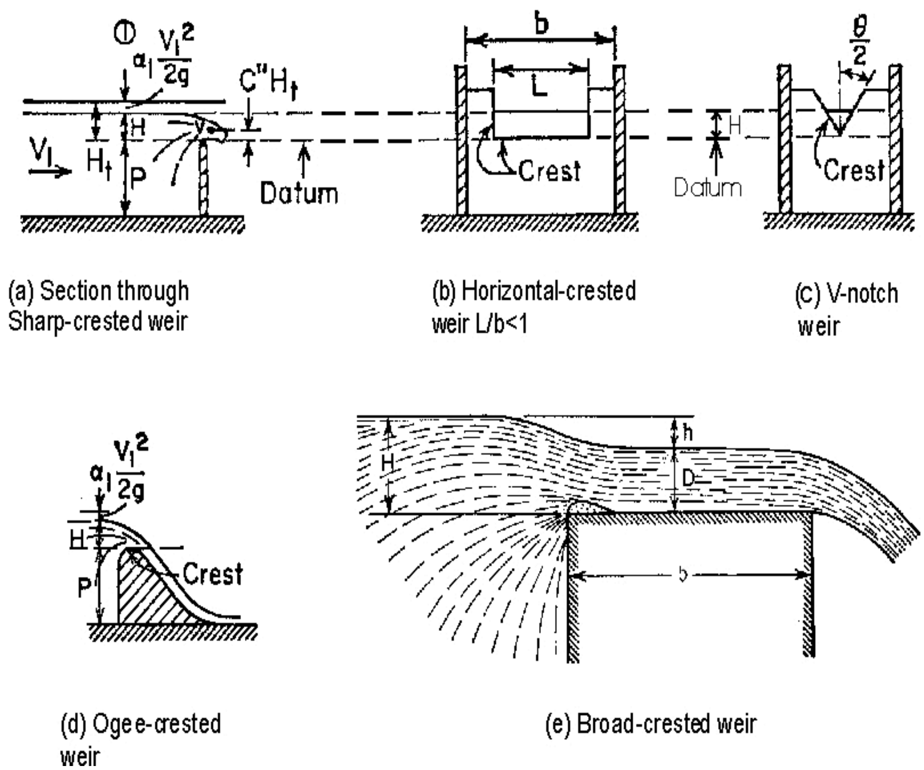
$$Q = CLH^{3/2} \tag{8.19}$$

$$Q = C_oA(2gH)^{1/2} \tag{8.20}$$

Weir-Type Spillways

Weir-type spillways can be generally classified as sharp crested, broad crested or compound curve (ogee) shaped (Figure 8.38). The primary difference between sharp crested and broad crested weirs is the thickness of the weir (in profile) relative to the depth of water passing the crest. Where the crest thickness is greater than 6/10 the depth of flow over the weir, the weir can be considered to be broad crested (Simon, 1981). In all cases, the weir equation is generally used to assess spillway performance and to establish a spillway capacity rating curve. However, the selection of the weir coefficient, C, is a function of numerous factors including the total head on the weir, the vertical height of the weir, inclined faces of the weir (both upstream and/or downstream), submergence conditions, and breadth of broad crested weirs. Care must be taken in selecting the value of C and in applying appropriate correction factors to C depending upon the structure configuration and flow conditions.

FIGURE 8.38
WEIR SPILLWAY CONFIGURATIONS
(ADAPTED FROM: Brater and King, 1976)



For sharp crested weirs, the weir coefficient can range from about 3.2 to an excess of 5.0. The Rehbock equation (Equation (8.21)) (Chow, 1959, page 362) is often used to estimate C:

$$C = 3.27 + 0.40H/h_w \tag{8.21}$$

where H is the measured head and h_w is the height of the weir. That equation is valid for H/h_w up to 5 but can be extended to H/h_w = 10 with fair approximation. Values of C in excess of 5.0 should not be used without careful deliberation of all factors including consequence of overestimated capacity. Typical C values are in the lower end of the aforementioned range. It is important to note that this discussion assumes that the nappe of water over the sharp crested weir is fully aerated. Insufficient aeration will result in undesirable performance, including pressure differential on the structure, unsteady and pulsing discharge over the weir, and increase in spillway discharge. Brater and King (1976) provides useful tables in selecting appropriate values for C.

Broad crested weirs have widely varying physical conditions which significantly affects the value of the weir coefficient. The normal range of C is from about 2.4 to about 3.5, however, use of values in excess of 3.1 must be carefully analyzed and are generally not recommended. A discharge coefficient of 3.0 is typical for flow over roadway embankments without backwater

(Bureau of Public Roads, 1978). See Section 5.3.3 for adjustment to C for roadway embankments subjected to submergence. The head, H, is measured at least 2.5H upstream of the weir for broadcrested weirs.

Ogee shaped spillways can offer the best hydraulic performance, however, the cost of such spillways is usually greater than other comparable weir types. Ogee spillways must be designed and analyzed by appropriate methods, such as those enumerated in the *Design of Small Dams* (USBR, 1987, p353 and 366-367).

It is important to note that downstream water surface elevation (tailwater) must be analyzed by appropriate methods (see Chapter 6) to assess potential for submergence of any weir.

Conduit Type Spillways

Impoundments that incorporate pipe or conduit in the principal outlet can be assessed as a culvert as detailed in Chapter 5. Under inlet control, the orifice equation provides a relation between ponded depth and outlet discharge. The orifice equation is useful in preparing rating curves for detention basins where one or more openings are incorporated into the riser of the primary outlet structure.

Principal spillway conduits other than those that can be analyzed by culvert hydraulics (see Chapter 5) can usually be analyzed under conditions of inlet and outlet control by procedures contained in hydraulic references such as *Design of Small Dams* (USBR, 1987, pages 453-470) or Brater and King (1976). It is important to note that such structures must be analyzed for both inlet and outlet control with appropriate consideration of tailwater conditions that may exist at the outlet of structure.

Stepped Spillway

Stepped spillways consist of a series of steps on a slope. Because energy is dissipated by flow over each step, stepped spillways produce cost savings in the size of the energy dissipator (US Bureau of Reclamation, 2006). There are two flow regimes for stepped spillways, which are nappe flow and skimming flow as shown in Figure 8.39 and Figure 8.40. For nappe flow, the water plunges from one step to another. Nappe flow is associated with low discharges. As the discharge increases, the nappe flow will transition to skimming flow. For skimming flow, the water flows down the steps as a coherent stream skimming over the steps. A comprehensive review of stepped spillways can be found in US Bureau of Reclamation (2006). Other references on stepped spillways are Chanson (1994a, 1994b, and 2001), Gonzales and Chanson (2007), and Boes and Hager (2003a, 2003b, and 2005).

FIGURE 8.39
NAPPE FLOW ON STEPPED SPILLWAY
(US Bureau of Reclamation, 2006)

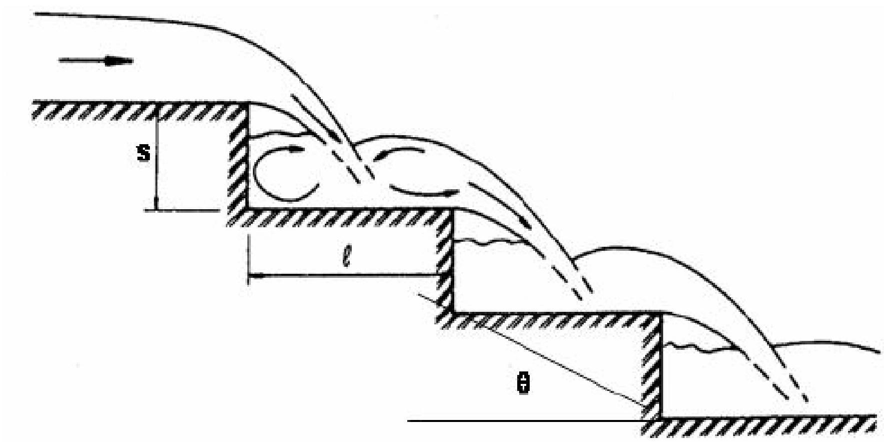


Figure 1. - Schematic of nappe flow over steps showing the flow impingement on the step tread and air pocket in the step offset.

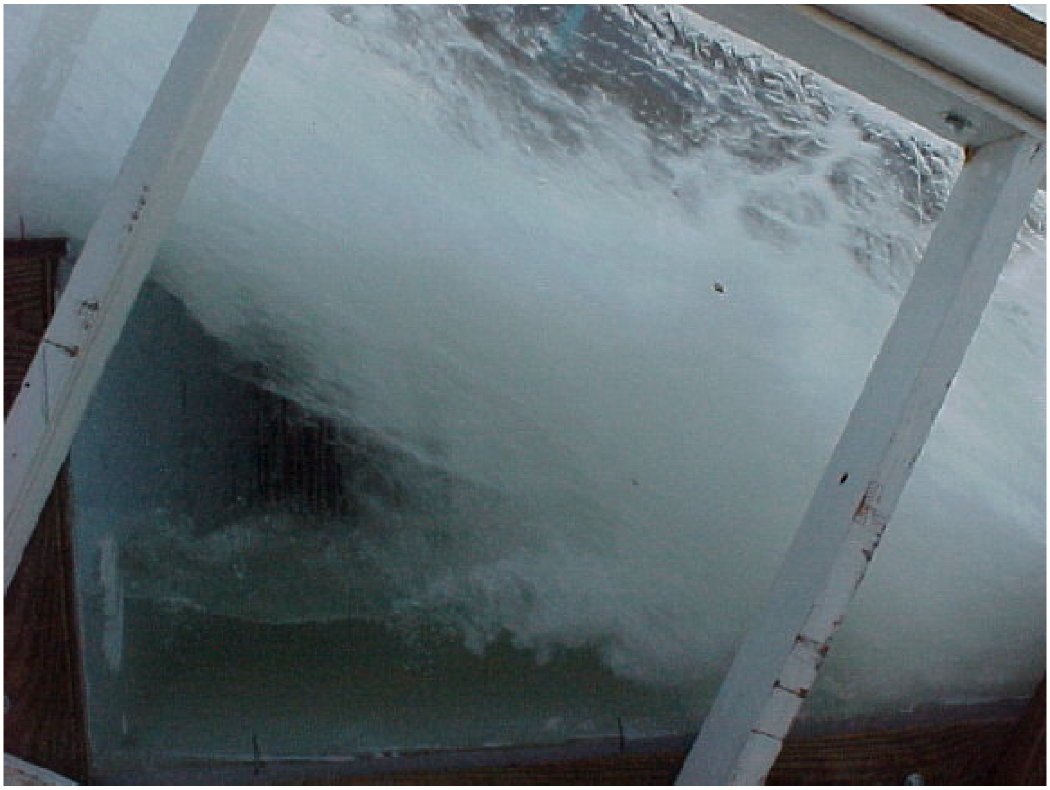


Figure 2. - Flow over a prototype 2-ft-high step showing nappe flow for q= 15 ft^3/s/ft.

FIGURE 8.40
SKIMMING FLOW ON STEPPED SPILLWAY
(US Bureau of Reclamation, 2006)

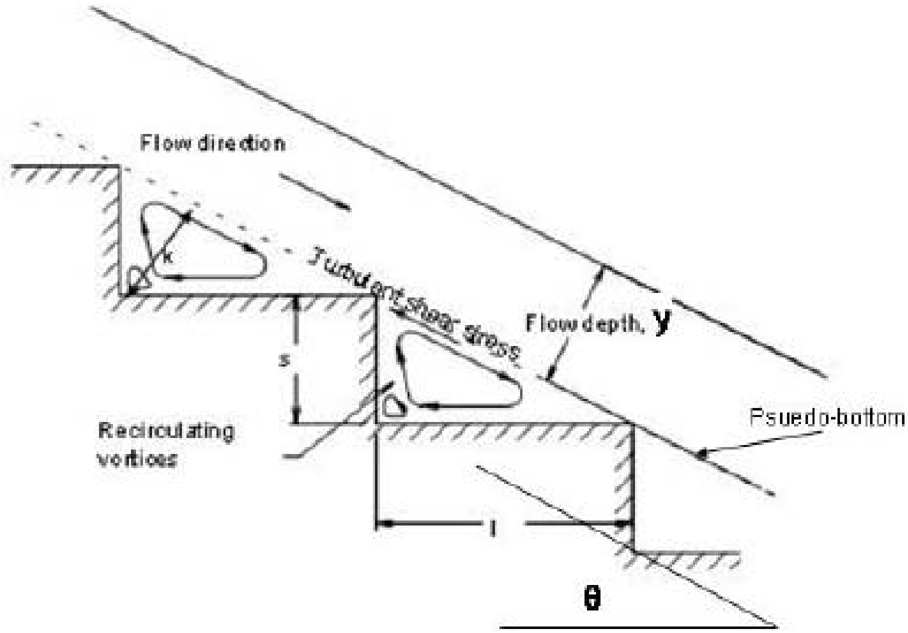


Figure 4. - Schematic of the skimming flow regime over steps.

Figure 5. - Skimming flow over 1-ft-high steps at the CSU flume for a 15 ft^3/s/ft unit discharge at step 40 down from the crest near the bottom of the flume.

Compound Rating Curves

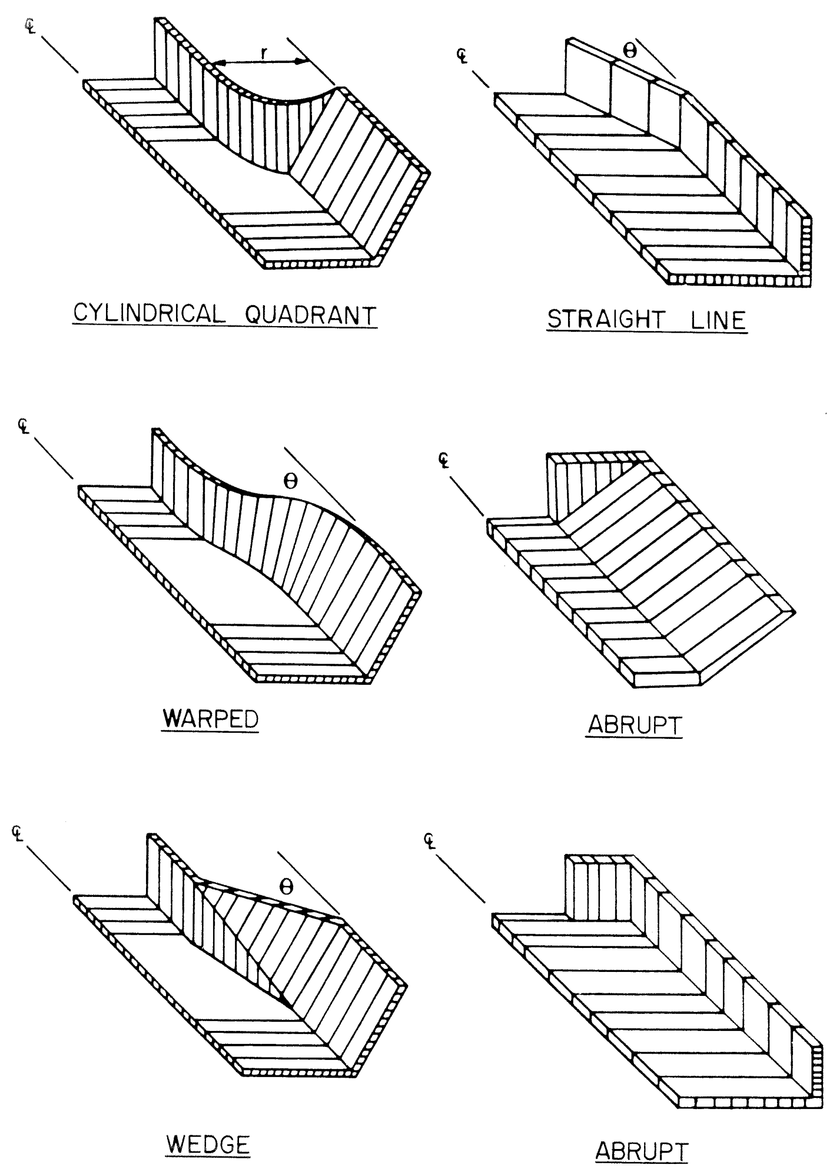
When an impoundment incorporates more than one spillway, a compound rating curve is developed for use in storage routing. Coupled with stage-storage data, an inflow hydrograph can be routed through a basin, thereby estimating ponded water surface elevation and outflow discharge (See Chapter 9). The principal and emergency spillways are individually assessed for discharge over a range of impoundment water levels, starting at the lowest anticipated level to above the height of the dam. The discharge from each spillway at each elevation is totaled to develop the compound rating curve. For stormwater detention facilities, it is usual to prepare compound rating curves for the principal spillway as these structures may have low, middle, and high level inlets to meter outflow from the basin. The controlling hydraulic conditions must be considered when developing a rating curve for an outlet structure. For example, consider a principal spillway represented by a pipe culvert with a grated drop inlet. The weir equation is used to develop a discharge rating based upon the length and width of the drop inlet. The orifice equation is used to develop a discharge rating based upon the grate opening. For these two ratings, the lesser discharge for a given elevation is the governing discharge for the outlet rating curve. In this example, the outlet pipe capacity would also be assessed to verify that it does not control outlet hydraulics.

8.6 SPECIAL CHANNEL STRUCTURES

8.6.1 Channel Transitions

A flow transition is a change of the open channel flow cross section designed to be accomplished in a short distance with a minimum amount of flow disturbance. Types of transitions are illustrated in Figure 8.41. Of these, the abrupt (headwall) and the straight line (wingwall) are the most common.

FIGURE 8.41
CHANNEL TRANSITION TYPES
(ADAPTED FROM: USDOT, FHWA, HEC-14, 1983)



Contractions

Specially designed open channel flow transitions (contractions) are normally not required for highway culverts. A culvert is normally designed to operate with an upstream headwater pool which dissipates the channel approach velocity and, therefore, negates the need for an approach flow transition. The side and slope tapered inlets for culverts are also designed primarily as submerged transitions and are discussed in Chapter 5.

Special inlet transitions are useful when the conservation of flow energy is essential because of allowable headwater consideration, such as an irrigation structure in subcritical flow, or where it is desirable to maintain a small cross section with supercritical flow in a steep channel. Furthermore, special transitions should be considered at locations where channel geometry changes, bridges, chutes, and other structures.

Expansions

Outlet transitions (expansions), changes in Q, right-of-way, channel geometry, bridges, chutes and other structures must be considered in the design of all culverts, channel, protection, and energy dissipators. Design considerations for subcritical channel transitions are presented in *Hydraulic Design of Energy Dissipators for Culverts and Channels* (USDOT, 1983)

Bifurcation Structures

It may occasionally be necessary to divert part of the flow in a channel. For example, the designer may need to divert a portion of the flow to a stormwater basin or, the downstream right-of-way may be too narrow to accommodate the full flow and a portion of the flow may have to be diverted to another outfall point. In these instances the designer will have to provide a “splitter” or bifurcation structure to apportion the flow in the appropriate direction.

In order for the structure to work as designed, the water surface elevation must be the same in all three channels at the proposed structure. This is accomplished by determining the water surface elevation in the upstream channel at the proposed structure. Then, the exact location of the splitterwall to divert the desired amount of water is calculated. Last, the geometry of both downstream channels must be adjusted to produce water surface elevations at the structure that match the water surface elevation in the upstream channel.

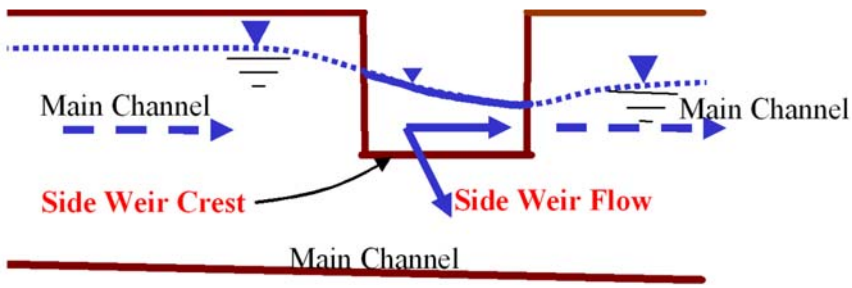
If the flow in the channel at the structure site is supercritical, the process is reversed and the water surface profiles are calculated in the downstream direction. However, considerable caution should be exercised in attempting to split supercritical flows. Readers are strongly encouraged to consult appropriate references listed at the end of this chapter or seek the advise of an experienced professional.

Once the water surface at the structure site has been established, the amount of flow in each area of the upstream channel can be calculated and the precise horizontal location of the splitter wall established. The initial angle of departure of the diverted channel should not exceed 12 degrees. This will minimize the formation of standing waves and turbulence that could encroach on the channel freeboard or otherwise reduce the capacity of the channel.

Side Channel Weirs

Side weirs, also known as lateral spillways, are used as key structures in many flood control projects. Side weirs are usually installed along the side of the main channel to divert water into another hydraulic structure when the flow surface in the main channel rises above the side weir crest. Figure 8.42 shows a side view of a channel with a side weir.

FIGURE 8.42
SIDE VIEW OF A CHANNEL WITH A SIDE WEIR



Hager's Weir Discharge Coefficient Equation. Hager's equation deals with three types of side weirs: sharp-crested weir, broad-crested weir, and round-crested weir (Hager, 1987). Figure 8.43, Figure 8.44 and Figure 8.45 show the side view of these three types of weirs. In Figure 8.43, Figure 8.44 and Figure 8.45 water in the main channel flows perpendicular to the figure view.

FIGURE 8.43
SHARP-CRESTED WEIR
(Hager, 1987)

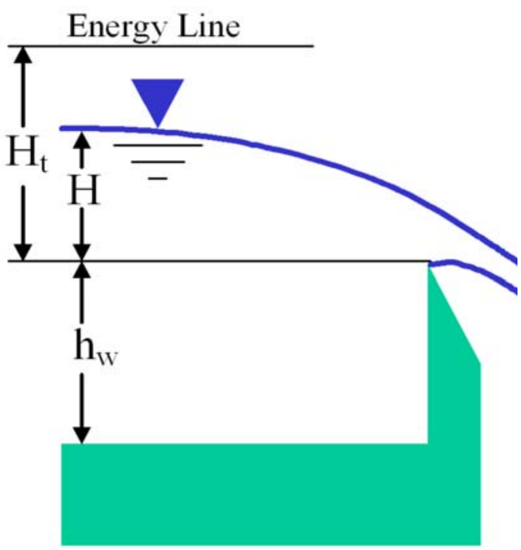


FIGURE 8.44
BROAD-CRESTED WEIR
(Hager, 1987)

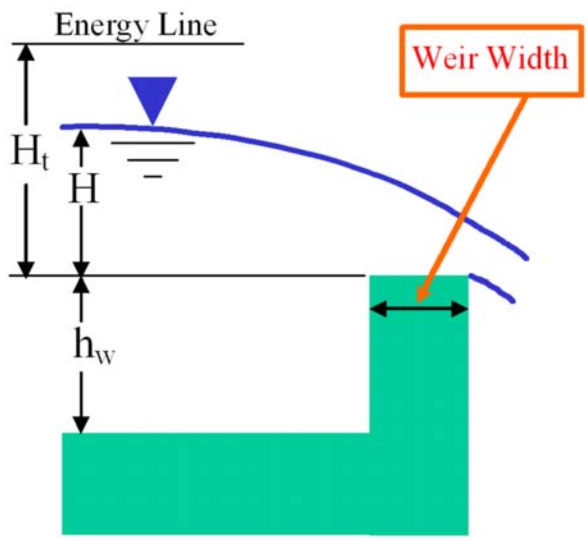
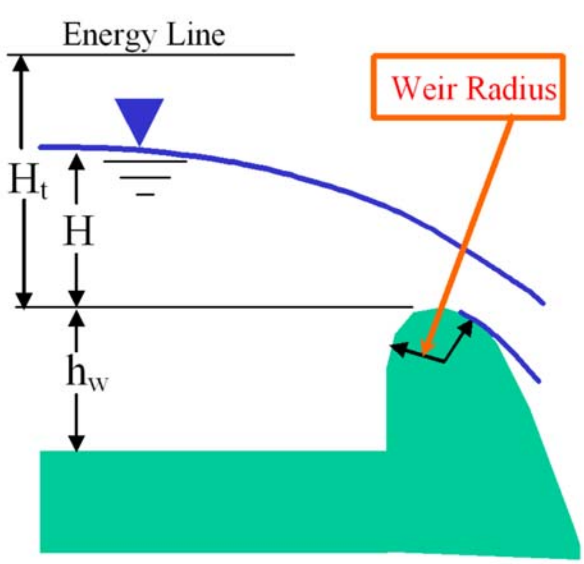


FIGURE 8.45
ROUND-CRESTED WEIR
(Hager, 1987)



The flow equation for the side weir can be expressed in the conventional weir equation as follows:

$$Q_w = CLH^{1.5} \tag{8.22}$$

where:

L = Side weir length (along the main channel flow direction);

H = Head measured from the top of the weir crest (excluding velocity); and

$$C = \frac{3}{5} m C_h \sqrt{g} \left[\frac{1 - W_h}{3 - 2y_h - W_h}\right]^{0.5} \left\{1 - (\theta + S_0)\left[\frac{3(1 - y_h)}{y_h - W_h}\right]^{0.5}\right\} \tag{8.23}$$

where:

m = Number of side weirs (1 or 2);

$$W_h = \frac{h_w}{H_t + h_w} \tag{8.24}$$

$$y_h = \frac{H + h_w}{H_t + h_w} \tag{8.25}$$

where:

H_t = Total head measured from the top of the weir crest, ft;

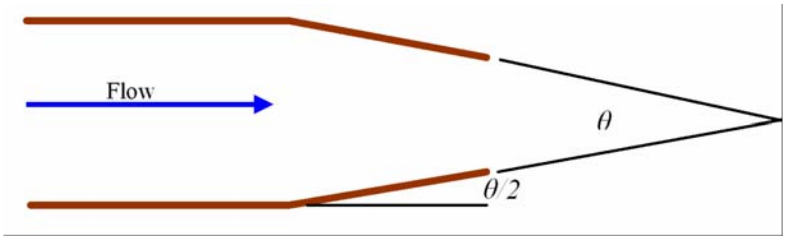
h_w = Weir height, ft;

g = Acceleration due to gravity, 32.2 ft/sec^2;

S_0 = Main channel bed slope, ft/ft;

θ = Main channel contraction angle in radians (see Figure 8.46).

FIGURE 8.46
MAIN CHANNEL CONTRACTION



For a sharp-crested weir: $C_h = 1$

For zero weir height: $C_h = 8/7$

For a broad-crested weir (b is weir width per Figure 8.44):

$$C_h = 1 - \frac{2}{9\left[1 + \left(\frac{H_t}{b}\right)^4\right]} \quad (8.26)$$

For a round-crested weir (r_w is weir radius per Figure 8.45):

$$C_h = \frac{\sqrt{3}}{2}\left[1 + \frac{\frac{22}{81}\left(\frac{H_t}{r_w}\right)^2}{1 + \frac{1}{2}\left(\frac{H_t}{r_w}\right)^2}\right] \quad (8.27)$$

HEC-RAS and Hager's Equation.

As part of cooperation between FCDMC and HEC, Hager's side weir equation (Hager, 1987) has been incorporated into HEC-RAS version 4.1.0 and later for both steady state and unsteady state flows. It should be noted that if an off-line detention basin is to be designed, an unsteady state HEC-RAS model should be used. More discussion can be found in the HEC-RAS Hydraulics Reference Manual (USACE, 2008).

Channel Junctions

Special design considerations are needed for channel junctions as follows:

- The design water surface elevations immediately upstream of the confluence should be equal.

- The angle of junction intersection should be less than 12 degrees (zero is preferred). The centerline radius of any channel can not be less than 3 times the top-width at the water surface.
- The design depth of the main channel below the junction should be the same (or virtually so) as the main channel upstream of the confluence.
- For supercritical flow regime a momentum analysis as outlined in the Corps of Engineers document *EM 1110-2-1601* (USACE, 1994) must be undertaken. On a case by case basis, model testing will be required.
- Channels designed with Froude numbers between 0.9 and 1.13 will not be allowed.

8.6.2 Supercritical Flow Structures

Acceleration Chutes

Acceleration chutes, whether leading into box culverts, pipes, or high velocity open channels, are often used to reduce downstream cross sections, hence, reducing costs. Chute spillways may be used in connection with both off-stream and on-stream stormwater storage reservoirs for a control structure and/or a spillway.

Acceleration chutes are potentially hazardous if inadequately planned and designed (see USBR, 1974; Peterka, 1984; and SCS, 1976). High velocity flow can wash out channels and structures downstream in short order, resulting in property damage and uncontrolled flow. The references cited previously, address acceleration chutes in greater detail than can be discussed in this manual. Refer to these publications for a detailed analysis.

Chutes have four component parts:

1. Inlet
2. Vertical Curve Section
3. Concrete, Steeply Sloped Channel
4. Outlet

Several types of inlets can be incorporated depending on the physical conditions and the type of control desired, particularly when using chute spillways for off-stream stormwater storage facilities. The types of inlets to be considered are:

- Straight Inlet
- Box Inlet
- Side-Channel Inlet

- Culvert Inlet
- Drop Inlet

Normally, the flow must remain at supercritical through the length of the chute and into the channel or conduit downstream. Care must be exercised in the design to insure against an unwanted hydraulic jump in the downstream channel or conduit. The analysis must include computation of the energy gradient through the chute and in the downstream channel or conduit.

Bends

Structures are generally unnecessary in subcritical flow channels unless the bend is of small radius. Structures for supercritical flows are complex and require careful hydraulic design to control the flow.

Bends are normally not used in supercritical flow channels because of the costs involved and the hazards introduced. It is possible to utilize banking, easement curves, and diagonal sills (Knapp, 1951). Sometimes outside bank rollover structures might even be considered. All of these, however, are generally out of place in urban drainage works. Additional design guidelines for open channel bends may be found in *Hydraulic Design of Flood Control Channels* (USACE, 1991).

When a bend is necessary, and it is not practical to first take the flow into subcritical flow, the designer will generally conclude that the channel should be placed in the closed conduit for the entire reach of the bend, and downstream far enough to eliminate the main oscillations. A model test is usually required on such structures. Furthermore, the forces exerted on the structure are large and must be analyzed.

The forces involved with hydraulic structures are large, and their analyses are often complex. The forces created can cause substantial damage if provisions are not made for their control. In bends, forces are usually larger than what is intuitively assumed. The momentum equation permits solution for the force acting upon the flow boundary at a bend.

$$F_b = M\Delta V \qquad (8.28)$$

where ΔV represents the change in direction and/or magnitude of the velocity through the section bend. The force due to pressure on the bend should also be calculated when conduits flow under pressure.

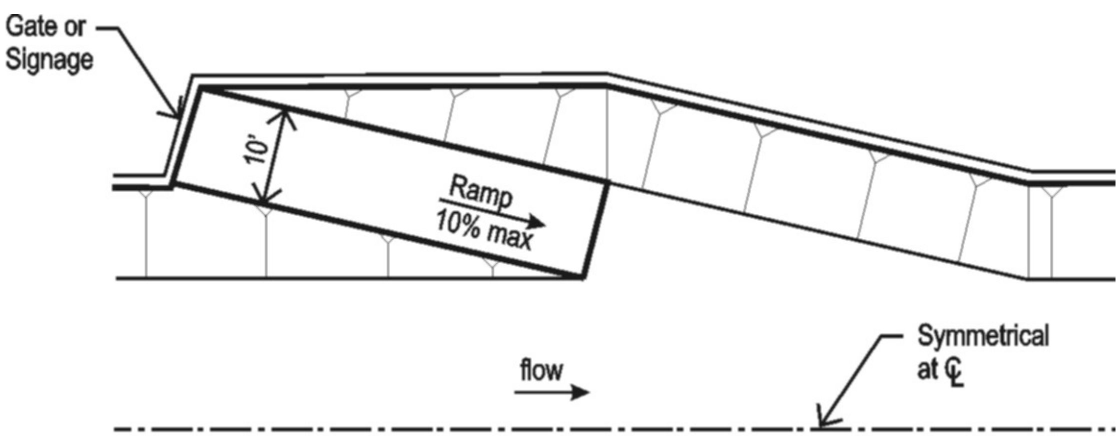
$$\Delta P = \frac{P}{2}(\Delta V^2) \qquad (8.29)$$

where ΔP represents the pressure change caused by the difference in the squares of the velocities through the bend. The total exerted force on the bend by the water, the total of momentum and pressure forces, must be counteracted by external forces. Allowable soil bearing should be determined using soil tests if necessary. Forces which cannot be handled by conduit bearing on the soil must be compensated for by additional thrust blocks or other structures.

8.6.3 Access Ramps

Vehicular access to drainage and flood control channels must be provided at periodic intervals to permit the efficient removal of sediment and accumulated debris and to facilitate structural maintenance. Access is typically provided by 10-foot wide ramps constructed in the channel sideslopes. Figure 8.47 illustrates a typical ramp design and a typical flared sideslope design.

FIGURE 8.47
TRAVERSE SLOPE CHANNEL ACCESS RIGHT ANGLE CHANNEL ACCESS



The City of Albuquerque, New Mexico investigated the hydraulic effects of vehicle ramps and flared side slopes in channels (Heggen, 1991).

Although the study is too long to be included in this manual, the final recommendations are consistent with other recommendations in this manual and can be summarized as follows:

1. For subcritical flow, the hydraulic consequences of occasional access structures are minor. For supercritical flow, the hydraulic consequences of channel cross sectional changes can be major. Hydraulic jumps or oblique waves can jeopardize the entire channel.

2. Ramps should be directed downstream.

3. The Froude number approaching downstream ramps should not exceed 2.2 for a one-sided configuration.

4. Flared sideslopes should be as steep as vehicle access allows.

5. The Froude number approaching 3:1 flared sideslopes should not exceed 3.5 for a one-sided configuration.

6. The Froude number approaching a 6:1 flared sideslopes should not exceed 2.2 for a one-sided configuration.

7. Structures should be symmetrical.

8. Upstream and downstream channel slopes are not a significant factor in performance.

9. Ramps perform somewhat better than 6:1 flared sideslopes, but not as well as 3:1 flared sideslopes.

As a general rule, access structures should be provided at the upstream and downstream side of every culvert and street crossing. Access over or around drop structures also needs to be considered.

8.6.4 Trashracks and Access Barriers

The necessity for trashracks depends on the size of the conduit, the nature of the trash and debris, public safety and other factors. These factors will determine the type of trashracks and the size of the openings. A smaller conduit will require closely spaced trash bars and a larger conduit requires more widely spaced trash bars. If there is no danger of clogging or damage from small trash, a trashrack may consist simply of struts and beams placed to exclude only the larger trees and such floating debris. For trashracks with approach velocities less than 3 ft/sec, it is not necessary to include a head loss for the trashrack; however, for velocities greater than 3 ft/sec, such computations are required.

Trashracks can promote debris buildup and the subsequent reduction of hydraulic performance. Thorough analysis of this potential should be undertaken prior to their use. Depending on the anticipated volume and size of the debris an open area between the bars of 1.5 to 3.0 times the area of the culvert entrance should be provided.

Trashrack losses are a function of velocity, bar thickness, bar spacing, rack angle, and orientation of the flow entering the rack, the latter condition being an important factor. Trashracks with bars oriented horizontally are not permitted, and horizontal bars used to support vertically oriented bars should be as small as practical and kept to the minimum required to meet structural requirements.

The expected head loss from a trashrack in a channel is greatly affected by the approach angle. The head loss computed by Equation (8.30) should be multiplied by the appropriate value from Table 8.7, when the approach channel and trashrack are at an angle to each other. Equation (8.30) applies to access barriers placed on conduit outlets and should be used when approach velocities are greater than 3 ft/sec. The approach angle loss factor does not apply when the outlet works trashrack is within a detention basin, reservoir, dam or other ponded area.

$$H_g = 1.5\frac{[V_g^2 - V_a^2]}{2g} \tag{8.30}$$

TABLE 8.7
LOSS FACTORS FOR APPROACH ANGLE SKEWED TO TRASHRACK
DERIVED FROM Metcalf and Eddy (1972)

Approach Angle (degrees)	Loss Factor
0	1.0
20	1.7
40	3.0
60	6.0

For trashracks in detention basins, reservoirs, dams or areas where the flow into the outlet conduit is ponded, the headloss shall be computed by Equation (8.31) (Metcalf and Eddy, 1972):

$$H_g = K_t\frac{V_n^2}{2g} \tag{8.31}$$

where K_t is given by Equation (8.32):

$$K_t = 1.45 - 0.45\frac{a_n}{a_g} - \left(\frac{a_n}{a_g}\right)^2 \tag{8.32}$$

A plugging factor of 50 percent shall be used for all trashrack analysis. For maximum headloss, 1/2 of the net area between the bars shall be considered blocked. This will result in twice the velocity through the trashrack. For detention basin and dam outlet works analysis, trashrack headloss shall be calculated for the plugged condition as well as the unplugged condition.

The trashrack/access barrier assembly shall be hinged or removable to allow access to the outlet construction. The screen shall be fabricated of a minimum of 1/2 inch x 2 inch flat steel bars or larger designed to withstand the hydrostatic load resulting from the 100-year design ponding with screen openings blocked. Attachment points shall be cast in the headwall concrete and anchored by substantial anchor bolts. Shear pins shall be in 1/8 inch, 3/16 inch or 1/4 inch rods depending on the size of the barrier involved. The largest size possible shall be utilized The rack assembly shall be galvanized steel or steel with a protective coating suitable for exposure to sunlight, as well as submerged conditions. An anti-vortex device should be included with the trashrack design if vortices are anticipated which could affect hydraulic efficiency and cause erosion of adjacent earth slopes.

8.6.5 Groins and Guide Dikes

There are several flow control structures that are similar in configuration and serve to reduce erosion/scour. Some of these also serve to train flow away from critical areas. Because of the similarity in form or function, the terminology used in practice tends to be overlapping in that the term used by one entity or organization conflicts in meaning with the same term used by another. In this section, two hydraulic structures will be discussed. The first, identified herein as groins, are used to train flow and reduce erosion in channels. The second, referred as guide dikes, serves a similar purpose, but are typically found in a natural floodplain setting.

Groins

Structures located along and protruding from the banks of a channel for purposes of training flow away from the bank, reducing velocities, or reducing erosion are termed groins herein. Other terms used for structures meeting this definition are spurs, hardpoints, and dikes. In a natural setting, these structures are often deployed at the outside of bends in a channel to reduce bank erosion and redirect higher velocities towards the center of the channel where higher velocities are better tolerated due to armoring. In the absence of armoring, these structures merely relocate the area subject to continued erosion (see Chapter 11 for further discussions on sedimentation). Hydraulically, groins create greater depths of flow upstream of the structure in subcritical flow conditions and flatten the energy grade line. Acting like a constriction, the energy grade line is steeper at the structure while backwater eddies are created immediately downstream of the structure unless they are drowned out by overtopping flow. These structures tend to be designed to train low to moderate flows without overtopping. Higher flood flows usually overtop the structure. Under certain circumstances, groins deployed on both sides of an engineered channel can be used to flatten the energy grade line, thereby allowing a steeper channel slope. Under all applications, the appropriate hydraulic analysis should be employed to evaluate velocities under the range of conditions expected or required to meet regulatory requirements. Erosion protection is often required at the groin and downstream of the groin.

Groins may be made of many different materials including riprap, gabions, piling (wood and steel), rock and earth filled cribs. Depending upon the entity responsible for maintenance, the designer should verify acceptable materials for the application at hand.

Guide Dikes

These structures are deployed upstream of bridge abutments and serve to transition flow into the bridge from the floodplain. Also called guide banks, these structures have been found to minimize scour of the abutments and piers. Here, the scour is relocated to the head of the guide dike, thereby offering hydraulic efficiency and scour protection to the bridge structure. Design procedures for guide banks are enumerated in *Bridge Scour and Stream Instability Countermeasures* (USDOT, 2001).

8.7 SAFETY

Hydraulic structures constructed in Maricopa County will usually be subject to public access. Designs for hydraulic structures must address the issue of safety. First, signage must be provided to identify the potential hazard of flooding or dangerous flow measures to the public. Second, appropriate measures must be designed to keep the public away from hazardous locations. For example, vertical drop structures should not exceed 2.5 feet in height with 6-foot horizontal aprons, and adequate fencing or railings must be provided along all other walls, such as wing walls or training walls.

Additional considerations for safety are discussed in the introduction to this manual (Chapter 1).

8.8 OPERATION, MAINTENANCE & AESTHETIC CONSIDERATIONS

8.8.1 Operation and Maintenance

Hydraulic structures should be designed so they can be maintained. As with other drainage facilities, maintenance operations will consist of scheduled and unscheduled operations. Scheduled operations include mowing, debris removal, graffiti removal, and rock replacement. Unscheduled operations are those which follow a storm event and include debris removal, rock replacement, erosion repair, fence or railing repair and other activities for which the frequency and scope cannot be predicted. Some maintenance considerations appropriate for hydraulic structures are presented below. Access to key areas (i.e. crest area, stilling basin area) for maintenance equipment and personnel is the primary consideration common to all structure types.

Slopes of 4:1 or flatter are recommended for mowing equipment on landscaped or grass bank and transition slopes. The local jurisdictional agency should be consulted regarding special circumstances for specific site constraints where a steeper slope may be necessary.

Transition areas upstream and downstream of the structures should be designed to drain completely. This applies particularly to stilling basins.

Selection and placement of rock for a stilling basin or upstream of a drop crest should consider a size range not easily displaced by flow as well as one not easily moved by vandalism. Grouted boulders are a suitable alternative.

Open channels are recommended in lieu of pipes for conveyance of low flows through the drop structure area. Pipes may plug or frequently overtop, leading to additional maintenance problems. Riprap should be provided at likely scour areas that are relatively expensive to access and repair later.

8.8.2 Structure Aesthetics

General

Aesthetics, safety, recreation, and overall integration with nearby land uses are important aspects in the design of hydraulic structures. The design and planning, construction, and maintenance of hydraulic structures and natural drainageways in an urban setting all offer opportunities for promoting aesthetic design and habitat features. Maximizing functional uses while improving visual quality requires good planning from the onset of the project, and the coordinated efforts of the owner/client, engineer, landscape architect, and planner. The significance of providing an aesthetic and visually appealing project depends on the number, type, and frequency of viewer; the viewing angle; project location; and the overall environment of the project area. Aesthetic considerations are site and project specific.

The combination and diversity of forms, lines, colors, and textures create the visual experience.

Material selection and landscape design can provide visual character and create interesting spaces in and around hydraulic structures.

Open Spaces and Parks

Creative planning concepts in urban and urbanizing areas, particularly in residential areas, emphasize multiple uses of flood control, recreation, and open spaces. Cluster housing and good subdivision planning may be coordinated to offer opportunities to maintain the natural habitat characteristics of the drainageway while fulfilling open space and recreation requirements.

Multiple use of flood control structures and open space parks has proven to be an effective and aesthetic land use combination. Athletic fields and stormwater storage areas which remain dry most of the time have been used in many communities. The design of overflow structures and crest controls can be combined with concrete pathways to blend with a park lined environment.

Materials

A variety of materials and finishes are available for use in hydraulic structures. Concrete color additions, exposed aggregates and form liners can be used to create visual interest to otherwise stark walls. The location of expansion and control joints in combination with reveals can be used to create effective design detailing of headwalls and abutments. Rock and vegetation can be used for bank stability and erosion protection around structures to provide visual contrast and diversity, and spatial character.

8.9 REFERENCES

8.9.1 Cited in Text

American Concrete Pipe Association (ACPA), 1988, *Concrete Pipe Handbook*.

Boes, R.M., and Hager, W.H., 2003a. Two-phase flow characteristics of stepped spillways. ASCE Journal of Hydraulic Engineering, 129(9): 661-670.

——, 2003b. Hydraulic design of stepped spillways. ASCE Journal of Hydraulic Engineering, 129(9): 671-679.

——, 2005. Closure to "Hydraulic design of stepped spillways" by Robert M. Boes and Willi H. Hager. ASCE Journal of Hydraulic Engineering, 131(6): 527-529.

Brater, E.F., and King, H.W., 1976, *Handbook of Hydraulics*, McGraw-Hill Book Company, New York, NY.

Bureau of Public Roads, 1978, *Hydraulics of Bridge Waterways, Hydraulic Design Series No. 1,* FHWA EPD-86-101 HDS 1.

Chanson, H., 1994a. Comparison of energy dissipation between nappe and skimming flow regimes on stepped chutes. IAHR, Journal of Hydraulic Research, 32(2), 213-218.

——, 1994b. Hydraulic design of stepped cascades, channels, weirs and spillways. Pergamon, Oxford, UK.

——, 2001. Hydraulic design of stepped spillways and downstream energy dissipators. Dam Engineering, 9(4): 205-242.

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

Gonzales, C.A, and Chanson, H., 2007. Hydraulic design of stepped spillways and downstream energy dissipators for embankment dams. Dam Engineering, 17(4): 223-244.

Hager, W., 1987, *Lateral Outflow Over Side Weirs*, Journal of Hydraulic Engineering, 113(4), ASCE, New York, New York.

Heggen, R.L., 1991, *Hydraulics of Channel Access Structures*: University of New Mexico, Albuquerque, NM.

Knapp, R.T., 1951, *Design of Channel Curves for Supercritical Flow*, ASCE.

Lane, E.W., 1935, *Security from Under Seepage*, Transactions, American Society of Civil Engineers, Vol. 100.

Maricopa Association of Governments (MAG), 1998, *Uniform Standard Specifications for Public Works Construction*, (Includes Revisions Through 2011), Arizona. http://www.azmag.gov/Documents/2011_Specifications_Book.pdf

McLaughlin Water Engineers, Ltd., 1986, *Evaluation of and Design Recommendations for Drop Structures in the Denver Metropolitan Area*, prepared for the Urban Drainage and Flood Control District, Denver, Colorado.

Metcalf and Eddy, 1972, Wastewater Engineering, McGraw-Hill, New York.

Peterka, A.J., 1984, *Hydraulic Design of Stilling Basins and Energy Dissipators, Engineering Monograph No. 25*, United States Department of the Interior, Bureau of Reclamation.

Rhone, T.J., 1977, *Baffled Apron as Spillway Energy Dissipator*, American Society of Civil Engineers Journal of Hydraulics, No. HY12.

Searcy, James K., 1967. "Use of Riprap for Bank Protection," Federal Highway Administration, Washington, D.C., pp. 43.

Simon, A.L., 1981, *Practical Hydraulics*, Jon Wiley & Sons, New York, N.Y.

Simons, D.B., Stevens, M.A., and Watts, F.J., 1970, *Flood Protection at Culvert Outlets*, Colorado State University, Fort Collins, Colorado, CER 69-70, DBS-MAS-FJW4.

Smith, C.D., 1975, *Cobble Lined Structures*, Canadian Journal of Civil Engineering, Volume 2.

Smith, C.D., and Strang, D. K., 1967, *Scour in Stone Beds*, Proceedings of 12th Congress of the International Association for Hydraulic Research.

Stevens, Michael A., 1981, *Hydraulic Design Criteria for Riprapped Chutes and Vertical Drop Structures,* prepared for Urban Drainage and Flood Control District, Denver, Colorado.

Stevens, M.A., Simons, D.B. and Lewis, G.L., 1976, *Safety Factors for Riprap Protection*, American Society of Civil Engineers Journal of Hydraulic Engineering, Paper No. 12115, HY5 pp. 637-655.

U.S. Army Corps of Engineers (USACE), 1991, *Hydraulic Design of Flood Control Channels*.

——, 1994, *Hydraulic Design of Flood Control Channels Change 1* ENG 4794-R, Engineering Manual 1110-2-1601.

——, 2008, *HEC-RAS River Analysis System*, Hydraulic Reference Manual.

U.S. Department of Agriculture, Soil Conservation Service (SCS), 1976, *Chute Spillways, Section 14, SCS Engineering Handbook*.

U.S. Department of the Interior, Bureau of Reclamation (USBR), 1974, *Design of Small Canal Structures.*

——, 1976, *Design of Gravity Dams.*

——, 1977, *Design of Small Dams, p 341-342.*

——, 1987*, Design of Small Dams, p353, p366-367, p453-470*.

——, 2006, Research state-of-the-art and needs for hydraulic design of stepped spillways. U.S. Department of the Interior. Hydraulic Laboratory Report HL-2005-06.

U.S. Department of Transportation (USDOT) Federal Highway Administration, 1983, *Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14*. [USDOT Hydraulics WEB Site]

——, 1988, *Design of Roadside Drainage Channels with Flexible Linings, Hydraulic Engineering Circular No. 15, Publication No. FHWA-IP-87-7*.

——, 1989, "Design of Riprap Revetment," Hydraulic Engineering Circular No. 11 (HEC 11), FHWA-IP-89-016, Washington, D.C.

——, 2001, *Bridge Scour and Stream Instability Countermeasures, Hydraulic Engineering Circular No. 23*, FHWA NHI-01-003.

——, 2006, *Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14, 3rd Edition*. [USDOT Hydraulics WEB Site]

U.S. Navy, 1982, *Design Manual, Foundations and Earth Structures, Naval Facilities Engineering Command, NAVFAC DM-7.2*.

Urban Drainage and Flood Control District (UDFCD), Revised 2008, *Drainage Criteria Manual*, Project Consultant Wright Water Engineers.

8.9.2 References Relevant to Chapter

Fletcher, B. P., and J. L. Grace, 1972, *Practical Guidance for Estimating and Controlling Erosion at Culvert Outlets*, Misc. Paper H-72-5, U.S. Army Waterways Experiment Station, Vicksburg, Mississippi.

Godi, Donald H., and Associates, Inc., 1984, *Guidelines for Development and Maintenance of Natural Vegetation*, Urban Drainage and Flood Control District, Denver, Colorado.

Hughes, C., 1976, *Rock and Riprap Design Manual for Channel Erosion Protection*, University of Colorado.

Mussetter, Robert A., Simons, Li and Associates, Inc., Fort Collins, Colorado, 1983, *Equilibrium Slopes Above Channel Control Structures*, published in Symposium on Erosion and Sedimentation, Ruh-Ming Li and Peter F. Lagasse, Simons, Li and Associates, Inc, co-editors.

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

Powledge, George R., and Dodge, R.A., 1985, *Overtopping of Small Dams—An Alternative for Dam Safety*, Bureau of Reclamation, Engineering and Research Center, Denver, Colorado; published in Hydraulics and Hydrology in the Small Computer Age, Volume 2, William R. Waldrop, editor.

Posey, Chesley J., 1955, *Flood-Erosion Protection for Highway Fills*, with discussion by Gerald H. Matthes, Emory W. Lane, Carl F. Izzard, Joseph N. Bradley, Carl E. Kindsvater, and Parley R. Nutey, ASCE.

Reese, Anderson J., 1986, *Nomographic Riprap Design*, Hydraulics Laboratory, U.S. Department of the Army.

Reeves, Gary N., 1985, *Planned Overtopping of Embankments Using Roller Compacted Concrete*, Hydraulics and Hydrology in the Small Computer Age, Volume 2, William R. Waldrop, editor.

Richardson, E.V., Simons, D.B., and Lagasse, P.F., 2001, *River Engineering for Highway Encroachments, Highways in the River Environment, Hydraulic Design Series No. 6*, U.S. Department of Transportation, Federal Highway Administration. [USDOT Hydraulics WEB Site

Sabol, George V., and Martinez, R.J., 1982, *Energy Dissipator/Grade Control Structures for Steep Channels, Phase II,* Albuquerque Metropolitan Arroyo Flood Control Authority and City of Albuquerque, Civil Engineering Department, New Mexico State University.

Shen, Hsieh Wen, Editor, 1971, *River Mechanics, Vol. I and II*, Colorado State University.

Shields, F. Douglas Jr., 1982, *Environmental Features for Flood Control Channels, Water Resources Bulletin, Vol. 18, No. 5*.

Simons, D.B., 1983, *Symposium on Erosion and Sedimentation*, Ruh-Ming Li and Peter F. Lagasse, Simons Li and Associates Inc., co-editors.

Urban Drainage and Flood Control District (UDFCD), 2001, *Drainage Criteria Manual*, Project Consultant Wright Water Engineers.

U.S. Department of Agriculture, Soil Conservation Service, 1977, *Design of Open Channels, Technical Release No. 25*.

——, 1976, *Hydraulic Design of Riprap Gradient Control Structures, Technical Release No. 59*.

9 STORMWATER STORAGE

TABLE OF CONTENTS

9.1 SYMBOLS

The following symbols will be used in equations throughout Chapter 9.

A	=	Drainage area, acres
C	=	Runoff coefficient, see Table 3.2 of the Hydrology Manual
d	=	Depth, feet or inches
H	=	Riprap toe thickness, feet or inches
P	=	Rainfall depth, inches
V	=	Calculated runoff volume, acre-feet
T	=	Riprap layer thickness, feet or inches

9.2 INTRODUCTION

Stormwater storage facilities are man-made storage structures intended to mitigate the negative impacts of urbanization on storm drainage, which include

- Increased peak flow rates.
- Loss of natural depression storage.
- Reduction of infiltration capacity in a watershed.
- Reduction of natural vegetation, which, in a natural state, reduces storm runoff through the process of increased infiltration and interception.
- Increased pollutant load in surface runoff.

Types of stormwater storage facilities include detention and retention basins.

Detention Basin - A basin or reservoir where water is stored for regulating storm water runoff. A detention basin uses gravity-flow outlets for discharging the stored runoff. Detention facilities do not reduce the volume of runoff, they do however lengthen the time flow will be present in the watercourse downstream of the facility. Due to the longer duration of flow downstream of detention basins, their use requires greater analysis to verify that peak discharges are not increased downstream. Care must be taken not to size the outlet too large, and a range of flood frequency events should be considered in the analysis. The design intent for the outlet is for post development peak outflows to be equal or less than pre-development flows for the design storm event(s).

Retention Basin - A basin or reservoir where water is stored for regulating a flood, however, it does not have gravity-flow outlets for discharging stored runoff as do detention basins. The stored water is disposed by other means such as infiltration into the soil, evaporation, injection (or dry) wells, low flow outlets, or pumping systems. The low flow outlets have a relatively constant discharge rate under ponded conditions (much less than existing peak discharges) and are intended to drain the basin within 36 hours. The design intent for retention basins is to capture the runoff volume for the design storm frequency and duration.

This chapter presents the engineering methodologies and details associated with the planning, analysis and design of detention and retention facilities. The guidelines herein are intended to achieve the following goals:

1. Design of stormwater storage facilities that satisfy ordinance provisions with regard to hydraulic function and maintainability;
2. Design of stormwater storage facilities that are amenities, and, where possible, incorporate multiple-use concepts; and
3. Design of facilities that will not jeopardize the quality of surface water or groundwater resources.

9.2.1 Interaction with Other Components of a Drainage System

Stormwater storage facilities are components of an overall stormwater management system that is also comprised of natural and man-made channels, storm sewers, inlets, streets and other drainage structures. Their purpose is to provide temporary storage of the stormwater runoff from developed areas and to control the increased peak rates of runoff. Proper planning and design of stormwater storage facilities must consider the interaction of storage with the other components of the drainage system.

The greater the number of detention facilities in a system, the more complex is the analysis of the interaction of the various discharges. Often the increased costs of construction and maintenance of a large number of smaller detention facilities offset any savings in reduced sizes of storm sewers downstream. Planning efforts should be oriented toward minimizing the number of detention facilities within a watershed. The converse is true for retention facilities due to their additional storage and lack of primary outlet structures. Effective flood control using retention does not depend on the nuances of hydrograph shape, just runoff volume.

As part of the planning and design process, the engineer must verify that releases from the stormwater storage facility will not adversely impact downstream conditions in terms of both manner and quantity of flow. Conditions such as peak flow, velocity, flow concentration, prolongation of flow and quality of discharge are factors to be considered.

9.2.2 Limitations on Use of Stormwater Storage Facilities

The requirement for a development to provide stormwater storage facilities will not be waived unless determined otherwise by the jurisdictional agency on a case by case basis. The use of detention instead of retention will also be reviewed on a case by case basis. Retention is the preferred stormwater storage method in Maricopa County. Whenever possible, the facilities shall be designed for multiple uses, such as parks or other recreational facilities, to offset the cost of open space and to encourage improved maintenance.

Residential developments (recorded subdivisions) shall not provide for nor rely on single-lot, on-site stormwater storage, and the design of common facilities shall not assume any individual lot on-site storage, unless approved by the jurisdictional agency. Developments with Homeowner's Associations will locate its facilities in private drainage tracts or in public sites dedicated by the developer, in accordance with the jurisdictional agency’s requirements. The Homeowner’s Association will maintain the private facilities, and the jurisdictional agency will usually maintain the public tracts. Common storage facilities for single family developments without a Homeowner's Association and with public streets will have maintenance provisions determined by the jurisdictional agency. The number and location of storage facilities within a development are to be approved by the jurisdictional agency. Dedication to the public may require the inclusion of recreational facilities or other features deemed necessary by the jurisdictional agency.

Single lot, non-residential developments that are not served by a public stormwater storage facility will provide the required storage on the lot itself and outside the right-of-way area, regardless of lot size. Maintenance shall be provided for by the property owner.

Single lot, residential parcels that are not a part of a recorded subdivision, such as lots created by parcel splits and minor land divisions, will also provide the required storage on the lot itself and outside the right-of-way area, if it is demonstrated that a common basin with adjacent parcels is not practical. Each jurisdictional agency may establish lot size requirements governing the application of this requirement, but in all cases the residential lots smaller than 1 acre in size shall provide the required storage.

Regional Stormwater Storage Facilities - Regional detention/retention facilities are large storage facilities located at strategic sites within a watershed to provide control of runoff. The regional approach is best suited to watersheds that were primarily developed prior to retention requirements instituted in Maricopa County in the late 1980s. The advantage of this type of facility is that the siting and design of regional storage facilities is normally incorporated as part of an overall drainage master plan. Thus, alternative siting combinations and their respective hydraulic routing effects can be investigated. Storage alternatives can be evaluated with other factors (that is, conveyance system, land and maintenance costs), to arrive at an optimal solution to alleviate flooding problems within the drainage basin.

9.3 DESIGN GUIDELINES

This section presents certain guidelines, procedures and criteria to be used in the analysis and design of detention and retention facilities. Because specific policies and criteria vary, the designer must contact the jurisdictional agency for the area in which the basin will be located before beginning design.

9.3.1 Guidelines for Stormwater Storage Facilities

The following general guidelines apply to the design of stormwater storage facilities.

Design Frequency

All stormwater storage facilities incorporated within new developments will be designed to retain the peak flow and volume of runoff from the design storm event. The duration and intensity of the design storm event is designated by the jurisdictional agency for the given area under study. In the special case of when a detention only facility is allowed, the requirement to retain the design storm runoff volume may be waived. However, the peak discharge requirement must still be met, and the effects of using a detention only facility on more frequent events must be determined.

In jurisdictions where multi-frequency control is required, the design will be prepared to regulate the peak discharge rates for one or more storm events in addition to the design storm. Specific multi-frequency events shall be verified with the appropriate jurisdiction, but it is recommended that the 2-, 10- and 100-year events be verified, as a minimum.

Hydrology

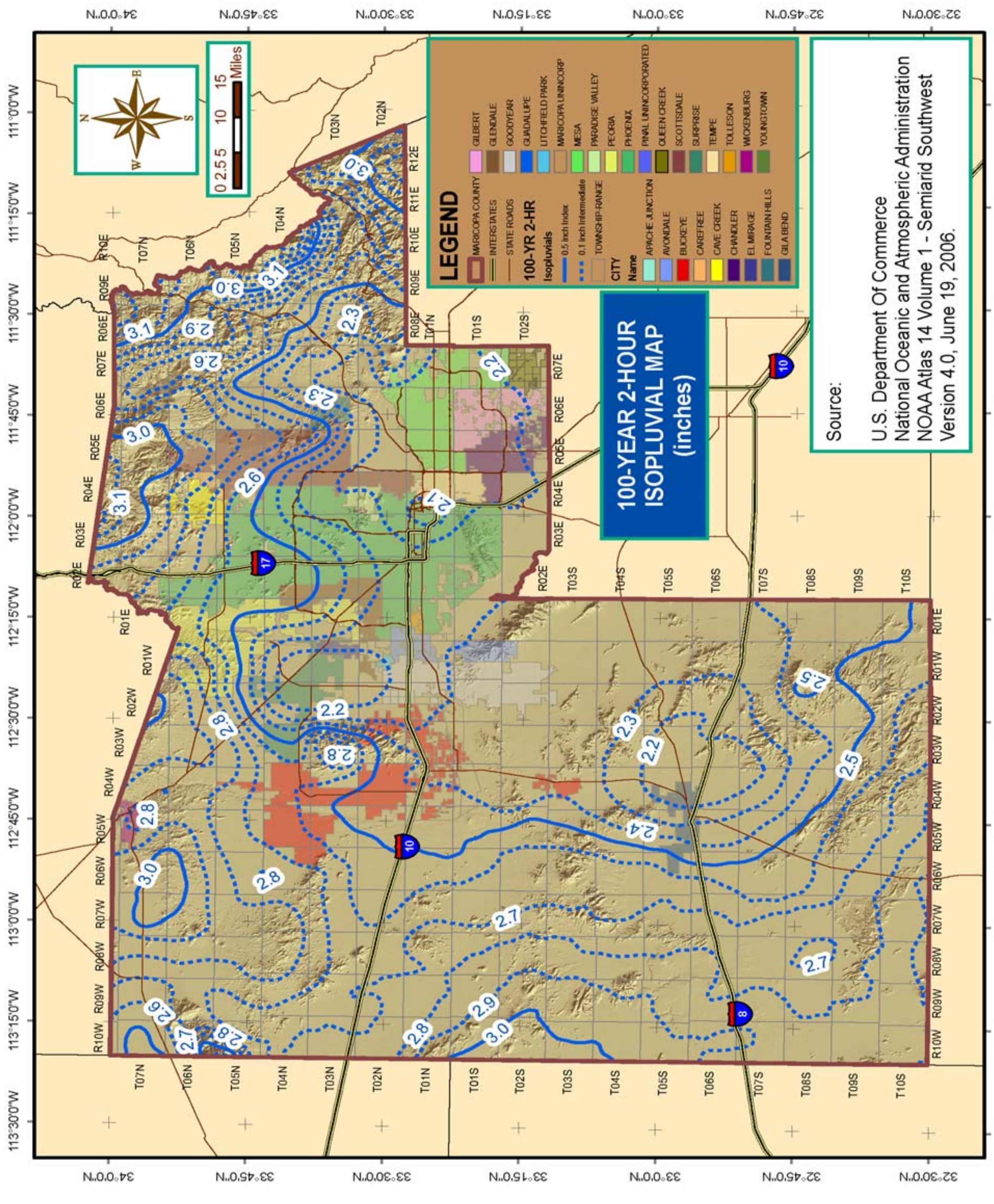
Procedures and criteria for development of inflow hydrographs for stormwater storage facilities are described in the *Drainage Design Manual for Maricopa County, Hydrology*.

Volume Calculations For Retention Facilities - Some jurisdictional agencies have developed simplified equations for determining the volume required for retention. The engineer should verify the methodology for calculation of the required storage volume with the appropriate jurisdiction. Where the rational method is approved for use, volume calculation should be done by applying the following equation:

$$V = C\left(\frac{P}{12}\right)A \qquad (9.1)$$

In the case of volume calculations for retention design, P equals the design storm depth in inches. The 100-year, 2-hour depth for Maricopa County is shown in Figure 9.1. The amount of rainfall for other frequencies and durations can be determined by using Section 2.2 of the Hydrology Manual.

FIGURE 9.1
ISOPLUVIAL 100-YEAR, 2-HOUR PRECIPITATION



Flood Routing - Routing methods are also acceptable to determine volume, particularly for larger, regional basins. For a typical stormwater detention facility, there are three variables to be considered in flood routing through the structure:

1. Inflow to the facility, which varies as a function of time;
2. Outflow from the facility, which varies as a function of inflow and storage volume; and
3. Storage, which is the result of the difference between the inflow and outflow for a period of time or time interval.

FIGURE 9.2
FLOOD ROUTING (INFLOW AND OUTFLOW HYDROGRAPH)

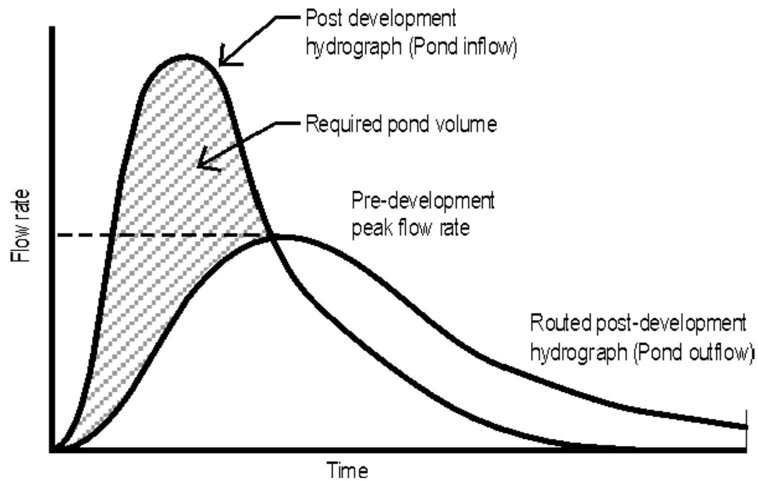


Figure 9.2 illustrates the general relationship among the three variables that must be considered for flood routing through a structure.

The outflow hydrograph from a proposed stormwater detention facility shall be determined using the “Storage Indication” or “Modified Puls” method of flood routing. Other similar hydrologic routing methods may also be used, provided that the chosen method is first approved by the appropriate review agency. Numerous computer software programs such as HEC-1 (USACE, 1990) have been developed for flood routing through detention facilities. Use of a particular computer program should be approved by the appropriate jurisdictional agency prior to its application on a particular project. If a computer program for flood/reservoir routing is intended to be used, documentation of the program shall be submitted to the appropriate review agency prior to commencing design. Non-tributary flows may not be routed through a detention facility unless specifically approved by the jurisdictional agency. Off-site flows should not be routed through a stormwater storage facility unless specifically approved by the appropriate jurisdictional agency.

Detention ponds in series (that is, when the discharge of one facility becomes the inflow of another) are complex and require special consideration and design by a hydraulic engineer. If such a system is unavoidable, the engineer must submit a hydrologic and hydraulic analysis which demonstrates the system's adequacy. This analysis must incorporate the construction of hydrographs for all inflow and outflow components, and rating curves for hydraulic structures.

Sedimentation in Stormwater Storage Basins

Depending on the watershed, sediment deposition into stormwater storage basins may be significant enough to reduce storage volume. Therefore, it is important during the design process to estimate the sediment yield from the watershed and add this volume to the storage volume. Refer to Chapter 11 of this manual for more information.

Siting and Geometry

With respect to siting, stormwater storage facilities which utilize a method of subsurface disposal shall be located such that the infiltration surface will be a specific distance, both horizontal and vertical, from any functioning water well. The appropriate jurisdictional agency should be contacted regarding regulations governing the siting of such facilities near wells or near the static groundwater table.

Basic requirements regarding facility shape, side slopes, depth and bottom configuration are provided below. Additional details are presented in Section 9.4, Section 9.5, and Section 9.6 in conjunction with guidelines regarding safety, operation and maintenance, aesthetics, and multiple use considerations.

Shape - As a general rule, curvilinear, irregularly shaped facilities will have the most natural character. A wide range of shapes can be considered and utilized to integrate the stormwater storage facility with the surrounding site development. Smooth curves should be used in the plan layout of the grading for the facility.

Side Slopes - Where grass is intended to be established, side slopes shall not be steeper than 4 horizontal to 1 vertical. Where other protection measures are intended, such as shrub planting, rock riprap or other structural measures, slopes shall not exceed 3 horizontal to 1 vertical unless approved by the appropriate jurisdictional agency. Where slopes abut the street right-of-way, the minimum slope shall be 4 horizontal to 1 vertical regardless of surface treatment. Some jurisdictions may require a flatter slope. The designer should verify the slope requirement prior to commencing design.

Transitions from slopes to level ground at the top and bottom of a facility shall be smooth curves. In all cases, slopes must be designed to allow for safe operation of maintenance equipment. Refer to Section 9.5.1 for maintenance access provisions. Side slope design should be done with the visual character of the completed facility in mind. A more natural appearance can be achieved by varying side slopes within a stormwater storage area.

Depth and Bottom Configuration - Maximum ponding depth and freeboard requirements vary within Maricopa County and specific criteria for such must be verified by the designer with the appropriate jurisdictional agency. With respect to grading, deep facilities should be avoided, if possible. For facilities with a depth in excess of 3 feet, consideration should be given to the use of flatter side slopes or the provision of intermediate benches along side slopes. For a detention facility, the bottom shall be designed to drain to a low flow channel.

Drain Time

The design of all stormwater storage facilities shall be such that the stored runoff is completely discharged from the facility within 36 hours after the runoff event has ended. The draining of stormwater storage facilities may be accomplished via in-situ percolation, bleeding off (low flow) outlets, drywells, pump station, or a combination thereof.

Lining/Surface Treatment

In keeping with the goal of stormwater storage facilities as amenities that incorporate multiple use concepts where possible, grass and/or landscape plantings are preferred surface treatments. As a general rule, grass and plant species used for landscape development and revegetation should be native to Maricopa County. A registered landscape architect should prepare the landscape design with consideration toward use of plant species appropriate for the level and frequency of inundation of the facility. Permanent irrigation systems are required for grass areas and most types of basin revegetation and landscaping. However, use of native and drought tolerant species (including seeding) may only require a temporary system to obtain effective germination and establishment. Whether permanent or temporary, that portion of the irrigation system within the flood zone must be designed to tolerate inundation and silt accumulations.

The use of inert materials is appropriate for stabilization and erosion control where steep slopes are unavoidable, including along channels, at inflow points, at the outlet control structure and any other location where flowing water may threaten stability. Use of these materials should be properly engineered (refer to Chapter 6) and should respond to aesthetic considerations. Inert materials for erosion control include:

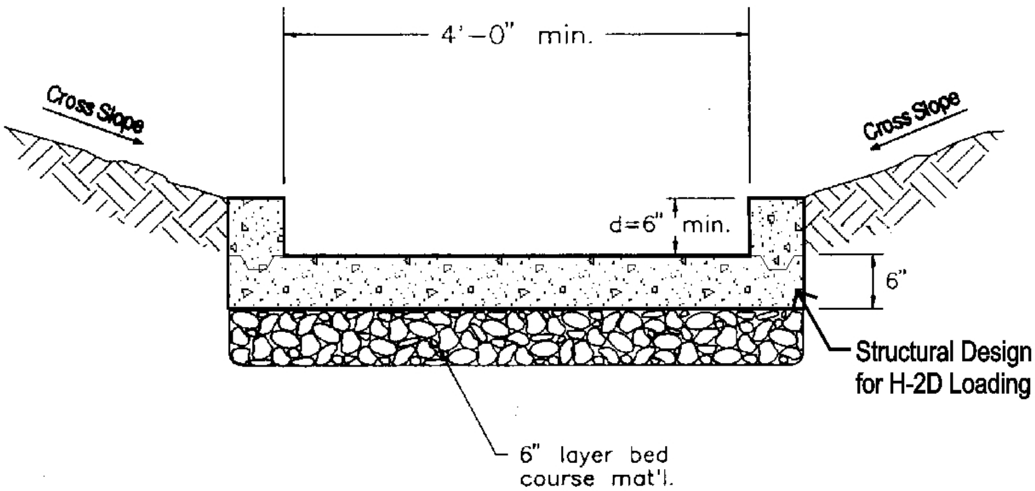
- Loose rock riprap with a specific, engineered gradation
- Loose or grouted boulders (minimum dimension 18 inches and larger)
- River stone
- Gabions
- Soil cement and concrete

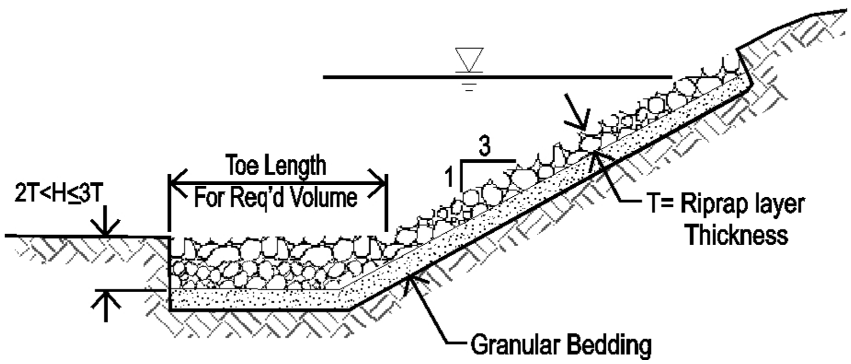
Designs that combine landscape planting with the use of inert materials are recommended. Voids can be designed within the inert material to allow installation of plants. The result is a durable and attractive method of protection.

Low Flow Channels

A low flow channel is required in the bottom of a detention facility to provide positive routing of drainage to the primary outlet structure. An example of a rectangular concrete low flow channel is provided in Figure 9.3. The engineer will provide design of the reinforcement of the channel. The channel shall have a 0.5 percent maximum longitudinal slope. Alternative low flow channel designs may be considered at the discretion of the individual jurisdictional agency; however, use of loose rock or other movable materials can only be made after careful consideration.

FIGURE 9.3
RECTANGULAR CONCRETE CHANNEL SECTION
(Adapted from: WRC Engineering, Inc. 1985)





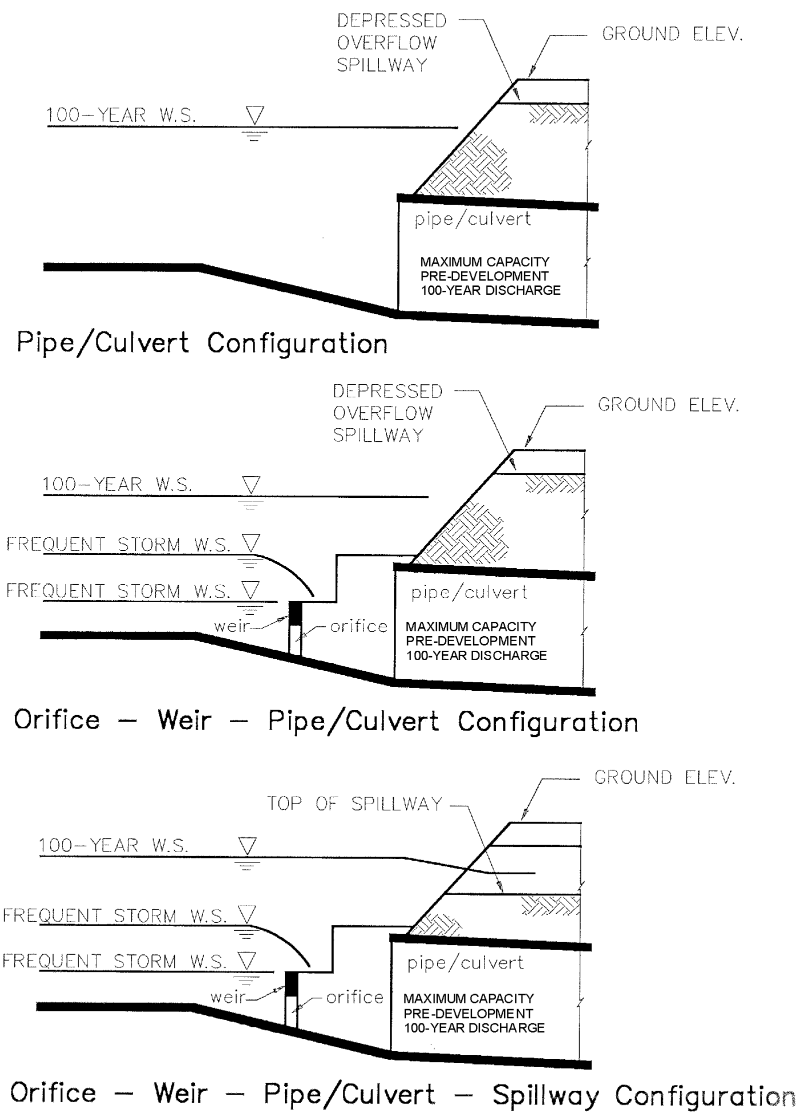
Stormwater Storage Facility Inlet and Outlet Structures

Conveyance of runoff into a retention facility often involves directing the inflow down a slope into the storage area. The design of an inlet structure shall be such that inflow is directed into the facility in a non-erosive manner and without adverse impacts to the facility or to upstream areas. The designer is referred to analysis methods presented in Chapter 6 for the design of inlet structures.

Outlet structures are an important component of stormwater detention facilities since they control the rates of release from the facility, the water depth, and storage volume in the facility. Outlet structures are classified as: 1) primary outlet structures that provide the hydraulic control for the specific design event(s) required by the jurisdictional agency; 2) emergency spillways that provide safe routes, typically via surface overflow, for storm events in excess of the design frequency or in the case of debris blockage or malfunction of the primary outlet structure; and 3) low flow/low level outlets.

Primary Outlet Structures - Jurisdictional agencies may require attenuation of a single frequency storm or a number of frequencies for a given detention facility. Refer to the specific requirements of the jurisdiction where the design is being prepared; however, two-stage and multi-stage control structures are becoming more widely used. Figure 9.4 presents examples of single frequency and multi-frequency outlet control structures. The minimum allowable pipe size for primary outlet structures is 18 inches in diameter.

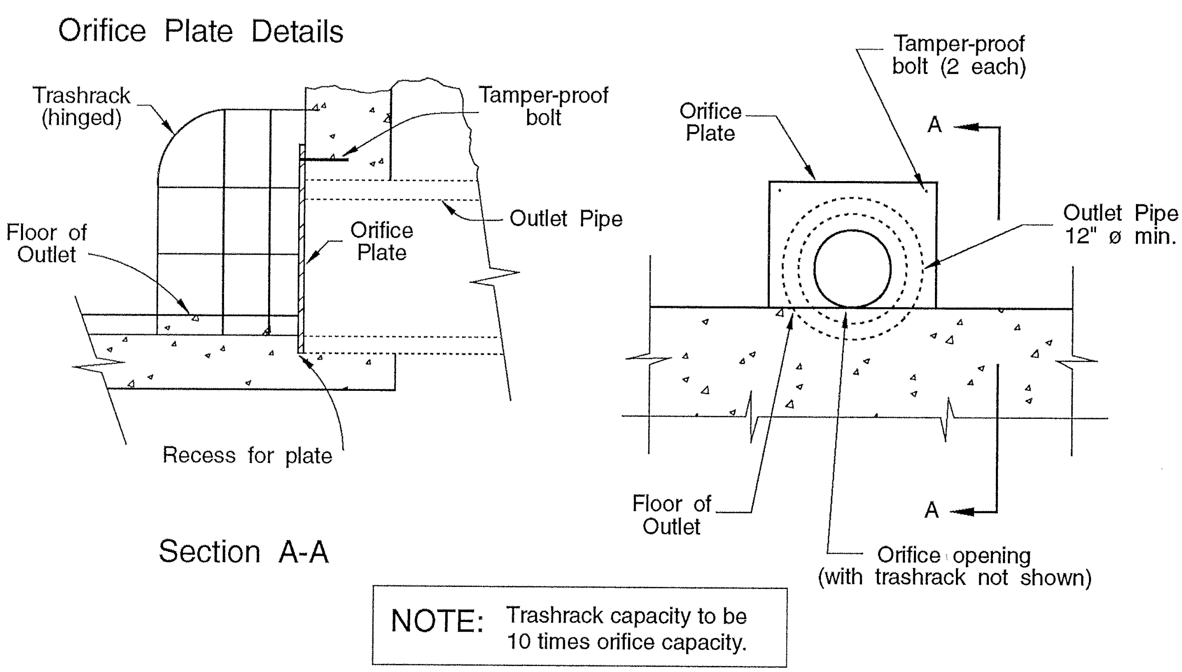
FIGURE 9.4
EXAMPLES OF PRIMARY OUTLET STRUCTURES
(Pima County Department of Transportation and Flood Control District, 1986)



If the flow capacity of an outlet pipe must be further reduced, an orifice plate may be attached, as shown on Figure 9.5(a). The orifice plate must be constructed of heavy, galvanized steel and attached by tamper-proof bolts. Other outlet configurations may be allowed provided they meet the requirements of the permitted release rates at the required volume and include proper provisions for maintenance and reliability.

Primary outlet structures, particularly those controlling multiple storm events, are often special design structures unique to specific site applications. Furthermore, consideration must be given to structural adequacy and flotation under hydrostatic loads.

FIGURE 9.5
ORIFICE PLATE DETAIL
(Adapted from WRC Engineering, 1987)



Trashracks - Trashracks shall be provided to inlets of pipe and orifice outlet structures. See Chapter 8 for hydraulic analysis guidelines and Chapter 1 for safety considerations.

Energy Dissipation at Outlet - Adequate energy dissipation measures shall be provided at the downstream end of primary outlet structures. Such measures shall be designed to control local scour at the pipe outlet and to reduce velocities to pre-development conditions prior to exiting onto the downstream property.

Emergency Spillways - Emergency spillways are normally surface overflow weirs, channels, or combinations thereof, provided for the safe overflow and routing of floodwaters under unusual circumstances. Such situations include the blockage or malfunction of the primary outlet structure or the occurrence of a storm event larger than that for which the facility was designed. Consideration must be given to the layout and configuration of the emergency spillway so that excess flow is safely released and conveyed without increasing flood hazards to adjacent properties and in the same manner and direction as would have occurred under pre-development or historic conditions. Emergency spillways must be designed to convey the unattenuated 100-year peak discharge at non-erosive velocities. For criteria regarding design of emergency spillways for embankments, refer to Section 9.3.3.

Low Flow/Low Level Outlets - For health and safety reasons, stormwater storage facilities must drain within 36 hours for the design storm runoff volume. For stormwater facilities in a series, the cumulative post storm drain time is 36 hours. In addition, the peak discharge from a low flow outlet shall be significantly less than the existing watershed peak discharge for retention facilities. These guidelines form the basis for design. Compliance with NPDES requirements often dictate a third criteria for low flow/low level outlets. Here, the outlet is often designed to retain the first flush and/or the floating hydrocarbon pollutants. In this situation, undershot weirs or inverted siphons may be used. See Section 9.7 for an additional discussion on water quality.

Subsurface Disposal

The primary methods of underground disposal of stormwater runoff at retention facilities are engineered basin floors and drywells. Infiltration rates of basin floors or drywells shall not be used in determining outflow rates in flood-routing procedures.

Engineered Basin Floors - Analysis and design of the bottom of a retention facility intended for subsurface disposal is detailed in *Underground Disposal of Stormwater Runoff Design Guidelines Manual* (USDOT, 1980); refer to that publication for specific design criteria.

Drywells - Drywells may be used for subsurface disposal of stormwater, if approved by the jurisdictional agency, and if criteria such as subsurface strata permeability, groundwater levels and maintenance can be satisfactorily addressed. The main cause of drywell failure is clogging of the transmission media (gravel) by silt and debris. Failure can be avoided by utilizing proper design and installation guidelines, and by following recommended maintenance procedures. Figure 9.6 shows a typical drywell installation, while Figure 9.7 shows examples of surface treatments.

All drywells must be registered with the Arizona Department of Environmental Quality (ADEQ).

FIGURE 9.6
TYPICAL DRYWELL INSTALLATION
([McGuckin Drilling Inc.](#), 1987)

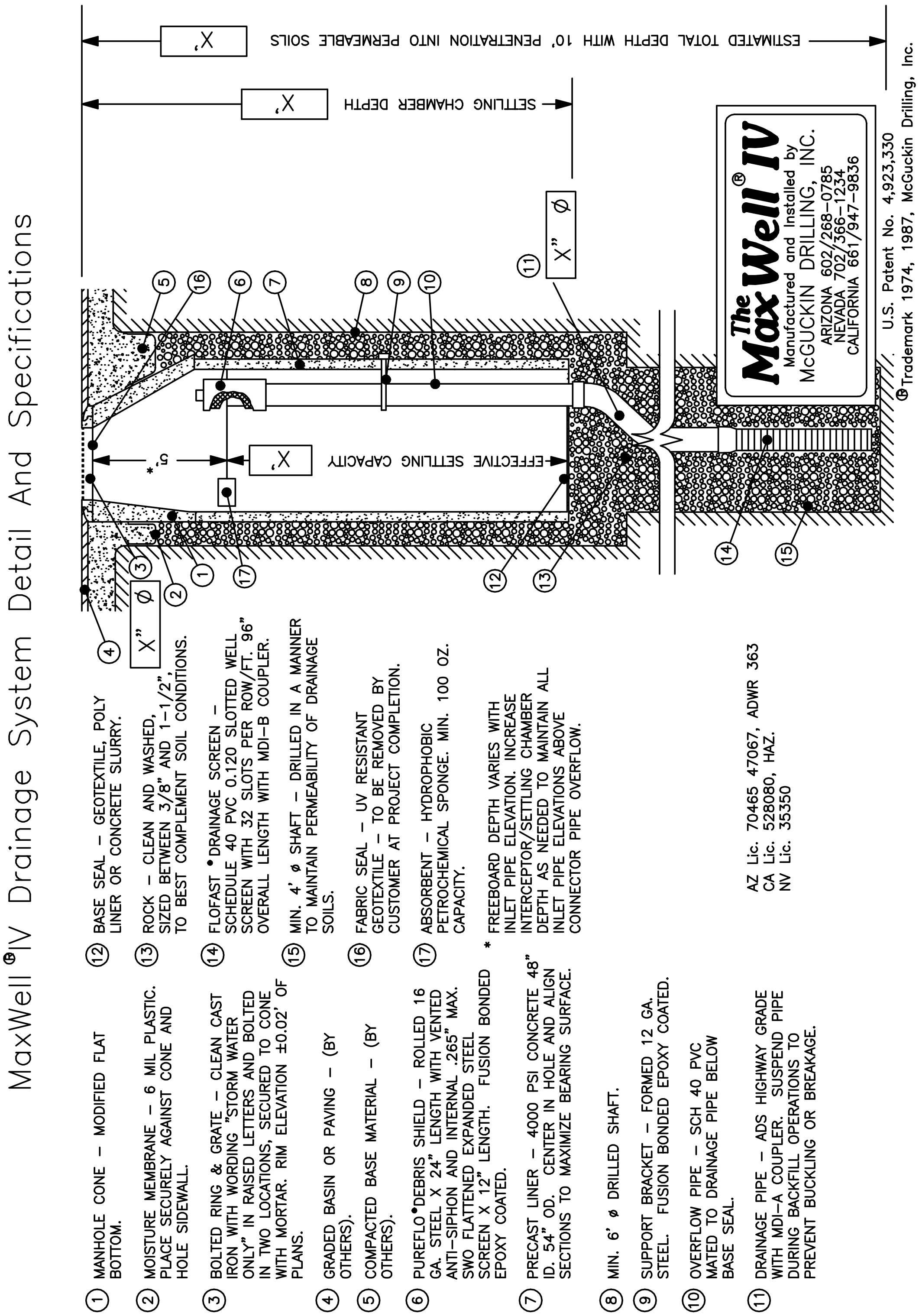
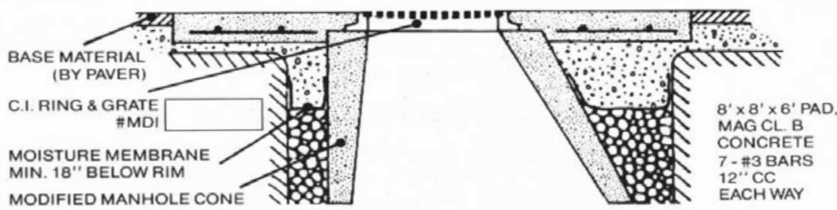


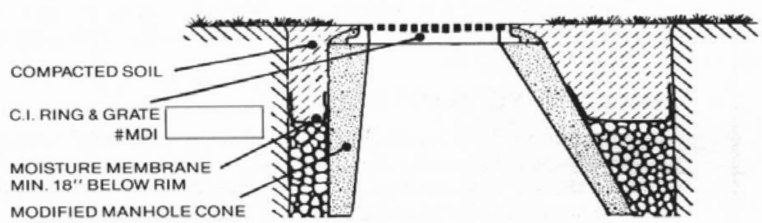
FIGURE 9.7
TYPICAL DRYWELL SURFACE TREATMENTS
(McGuckin Drilling Inc., 1987)

SURFACE TREATMENTS-DRAWING A-B-C OR D MAY BE USED IN PLACE OF THE TOP PORTION OF THE BASIC DRAWING ON PAGE 4, TO ADAPT IT TO THE VARYING SURFACE CONDITIONS SHOWN.

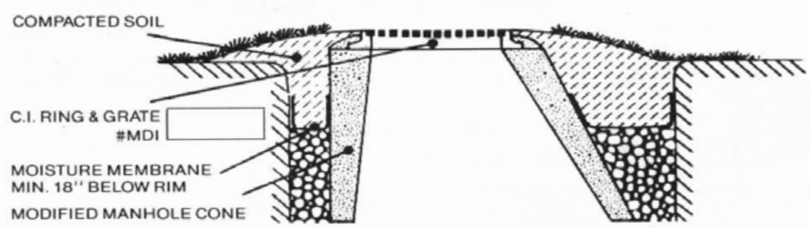
A. Adds a concrete pad for heavy traffic areas.



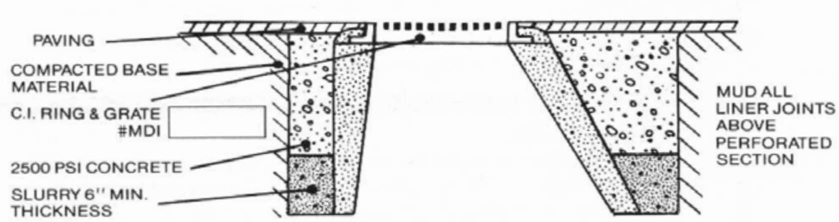
B. For landscaped retention ponds and planters. No paving or pad. In areas where silt might flow to drywell, use C.



C. Use in landscaped retention/detention basins or where heavy silt flow is anticipated. Height should be 4''±.



D. A special design where unstable soil conditions could cause surface subsidence. Also installed with connecting pipes and trenches.



The following list of general requirements and criteria shall be used in the design and construction of engineered basins and drywells (or other methods of subsurface disposal of stormwater). In addition, the engineer is referred to specific policies and standards of the applicable jurisdictional agency.

- Field investigations shall be performed and shall include soil borings and percolation tests taken at the bottom of the proposed basin to obtain percolation rates for use in the design of the stormwater storage facility. Procedures used should be one of the following two methods, listed by order of preference:

  1. ASTM D 3385-03, Double Ring Infiltrometer. If the soils present are outside the accepted range for application of ASTM D 3385-03, then method 2 shall be applied. Soils outside the acceptable range for ASTM D3385-03 are typically very pervious or very impervious with a saturated hydraulic conductivity greater than about 14 inches/hour or less than about 0.0014 inches/hour. Very impervious soils that are outside the range of applicability for ASTM D3385-03 are not suitable for stormwater percolation disposal system applications. Dry wells may be a better choice for these conditions. If there is a question regarding the applicability of this method for the soils at a particular site, ASTM D 3385-03 should be applied and the results checked against the acceptable range of values of hydrologic conductivity. ASTM D 3385-03 may also not be applicable for dry or stiff soils that will fracture when the rings are installed, or gravels that do not allow penetration by the rings.

  2. EPA Falling Head Percolation Test Procedure from *Design Manual - Onsite Wastewater Treatment and Disposal Systems* (EPA, 1980).

  A minimum of two (2) tests are required per retention basin. Refer to the Policies and Standards manual of the jurisdictional entity for the type and total number of tests required. The recommended testing frequency, based on the basin bottom area proposed for percolation, is listed in Table 9.1. Each test should include one soil log hole and one percolation test. Each soil log boring hole should extend at least 10-feet below the bottom of the proposed basin. A soil horizon log should be prepared for each boring to obtain the approximate soil texture of each soil layer (horizon) observed and to identify soil horizons that may impede percolation.

TABLE 9.1
MINIMUM QUANTITY OF SOIL LOG HOLE/PERCOLATION TESTS REQUIRED

Retention Basin Bottom Area (A_p) (sf)	Minimum Number of Tests Required
$A_p < 10,000$	2
$10,000 \le A_p < 20,000$	3

TABLE 9.1
MINIMUM QUANTITY OF SOIL LOG HOLE/PERCOLATION TESTS REQUIRED

Retention Basin Bottom Area (A_p) (sf)	Minimum Number of Tests Required
$20,000 \le A_p < 30,000$	4
$30,000 \le A_p < 43,560$	5
$A_p > 43,560$	A minimum of 5. Additional percolation tests may be required if the soil borings indicate variation in soil texture within the proposed percolation area.
The tests should be distributed evenly throughout the retention basin using engineering judgment. For example, when 5 tests are required, the typical distribution assuming a square basin would be a test in each corner and one in the middle.	

Field percolation test values should be reduced by a safety factor when designing any percolation facility (Stahre and Urbonas, 1990). This is necessary because soils will tend to clog with time, which has proven to be a significant cause for basin failure to drain within 36-hours in Maricopa County. The de-rating factors for Method 2 (EPA, 1980) should include negation of sidewall percolation and a higher degree of uncertainty in the results when using this approach. Recommended de-rating factors are shown in Table 9.2. The selected percolation rate should then be de-rated using Equation (9.2). The tests shall be performed by a testing laboratory, and the results sealed by a civil engineer, licensed to practice in the State of Arizona. Stormwater disposal by percolation is not recommended if the percolation rate, after application of the de-rating factor, is less than 0.5 inches per hour. Stormwater disposal by percolation is also not recommended if groundwater or an impermeable layer is encountered within 4-feet below the bottom of the basin.

$$P_d = \frac{P_r}{D_r} \tag{9.2}$$

Where P_d is the design percolation rate in inches/hour, P_r is the lowest measured percolation rate in inches/hour, and D_r is the de-rating factor.

Basin drain time is estimated by using Equation (9.3).

$$T_d = \frac{V}{A_p \frac{P_d}{12}} \tag{9.3}$$

Where T_d is the retention basin drain time in hours, A_p is the percolation area of the basin bottom in acres, P_d is the design percolation rate in inches/hour, and V is the retention basin design storage volume (100-year, 2-hour) in acre-feet.

Only the bottom area of the retention basin may be used for computing the basin drain time by infiltration/percolation. The side slope areas shall not be used in the drain time computation unless the basin configuration is "V" shape without a flat bottom. For a "V" shaped basin without a flat bottom, the bottom area assumed available for percolation is recommended to be computed using Equation (9.4).

$$A_p = \frac{(D/3)(SS_L + SS_R)L}{43,560} \tag{9.4}$$

Where *D* is the design ponding depth in feet, *L* is the length of the basin in feet, and SS_L and SS_R are the left and right basin side slopes in feet horizontal per foot vertical.

TABLE 9.2
RECOMMENDED MINIMUM PERCOLATION DE-RATING FACTORS
(FOR RETENTION BASIN DESIGN)

Condition	De-Rating Factor				
	Method 1	Method 2 (by test hole diameter)			
		6-inch	8-inch	10-inch	12-inch
No groundwater or impermeable layer is encountered within 10-feet below the bottom of the basin, and the soils are of similar texture to those where the percolation test is taken. The geotechnical engineer may specify a higher de-rating factor based on analysis of the soil conditions below the basin bottom.	2	10	8	7	6
Groundwater or an impermeable layer is encountered within 4-feet to 10-feet below the bottom of the basin.	4	20	16	14	12

- Drywells shall be designed, operated, and maintained in conformance with the most current ADEQ guidelines. EPA (1980) procedures may be used for estimating initial design percolation rates. The final design rate is recommended to be based on a constant-head percolation test performed on each completed well at the site. Refer to the Policies and Standards of the jurisdictional agency for design standards for drywells. The test results for each well should be de-rated based on the in-situ soil conditions. A de-rating factor of 2 is recommended for coarse-grained soils (cobbles, gravels and sands). A de-rating factor of 3 is recommended for fine grained soils (silts and loams). A de-rating factor of 5 is recommended for clay soils. These de-rating factors are required to compensate for deterioration of the percolation capacity over time in addition to providing a factor of safety for silting and grate obstruction. The accepted design disposal rate for a dry well, after application of the de-rating factor, should not be less than 0.1 cfs per well. The maximum allowable rate, after application of the de-rating factor, is not recommended to exceed 0.5

cfs per drywell for design purposes. It is the owner's, or owner's representatives', responsibility to clean and maintain each dry well to ensure that each remains in proper working order. The regular maintenance schedule is not recommended to exceed 3-years. Drywells that cease to drain a retention basin with 36-hours should be replaced or refurbished by the owner or his representative. Maintenance requirements are to be written in the CC&R's for subdivisions where dry wells are used to drain retention basins. In accordance with ADEQ requirements, the installation of any subsurface drainage structure must be located into a permeable porous strata at least 10-feet above saturated soils and 100-feet away from any water supply well.

- A test well shall be installed for any retention facility utilizing drywells for stormwater disposal. Upon approval of performance, adjusted as presented above, this test well may then be used as one of the functioning drywells within the retention facility.

- The design of a drywell must include provisions for trapping sediment within a settling chamber. The system shall use a floating absorbent blanket or pillow to enhance the removal of petroleum-based organics floating on the water. A hydrophobic petrochemical absorbent with a minimum capacity of 100 ounces per chamber is recommended. This measure will significantly increase both the efficiency and useful life of the well. Once a year, at a minimum, the settling chamber should be inspected, and it should also be inspected after any major inflow to the drywell. Sediment shall be removed from the chamber at such time that approximately 15 to 20% of the original volume of the chamber is filled. All sediment removed from a settling chamber shall be disposed of either at an authorized sanitary landfill or at any other suitable location approved by the governing jurisdiction.

- Infiltration rates of drywells shall not be used in determining outflow rates in flood-routing procedures. Any retention facility which relies solely upon infiltration as its method of drainage shall be sized to contain the maximum storage volume that would be required without considering an outflow rate.

- Disposal methods using infiltration shall not be permitted for stormwater runoff which carries significant concentrations of sediment. This includes stormwater runoff flowing through sand bed channels, as well as stormwater runoff emanating from a predominantly natural watershed.

- During site development, all drywells shall be securely covered with filter cloth or other material to prevent the introduction of excessive sediment into the settling chamber.

- Retention of runoff emanating from industrial developments and infiltration of runoff to the subsurface will be handled on a case-by-case basis by the appropriate reviewing agency subject to water quality concerns.

- Runoff stored in a retention facility shall be completely drained from the facility within a maximum time period of 36 hours after the runoff event has ended. Drywells that cease to drain a facility within the 36-hour period shall be replaced by the owner with new ones, unless an alternate method of drainage is available.

Permanent Pools

- Certain jurisdictions permit the design of a stormwater storage facility that incorporates a permanent pool for aesthetic purposes. The engineer should contact the appropriate jurisdiction for specific criteria and regulations regarding such facilities. General considerations for facilities incorporating permanent pools are listed below:

- Flood storage volume shall be maintained above the level of the permanent pool. Provision for draining the full depth of the pond shall be included at the outlet structure.

- Maintenance of a minimum water level should be provided either by the inflow from the watershed, and/or by augmentation from other sources during prolonged dry periods and by the capability of the bottom of the facility to retain water. Seepage and evaporation losses shall be considered.

- Maintain water quality and minimize algae growth by designing for sufficient minimum depth and incorporating use of recirculation and aeration measures.

- Consider public safety as primary in the design of all features related to the permanent pool.

- Geometric characteristics of the pond include:

  –Choose bottom lining material suitable for retention of water and with consideration toward maintenance (that is, ease of sediment removal, etc.). Provisions for completely draining the pond should be made.

  –Create aesthetic yet maintainable edges. Edge design also should consider the effect of drawdown of the water surface. That is, a drop in water surface elevation should not create a wide expanse of unsightly shoreline. Similarly, the area surrounding the permanent pool should be designed for periodic inundation. The area should drain completely and return to a stable surface following a flood event.

  –Provision of stable side slopes above and below the permanent water surface.

  –The pond edge shall be designed to minimize safety hazards. Water depth should be limited to 1.5 to 2 feet within 8 feet of the shoreline.

  – Resolve permanent pool water depth issues versus safety needs; a 3-foot depth at shoreline required to limit pond edge vegetation growth exceeds the recommended pond edge depth (1.5 to 2.0 feet). Therefore, other safety measures must be considered (see Section 9.4).

- The design should consider measures to minimize sediment inflow to the pond. Once sediment has entered the permanent pond, then removal can be expensive and may require draining the pond. Erosion should ideally be controlled at the source or by mitigation measures along the incoming channel. However, if such measures are not feasible, a sediment trap should be designed at the pond inflow location to intercept the majority of the incoming sediment and to facilitate removal (see Section 9.7.4).

- If the stormwater storage facility and permanent pool are created by a retaining structure, such as an earth embankment, then the design guidelines for embankments shall be followed, with particular emphasis on seepage control and embankment stability (see Section 9.3.3).

- Potential impacts downstream shall be considered. The designer should be aware that an impoundment may improve, worsen or maintain existing downstream flow characteristics, and that any changes, even apparent improvements, may be viewed as infringements of downstream riparian rights.

- Since a permanent pool is most often desired for creation of a focal amenity for a development, it is appropriate that a registered landscape architect work in conjunction with the engineer to achieve an aesthetic design with consideration of costs of construction and maintenance.

9.3.2 Criteria for Special Stormwater Storage Methods

Methods of stormwater storage include underground storage, conveyance storage, roadway embankment storage, and storage in parking lots, pedestrian plazas, courtyards and common areas.

Subject to the specific policies and standards of the local jurisdictional entity, the use of rooftops as storage areas for runoff is generally not permitted. Furthermore, basins established in the bottoms of channels are generally not permitted since these are prone to on-going sedimentation problems.

Application of the special measures discussed below is regulated according to specific jurisdictions. Contact the local jurisdiction before beginning to design using any of these methods.

Since the following methods often result in facilities near buildings, it should be emphasized that the finished floor elevation of a structure shall be a minimum of 1 foot above the 100-year water surface of the stormwater storage facility. The finished floor elevation shall also be above the emergency outfall for the basin.

Underground Storage

This type of storage involves the construction of underground tanks, pipes, or vaults, which accept stormwater runoff by means of inlets and storm drain pipes. Due to the high cost of this type of installation, it is generally limited to high-density developments, where surface storage is not feasible due either to the scarcity or high cost of land, or both.

Underground storage facilities must be provided with some method of outfall (that is, gravity drains, pumps, or infiltration). In all cases, manholes (or some other means of access to the underground storage facilities) must be provided for maintenance purposes.

Conveyance Storage

During the period that channels and floodplains are filling with runoff, the stormwater is being stored in transient form. This type of storage is known as conveyance storage. Construction of slow velocity channels with large cross sectional areas assist in the accomplishment of such storage. Conveyance storage systems are usually feasible only on large projects, and require detailed hydrologic modeling for analysis.

Roadway Embankment Storage

When feasible, use of roadway fill slopes as an embankment for a stormwater storage basin provides an economical means of stormwater storage. Special considerations must be given both to the stability of the embankment and to the protection of the embankment from erosion. Additionally, State of Arizona dam safety requirements may need to be addressed if the embankment height and/or the potential storage volume exceeds certain limits (see Section 9.3.3).

Parking Lot Storage

Using parking lots for stormwater storage is a special case of surface storage. It is an economical option for meeting stormwater storage requirements in high density commercial and industrial developments. Planning of areas within a parking lot, which will accept ponding should be such that pedestrians are inconvenienced as little as possible.

Refer to local jurisdictional standards on the percentage of the parking lot that can be used as retention area and the allowable ponded depth. The maximum depth of ponded water within any parking lot location shall be 1 foot, unless separate approval is obtained from the local jurisdiction. Deeper ponding, if approved by the local jurisdiction, should be confined to remote areas of parking lots, whenever possible.

Drainage of parking lots can be accomplished by means of drywells (if permitted), curb openings, weirs, storm drains, orifices in walls, or gated outlets.

The minimum longitudinal slope permitted within parking lot storage facilities is 0.005 ft/ft, unless concrete valley gutters are provided. With concrete valley gutters, a minimum longitudinal slope of 0.002 ft/ft may be permitted.

Storage in Plazas, Courtyards and Common Areas

Landscaped common areas, pedestrian plazas and courtyards, which are typically provided in conjunction with high density residential, commercial and office developments, provide opportunities for multiple use as stormwater storage facilities. Such facilities should be designed to minimize public inconvenience, especially during frequent storm events. Public safety issues are also very important with this type of facility (see Section 9.4). Positive drainage to the outlet structures and trash/debris control must be provided so that the facility drains completely and efficiently.

9.3.3 Embankment Design Criteria

The use of embankments for stormwater storage is not recommended. Whenever possible, stormwater storage facilities should be constructed with the storage volume located entirely below the natural ground surface adjacent to the basin. However, in some instances this may not be possible, and embankments may be necessary to provide the required storage volume. Since the use of embankments may create a potential downstream flood hazard due to failure of the embankment, the following design considerations must be addressed in conjunction with their use.

State Dam Safety Requirements

The Arizona Department of Water Resources (ADWR), has legal jurisdiction over all dams (embankments) which exceed certain height and storage limits. Refer to the ADWR regulations and the local jurisdictional entity's standards for additional information regarding jurisdictional criteria.

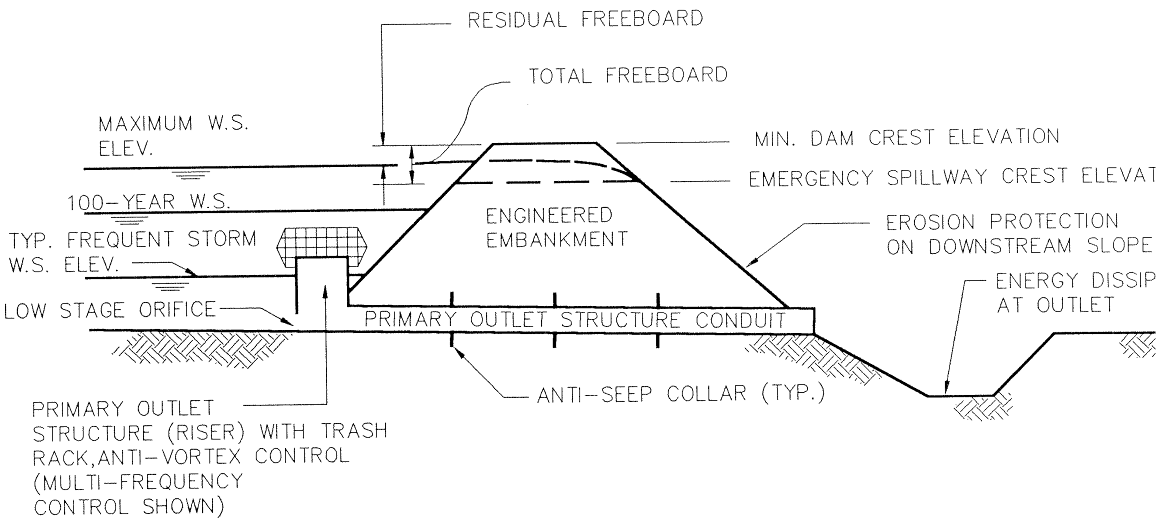
ADWR should be contacted regarding specific dam-safety requirements in conjunction with the design of any embankment, which might come under its jurisdiction. Those, which do fall within the jurisdiction of ADWR, shall comply with applicable ADWR design requirements.

Design of Embankments Not Regulated by ADWR

Embankments for stormwater storage facilities that are "non-jurisdictional" according to the state criteria will generally be classified by the state as small dams with an associated hazard potential. The hazard potential classification is related to the conditions downstream of the embankment. In the urban environment, the potential for probable loss of life and excessive damage to development downstream (existing or future) is great. Therefore, all embankments for stormwater storage facilities over 2.5 feet high will be considered as having high hazard potential.

The design reports, calculations, plans and specifications for construction of an embankment over 2.5 feet high for a detention or retention facility shall be prepared by, or under the direction of, a professional engineer registered under the laws of Arizona, and having proficiency in civil engineering as related to dam technology. The engineer should check with the appropriate jurisdiction for specific submittal requirements for embankment dam designs. Figure 9.8 shows a typical section of an embankment dam with common components applicable to a typical detention or retention facility.

FIGURE 9.8
TYPICAL SECTION OF A STORMWATER STORAGE FACILITY EMBANKMENT DAM



Geotechnical Engineering Studies

A geotechnical engineering study shall be conducted by a qualified civil or geotechnical Arizona Registered Professional Engineer prior to the design of any embankment with a hydraulic height over 2.5 feet high for the design event[1]. The study shall provide information on the embankment site conditions such as the embankment foundation and abutments (valley floor and sides), and shall provide evaluation of soil materials proposed for construction of the embankment. Samples obtained from borings and exploratory pits will be tested under laboratory conditions to evaluate more precisely the soil and rock classification, properties, strength, permeability, compatibility and other specialized tests pertinent to the specific project conditions. Analyses shall be conducted to evaluate conditions such as, but not limited to embankment slope stability, foundation stability, embankment and foundation seepage, internal and external erosion potential and embankment settlement. The results of these analyses are used to develop criteria for economic and safe design of embankments. These criteria include the types and zones of embankment fill materials based on using available borrow materials, upstream and downstream embankment slopes, and recommended measures for control of seepage.

Emergency Spillway

All embankments for stormwater storage facilities shall incorporate an emergency overflow spill-

1. Hydraulic height is defined herein as the vertical distance from the lowest natural ground at the toe of the slope to the water surface of the unattenuated 100 year inflow flowing over the spillway or emergency overflow area.

way for the safe overflow and routing of floodwaters under unusual circumstances[1]. Such conditions include the blockage or malfunction of the primary outlet structure or the occurrence of a storm event larger than that for which the facility was designed. Floodwaters that might otherwise overtop the embankment shall exit the facility via the emergency spillway and flow downstream out of the project property in the same manner and direction as would have occurred under pre-development or historic conditions.

The design of emergency spillways shall incorporate adequate erosion control and energy dissipation measures. Due to the high hazard potential of embankments, the minimum design standard for emergency spillways for embankments not regulated by ADWR shall be as indicated in Table 9.3. Spillway design standards for jurisdictional dams and total freeboard and residual freeboard dimensions shall conform to the applicable ADWR design requirements.

TABLE 9.3
NON-REGULATORY EMERGENCY SPILLWAY DESIGN CAPACITY REQUIREMENTS
(for an Embankment that is Not Regulated by ADWR)

Dam Height	Spillway Design Capacity
H < 6 ft.	unattenuated 100-year peak inflow
6 ft. < H < 25 ft.	½ Probable Maximum Flood

Primary Outlet Structure

The primary outlet structure is the main outlet structure by which stormwater is discharged from a stormwater storage facility. It is typically a closed conduit structure with an inlet specifically designed to control a single frequency storm or multiple events depending on the requirements of the specific jurisdiction. Special consideration must be given to seepage control along outlet conduits within an embankment dam, as discussed below.

Seepage

There are basically two categories of seepage considerations in embankment dam design. The primary concern is that seepage does not adversely affect the integrity or stability of the dam. The other category, water storage loss, is something the owner is usually most concerned about. This category relates to design of additional seepage control measures as required to maintain a permanent pool for reuse (water harvesting), or aesthetic or recreational purposes. Analyses shall be conducted in the following areas at a minimum, to address seepage.

Foundation - The flow of water through a pervious foundation produces seepage forces as a result of the friction between the percolating water and the soil medium. As the water percolates upward at the toe of the embankment, the seepage forces lift the soil by reducing its effective weight. In certain cases, this “piping” of the foundation soil can result in the failure of an embankment. A very common approach used is to excavate a cutoff trench into the foundation strata,

1. Generally, these are designed as a broad crested weir (see Chapter 8, Hydraulic Structures).

typically into an impervious layer. The trench is then carefully backfilled with relatively impervious material.

Embankment - Seepage through an embankment will occur, even with the tightest materials. On the upstream side of the dam, the embankment soils will reflect a water level equal to the impounded water level. As the water seeps through the dam, its pressure reduces and the water level drops. Design of the embankment should be such that seepage at the downstream toe occurs with no residual pressure. If the seepage were excessive, or were to emerge at an unplanned higher location, then erosion could begin at the discharge point and rapidly remove materials from within the embankment. Toe drains are typically designed to intercept the planned seepage flow, preventing nuisance conditions and enhancing slope stability.

Slope Stability - Combined with seepage analysis, slope stability analysis is critical. The forces pushing a mass of soil are analyzed with respect to the force resisting that movement. A related problem is slope stability during conditions of rapid change. A common concern is during a rapid drawdown, such as when operational problems with outlet works or seepage occur. With such operational problems, pressures in the soil may cause the slopes to fail during drawdown.

Piping along Boundary Conditions - Wherever there are boundary conditions, such as along an outlet conduit, spillway wall, cutoff trench or more subtle situations (such as layers of fill that have been rolled to a smooth hard surface), there is the potential of creating a more direct route for piping. The water flows at a higher erosive rate because it has a shorter, more efficient route. The technique that is often used along conduits and walls is to construct cutoff collars which extend laterally at intervals into the trench or embankment. When a much longer flow path is created, piping is minimized.

9.4 SAFETY

Public access and safety are inherent elements in the design of a detention or retention facility. These elements are of primary importance, particularly in the case of multiple-use facilities where public use is encouraged in areas subject to potential flooding. See Chapter 1 for a more thorough discussion on safety issues at stormwater facilities.

9.5 OPERATION AND MAINTENANCE

There are two major components to the maintenance of a stormwater storage facility. The first is to design a facility that is maintainable, and the second is the physical work required to keep the facility operating as designed and constructed. Maintenance of a stormwater storage facility falls into two categories; scheduled and unscheduled. Scheduled maintenance includes those activities such as mowing, pruning, and trash removal. These activities can be predicted and can be performed on a regular basis.

Unscheduled maintenance will involve the repair of facilities after storms and flooding. The frequency and scope of this type of maintenance cannot be predicted. Some examples of unscheduled maintenance are:

1. Embankment repair to keep erosion or rock riprap or earth fill sloughing from weakening the dam structure.
2. Debris removal during and following storms.
3. Inlet and outlet channel repairs to halt erosion and maintain hydraulic capacity.
4. Inlet and outlet structure repair so that the facility will function as intended.

It is important that adequate funding be provided for unscheduled maintenance such that repairs can be made immediately after flood or inundation damage occurs.

The following sections outline design considerations and recommendations which facilitate maintenance of stormwater storage facilities.

9.5.1 Access

Access roads for service and maintenance vehicles should be maintained to allow for equipment access to the facility, whenever needed. Access control gates should be provided if restricted access is required.

Design Recommendations

- Access ramps into the facility shall be graded at 10 percent or less. Turning radii shall be 50 feet or greater. Access ramps shall be designed for vehicle wheel capacities not less than 12,000 lbs.
- Service drives and gates shall be located in readily accessible, but inconspicuous, locations so as to not encourage unauthorized use.
- Design access control gates and adjacent areas shall be as secure as economically feasible. Initial expenditures for access control can save significant costs in future repairs.

9.5.2 Sediment Removal

Sediment will inevitably be deposited in the stormwater storage facility. Conditions will be worst during years when construction activity in the watershed is greatest.

Design Recommendations

- Provide stilling basins or fore-basin collection points where most sediment will be deposited (see Section 9.7.4).
- Provide controlled vehicular access into the facility for trucks and front-end loaders.

9.5.3 Repair of Eroded Slopes

Immediate repair of eroded slopes can minimize the ultimate cost for this activity. Small areas can be repaired by hand with on-site materials. Large eroded areas are much more difficult and expensive to correct because they may require larger equipment and placement of imported material.

Design Recommendations

- Keep side slopes flat to reduce likelihood of erosion.
- Provide vegetative or inert material cover on all slopes to minimize erosion.
- Adequately protect slopes subject to moving water or foot traffic. Make detailed evaluation of anticipated conditions and design protection accordingly. Use collector ditches for on-site drainage at the top of slopes.

9.5.4 Weed Control

Weed growth can adversely affect the use, appearance, and hydraulic characteristics of a basin. Therefore, weed growth shall be controlled.

Extensive use of herbicides in basins where the primary or secondary purpose is groundwater recharge is not acceptable.

Design Recommendations

- Plant or seed all non-paved areas in and around the basin to establish a vegetation cover. Weed infestation is much less likely in areas which have a cover of desirable plants than on disturbed or untreated areas.
- Design basins to allow all areas, including slopes, to be accessible by equipment such as flail mowers which can cut or remove weed growth.

9.5.5 Maintenance of Low Flow Channels and Drainage Structures

In-basin drainage structures and facilities must be maintained for proper operation. Design can influence maintenance requirements.

Design Recommendations

- Provide access to channels for front-end loaders and hauling equipment. Provide accessible areas, free of trees, to accommodate equipment movement.
- Provide energy dissipators to prevent damage to the channel or drainage structures during high inflow conditions.
- Design structures so that they will not collect debris, which could impact proper operation.

9.5.6 Landscape Maintenance

Some degree of plant and landscape maintenance will be required even when native, drought-tolerant species are planted.

Design Recommendations

- Select species with growth habits that minimize pruning and trimming or other maintenance requirements.
- Specify and use the largest plants within budgetary constraints. This can minimize potential damage during initial growth seasons.
- Space trees or plant masses for maintenance and equipment access.

9.5.7 Irrigation System Maintenance

Maintenance considerations of irrigation systems are critical, particularly when a permanent irrigation system is installed.

Design Recommendations

- Specify and use equipment that will continue to operate when “contaminated” with sand or other soil deposition. For example, large sprinkler head orifices, verses drip emitters, are less likely to clog when lake or well water is used for irrigation.
- Zone and layout system to avoid crossing channels where scour and erosion are likely to occur.
- If required, increase depth of bury or encase pipelines in concrete (particularly mainlines) that cross channels that are likely to be eroded.
- Install control equipment (other than remote control valves) in areas not subject to stormwater inundation or vandalism.

9.5.8 Sign, Wall, and Fence Maintenance

For the protection of the public, informational signs and fences must be maintained and kept in good repair.

Design Recommendations

- Use signs that are made of aluminum or other durable material that does not corrode or cannot be burned.
- Secure signs to posts or standards with tamper-proof fasteners. Use posts or standards that will not be damaged by anticipated flooding or vandalism.

- Locate fences away from areas likely to collect debris and act as dams to incoming water or water moving within the basin.

- Design fences, gates, walls, etc., to minimize damage or accidental opening during normal area use or by flooding.

- In non-critical areas, design fences with an open or “clear-space” at grade to allow shallow water and debris to flow or blow under them.

- Design fences, such as backstops, with break-away or swing-away panels so flow is not impeded through the basin.

9.6 MULTIPLE-USE CONCEPTS & AESTHETIC DESIGN GUIDELINES

The goal is to design stormwater storage facilities as amenities and, where possible, to incorporate multiple-use concepts. Flood control functions and other uses in stormwater storage facilities are generally compatible. Rationale for multiple-use facilities includes decreased facility costs and an increased community acceptance. Combining flood storage with recreation uses or other community facilities on a single site decreases total costs for land acquisition and site development. The development of stormwater storage facilities as parks or urban green space increases the acceptance by area residents and minimizes maintenance requirements and costs. If appropriately designed, use conflict is a minor concern. Stormwater storage facilities should be designed as a focal point to encourage proper usage and maintenance.

The planning and development of facilities for multiple-use requires cooperation between the engineer, a qualified landscape architect, intergovernmental agencies, community organizations, park and recreation departments, and risk management agencies.

Appropriate uses for stormwater storage facilities include active and passive recreation, urban green space, water amenities, water harvesting, and groundwater recharge. Use(s) in addition to flood control should address specific community needs and be clearly identified before the facility is designed.

9.6.1 Recreation Elements

Active Recreation

Active recreation includes a wide range of organized and unstructured activities that involve some type of physical movement. This type of recreational activity—both individual and group—generally requires larger areas (> 10 acres) than passive recreation uses. Because of their size, regional stormwater storage facilities can provide more opportunities for group sports with large space requirements. Field sports (soccer, football, baseball) require areas with standardized dimensions. Active recreation elements are more suitable in portions of stormwater storage facilities having lesser degrees of flood risk and frequency.

Passive Recreation

Passive recreation generally involves individuals or small groups and a minimal amount of physical activity. Typically, passive recreation does not require large open spaces, and is, therefore, appropriate for both large and small (< 10 acres) stormwater storage facilities. Passive recreational elements should be incorporated in portions of stormwater storage facilities having the greatest potential flood risk and frequency.

Design Considerations

Several design considerations are fundamental to incorporating recreation elements within stormwater storage facilities. Frequent inundation from low flows must be confined to areas that will characteristically require only limited maintenance. Contouring within facilities is recommended to create internal elevation variations (or tiers) that have differing frequencies and depths of inundation and differing flood risk. Suggested tiers may include: Lowest lying areas – semi-natural riparian zones, wetlands, habitat areas.

- Lower elevated tiers – passive recreation zones, picnic areas, open fields.
- Intermediate elevated tiers –ballfields, soccer fields.
- Upper elevated tiers – court games, play areas, tot lots, pit games, parking facilities.
- Areas elevated above 100-year flood level – restrooms, habitable structures, and swimming pools.

In addition, internal drainage within stormwater detention facilities should provide for positive flow across elevated tiers and to the basin floor to prevent nuisance-standing water within recreation areas. Internal slopes should be flat enough to allow for mowing of turf areas and to allow for other routine recreation related maintenance activities. Hydraulic design components (inflow structures, outflow structures, spillways, sediment basins, etc.) should be included as needed. Figure 9.9 provides a generic site plan for recreation elements that might be incorporated into a stormwater storage facility. Figure 9.10 depicts a typical cross section of the site plan.

FIGURE 9.9
STORMWATER STORAGE FACILITY RECREATION ELEMENTS

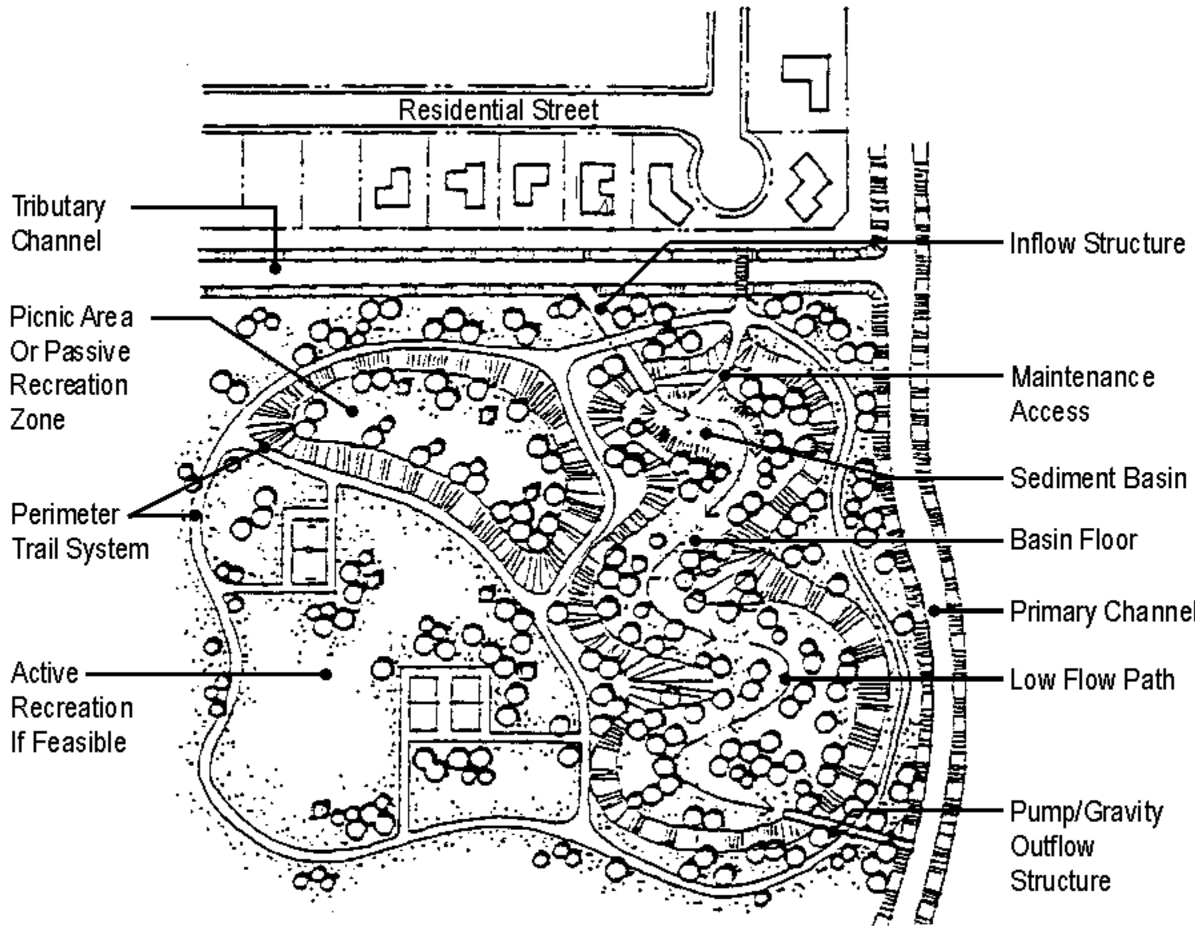
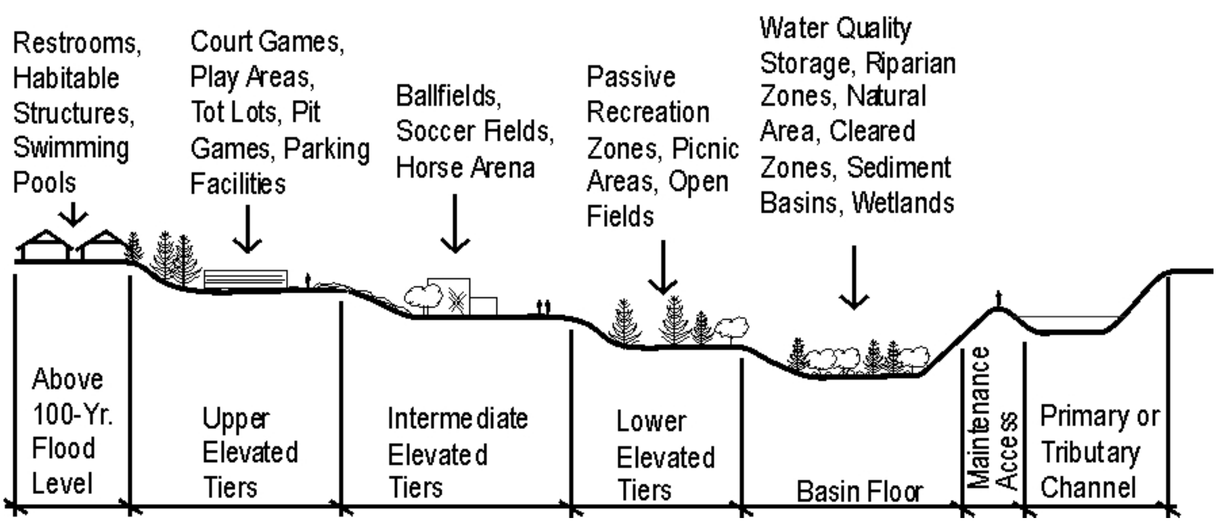


FIGURE 9.10
TYPICAL CROSS SECTION OF STORMWATER STORAGE FACILITY W/RECREATION ELEMENTS



9.6.2 Stormwater Storage Facilities as Water Amenities

Facilities that incorporate a permanent pool can provide physical and psychological relief from the hot desert environment. The use of a permanent pool for stormwater storage facilities is limited strictly to a visual amenity because body contact activities, such as swimming or wading, are specifically excluded.

9.6.3 Urban Green Space

Urban green space provides a visual resource within the community. As urbanization continues, the value of green space typically increases. Green space provides visual breaks from the urban environment, acts as a filter to clean the air and can reduce erosion from wind and rain. Landscape materials in a stormwater storage facility should respond to the recessed nature of the land form, the scale of the facility and the occurrence of frequent flooding.

The use of native and non-native, drought-tolerant species for landscape planting is highly recommended. The following basic zones should be considered in the landscape design for a stormwater storage facility.

Channels - These are areas where there will be flowing water. Planting in these areas should be limited to grasses, groundcovers and low growing shrubs, with preference given to vegetation with flexible branching and resilient growth habits.

Basin Areas - There may be inundation and standing water in basin areas at some time during the year. Choice of plant materials should reflect these conditions. Trees, shrubs and grasses can be planted judiciously in these zones.

Elevated Areas - These areas may be occasionally inundated. The choice of plant material will depend on the use assigned to the area. Trees, shrubs and grasses can be planted and more easily maintained in areas of higher ground elevation.

9.6.4 Water Harvesting for Reuse or Recharge

A basic water harvesting system consists of three components: collection, storage and dispersion. Since stormwater storage facilities will already be designed to collect and store runoff, some simple additions may allow harvesting the water for reuse.

All applicable requirements of the Health Department and the Arizona Department of Water Resources must be met in addition to the normal review requirements.

When reusing stormwater, such as on-site landscape irrigation, the facility must be lined by an impermeable membrane or by treating the soils to increase impermeability with native or imported clay or other measures. The local jurisdiction must be contacted regarding the acceptability of soil treatment measures in terms of the effect on water quality. Grading of the surrounding site should optimize runoff to the storage facility. An evaporation control mechanism may be appropriate for a surface storage system. Distribution of water is typically achieved by pumping from the pond for irrigation.

A facility may be designed specifically to augment the groundwater aquifer. The facility should be designed to maximize the surface contact area between the stored water and the soil, thus maximizing the potential for water to percolate through the subsurface to the groundwater table. Potential siltation problems must be addressed by providing a settling basin at the inlet or by other suitable measures.

Runoff water stored for recharge or reuse purposes should not occupy volume needed for stormwater storage within the storage facility. Adequate volume for stormwater storage must be provided at all times, in addition to the volume provided for harvesting water.

9.7 WATER QUALITY

9.7.1 Introduction

Urban runoff is distinguished from undeveloped area runoff in two principal ways: it typically occurs at greater discharge rates and volumes, and it contains varying but commonly higher concentrations of toxic substances, bacteria, and dissolved organic matter. Stormwater storage facilities can play a significant role in mitigating the pollution problems associated with urban runoff.

9.7.2 Major Pollutants and Their Sources

Major pollutants associated with urban runoff include the following:

Sediment - Construction activities associated with urbanization and agricultural practices often result in erosion and sedimentation.

Suspended Materials - Particulate matter and floating material, such as oils, scum and sediment, are included as suspended material. Suspended solid concentration in stormwater may be 2 to 3 times that found in domestic sewage.

Oxygen Demanding Materials - These include degradable organic matter and certain nitrogen compounds that consume the available dissolved oxygen as they degrade. The biochemical oxygen demand of stormwater runoff is usually in the 20 to 30 mg/l range, almost the same range as sewage effluent after secondary treatment.

Pathogenic Bacteria and Viruses - These include coliform, fecal coliform, and fecal streptococci, the same pathogenic bacteria and viruses found in domestic sewage.

Toxic Substances - These include heavy metals and a full range of EPA designated pollutants. The EPA list contains approximately 100 primarily organic substances, such as TCE.

Studies show that the areas contributing the greatest amounts of pollution are those with highly erodible surface conditions, such as plowed land or construction sites, or those areas characterized by highly impermeable surfaces, such as shopping malls, industrial areas and large housing complexes. Runoff from vehicular right-of-way (which accounts for over 20 percent of some urban lands), will contain hydrocarbons, other organics, and a diminishing—but still significant—amount of lead. Fertilizers and pesticides are transported by runoff from residential and agricultural areas.

9.7.3 Role of Stormwater Storage Facilities in Water Quality Control

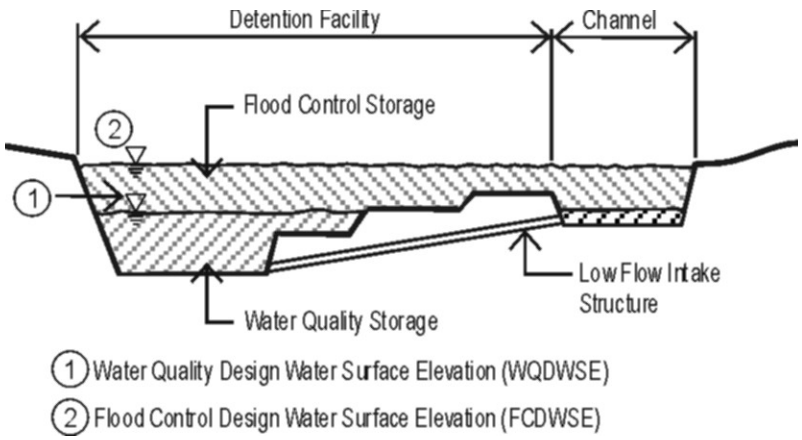
Most pollutants of concern have a high affinity for suspended solids in runoff and for soil particles. Thus, the most logical way to achieve pollutant removal is through sedimentation and infiltration. Consequently, stormwater storage facility design for water quality control should maximize settling to the extent possible. This consideration may alter typical design features. In general, quiescent conditions and infiltration should be maximized while short-circuiting should be minimized. Design techniques that will accomplish these objectives are:

- Using long, narrow basin configurations that is length to width ratios of 2:1 to 3:1, with the length measured along a line between the inlet and outlet.
- Installing inlet and outlet structures at extreme ends of the basin.
- Using baffles or flow retarders.
- Constructing ponds with active "wet" storage and inactive "flood" storage. An example of a dual-purpose detention facility is illustrated in Figure 9.11.
- Using riser outflow structures instead of ground level pipes to maintain a slow-draining pool encouraging infiltration. Here, undershot weirs or inverted siphons should be considered to keep floating pollutants from conveyance downstream.
- Developing a grass cover for the basin floor.
- Using underground tile drains for outlet discharge to provide soil filtration of the runoff.

Using wet rather than dry ponds will generally improve quiescent conditions, maximize infiltration, and provide a degree of biological treatment.

The function of water quality storage elements within a stormwater detention facility is to provide for settlement of pollutants, thereby improving downstream water quality. Periodic removal and proper disposal of accumulated sediments will be needed to prevent their accumulation from reaching toxic levels. A program for occasional reworking and/or removal of accumulated sediments within stormwater detention facilities is essential.

FIGURE 9.11
STORAGE COMPONENTS OF A DUAL PURPOSE STORMWATER STORAGE FACILITY

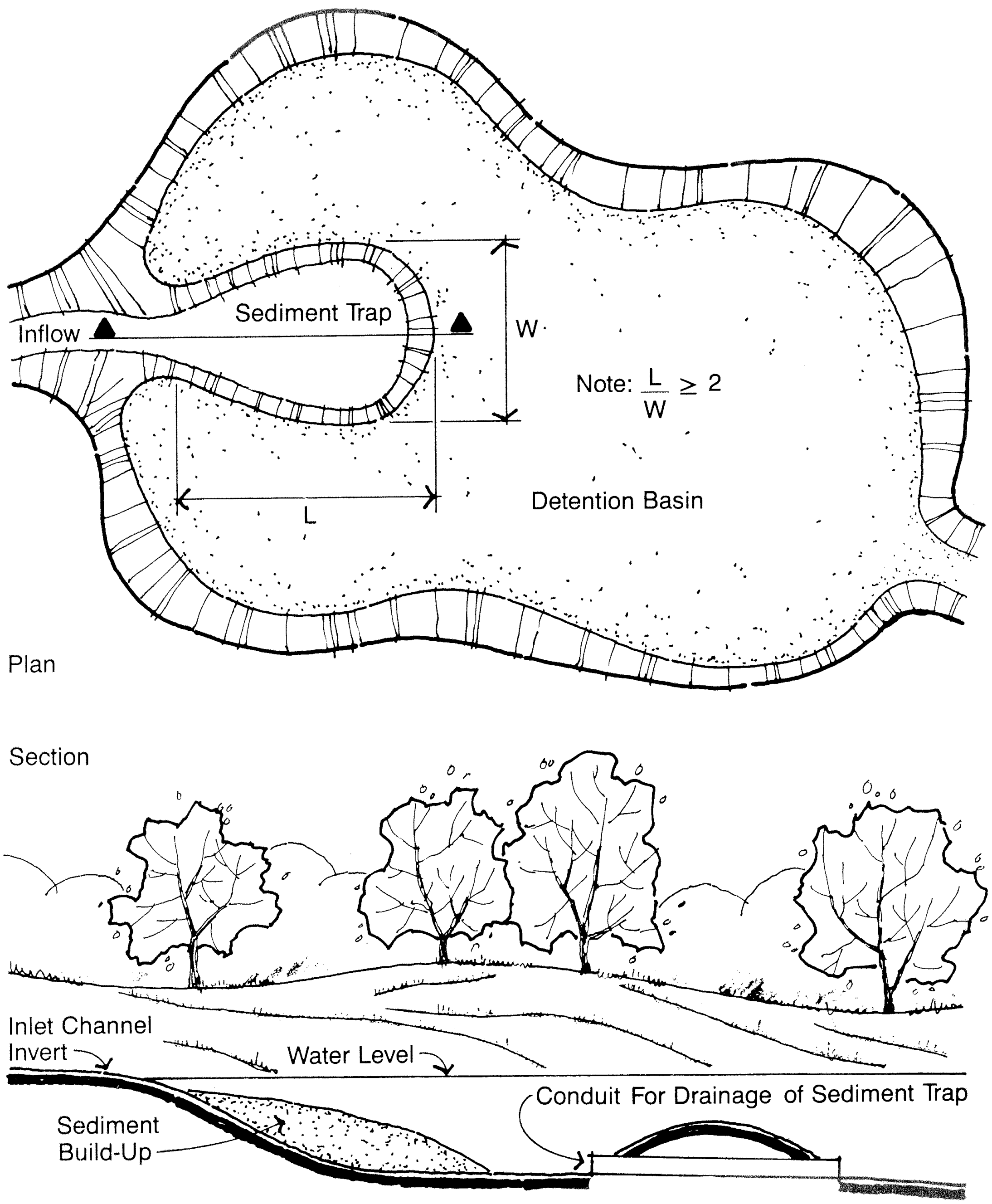


9.7.4 Method for Control of Sedimentation

Sediment removal within a stormwater storage facility may be facilitated by the use of a "sediment trap" at the inlet, which will concentrate the majority of the incoming sediment bed load to a small portion of the facility. Sediment traps should be provided in conjunction with all stormwater storage facilities, which are intended as multi-use facilities and serve larger watersheds. Figure 9.12 is a conceptual sketch of a typical basin sediment trap. The following list provides guidelines for the design of efficient sediment traps.

1. An additional sedimentation volume should be provided within the sediment trap at an elevation below the invert of the inflow channel.
2. The length/width ratio of the sediment trap should be a minimum of 2:1, with the length measured along a line between the inlet and outlet.
3. The basin shape should be wedge-shaped, with the narrow end located at the inlet to the basin (see Figure 9.12).
4. Provisions for total drainage and accumulated sediment removal of the sediment trap must be provided. Maintenance access should also be provided and designed to accommodate heavy trucks and other equipment necessary for removal of accumulated sediment.

FIGURE 9.12
SEDIMENT TRAP CONCEPT
([Pima County Department of Transportation and Flood Control District](), 1986)



9.8 REFERENCES

9.8.1 Cited in Text

McGuckin Drilling, Inc., 1987, The MaxWell IV, Product Catalogue 187-1, Phoenix, Arizona.

Pima County Department of Transportation and Flood Control District, Hebel, Susan J. and Donald K. McGann, 1986, Guidelines for the Development of Regional Multiple-Use Detention/Retention Basins in Pima County, Arizona.

Stahre, P., Urbonas, B., 1990, Storm-Water Detention for Drainage, Water Quality, and CSO Management, Prentice-Hall, Inc.

U.S. Army Corps of Engineers (USACE), 1990, HEC-1 Flood Hydrograph Package, User's Manual. [HEC WEB Site)

U.S. Department of Transportation (USDOT), 1980, Federal Highway Administration, Underground Disposal of Stormwater Runoff Design Guidelines Manual, J.B. Hannon. [USDOT Hydraulics WEB Site]

U.S. Environmental Protection Agency, 1980, Design Manual - Onsite Wastewater Treatment and Disposal Systems, pgs 41-44.

WRC Engineering, Inc., 1985, Storm Drainage Design and Technical Criteria, prepared for the Urban Drainage and Flood Control District, Denver, Colorado, Arapahoe County (Colorado).

9.8.2 Relevant to Chapter

American Society of Civil Engineers, 1982, Proceedings of the Conference on Stormwater Detention Facilities, William DeGroot, B.R. Urbonas, M, Glidden, editors.

Arizona Department of Water Resources, 1985, Design Manual for Engineering Analysis of Fluvial Systems.

Chow, V.T., Maidment, D.R. and Mays, L.W., 1988, Applied Hydrology, McGraw Hill.

Clark County Nevada Regional Flood Control District, 1986, Comprehensive Flood Control District Plan

Heber, W.C., et al, 1982, Stormwater Management Model, Users Manual, Version III, U.S. Environmental Protection Agency.

Mays, Larry W., 1999, Hydraulic Design Handbook, McGraw-Hill.

Stantec Consulting Inc., 1994, Manual of Standards for Design of Joint-Use Stormwater Detention Facilities, Volume 4, City/County Drainage Manual, Prepared for County of Sacramento, Department of Public Works, Water Resource Division.

State of California, Department of Conservation, 1978, Erosion and Sediment Control Handbook.

Transportation Research Board, 1980, Synthesis No. 70, Design of Sedimentation Basins.

Urban Land Institute, 1975, Residential Stormwater Management: Objectives, Principles, and Design Considerations, Washington, D.C.

Urbonas, B.R., 1985, Stormwater Detention Outlet Control Structures, American Society of Civil Engineers.

U.S. Department of Interior, Bureau of Reclamation, 1987, Design of Small Dams, Third Edition.

U.S. Department of Agriculture, 1978, Soil Conservation Service, Stormwater Management Pond Design Manual, Maryland Association of Soil Conservation Districts.

——, 1972, SCS National Engineers Handbook, Section 4, Hydrology.

Page intentionally left blank

10 PUMP STATIONS

TABLE OF CONTENTS

10.1 SYMBOLS

The following symbols are used in equations in this chapter:

h = pipe head loss, ft

L = pipe length, ft

D = pipe diameter, ft

V = flow velocity, ft/sec

n = Manning’s roughness coefficient

H_L = minor loss, ft

K = minor loss coefficient

g	=	acceleration due to gravity, 32.2 ft/sec^2
t	=	pump cycling time, minutes
V_w	=	wet well volume, gallons
P	=	pumping rate, gallons/minute
Q	=	inflow rate, gallons/minute
t_m	=	minimum cycling time, minutes

10.2 INTRODUCTION

The consideration of a pumping station as an alternative will occur when an area to be drained is so low that construction of a gravity drain is not feasible. Pumping stations are used to drain depressed sections of urban roadways and paved areas and for discharge of water from retention basins when other means of gravity drainage are not available.

The design of pumping stations involves many different disciplines and the design approach is dependent on the size and purpose of the facility, and the consequences of system failure. This chapter provides general requirements and guidelines for planning and analysis of pumping facilities. For a more rigorous discussion of the design of stormwater pump stations, refer to the *Highway Stormwater Pump Station Design, HEC-24* (USDOT, 2001).

10.3 DESIGN CRITERIA

Gravity drainage of retention basins and other low lying areas is preferred; only under special circumstances with prior City/County staff approval should pump stations be used.

10.3.1 Design Frequency

Pump stations must provide sufficient capacity to discharge the volume of storm runoff generated by the design storm.

10.3.2 Drainage of Basins and Ponding Areas

Retention basins shall be drained within 36 hours following the storm.

The drain time for ponding areas may be required to be less than 36 hours by the City/County.

Retention basin pump stations are required for publicly maintained basins where the depth of water retained exceeds 1 foot, a gravity flow bleedoff system is not possible, and the use of drywells is not a viable option.

10.3.3 Bleedoff Lines

The minimum pipe diameter for pressure bleedoff lines is 8 inches, and 24 inches for gravity lines. A restrictor plate may be used to limit maximum rate of flow.

All bleedoff lines shall have a method to shut-off flows.

A control valve or gate shall be installed such that they are readily available for inspection by City/County forces.

Water cannot be discharged onto a City street or street gutter or alley.

The following, listed in order of preference, are methods of discharging from pump station facilities:

- Discharge to an open channel either natural or man-made.
- Discharge directly to a nearby storm drain system with a maximum discharge limited to the available capacity of the system as approved by the City/County.
- Discharge to the surface of a storm drain system if pumped water can be discharged directly into a catch basin or other inlet.

10.3.4 Pump Station Design Requirements

If a pump is to be used, the rate at which the pump discharges must not overtax downstream drainage systems.

Stormwater pump stations are classified by size: smaller or larger than 60 cfs. The larger stations will have additional requirements such as flow recording equipment.

Pump stations shall be located so that the pumps are accessible when the basin or sump is full. Pumping facilities (excluding components whose design requires submersion) will be set at an elevation at or above the anticipated level of the design storm event, considering that a total power failure may occur.

Pumps shall be capable of handling solids up to a maximum of 3 inches. Consideration for handling smaller solids can be made for pumping facilities that serve storage facilities.

An inflow hydrograph for the design of the storage reservoir shall be determined in accordance with the procedures in the *Drainage Design Manual for Maricopa County, Hydrology*.

Plugging factors will be used on inlets of pipe systems that are tributary to pump stations. Fifty percent of the area of the trashrack shall be assumed plugged.

Maximum use of surface storage, instead of underground storage, is desirable for minimizing storage costs. Volumes of cross pipes, inlets, manholes or catch basins should not be considered as part of the available storage reservoir volume.

The engineer shall provide the following design information:

- Headloss calculations for the entire system, including maximum and minimum Total Dynamic Head (TDH) and flow rate.
- Net positive suction head (NPSH) and pump level settings for on, off and alarm positions.

- Inflow and outflow hydrographs and accumulated inflow and outflow curves (mass flow curves). The use of HEC-1 is not appropriate for the design of pumping stations. A real-time procedure which routes the design inflow hydrograph using pump on and off elevations and actual pump performance curves must be used.

- Specifications for the model and type of pump(s) proposed including pump curves (single pump and parallel operation). Overloading the pump anywhere on the pump curve is not permitted.

10.3.5 Pump Station Facility Requirements

The collection system shall discharge into a separate sump that screens the water before entering the pump sump.

The wet well shall be a minimum of 6 feet by 10 feet inside dimensions and shall be provided with a means to drain it when the pump is not running.

A pump shall be provided with an automatic control switch with a vertical float mechanism. Larger stations shall provide communications equipment to permit transmission of failure signals to designated reporting locations and to allow remote operation of the pumps.

A potable water supply with backflow prevention and hose bibs shall be provided to aid in removal of silt and trash.

A ventilation system will provide intermittent ventilation of wet-wells.

A redundant pumping system may be required, particularly at small installations.

The site layout should consider adequate access for maintenance vehicles to refill fuel tanks, remove pumps and generators, and provide adequate aesthetics and mitigation of on-site noise.

10.4 DISCUSSION

10.4.1 Pump Selection Study

Information required for the pump selection study includes proposed station capacity and pertinent water surface elevations for maximum, average, and minimum flow conditions, points of discharge, proposed station locations and their soil conditions, proposed piping system and pertinent information on terrain, location of utilities, and the proposed method of operation.

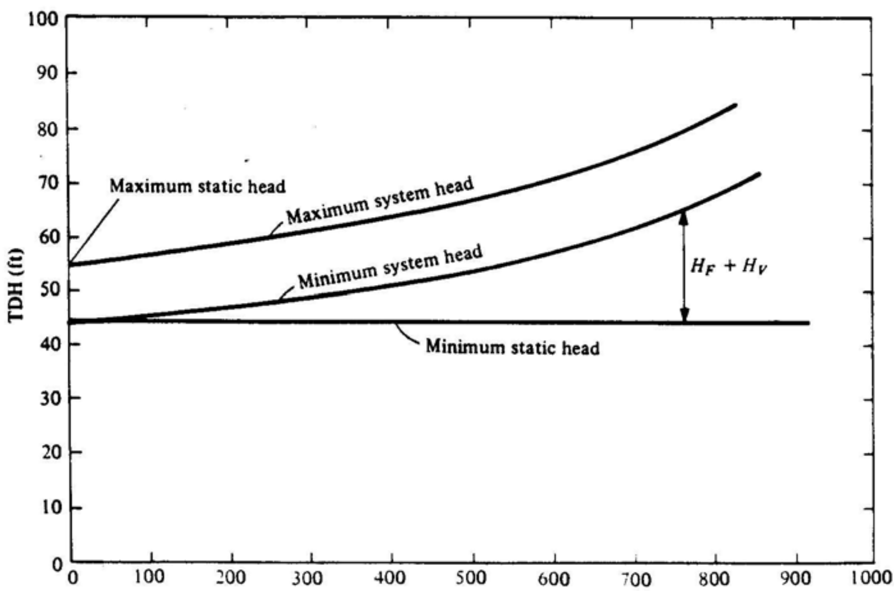
Possible requirements for future expansion must be considered in the planning. Expansion may be accomplished by modification of the existing units (such as increasing the pump impeller diameter), by adding pumps or by replacement of the original pumps. The last alternative is the least desirable one as it would entail considerable alteration in the facility. Allowances for any of the alternatives should be made in the original design, as should the provision of a large enough motor if the pump impeller diameter is to be increased at a later time or the provision of space and foundation size for an additional pump along with proper sizing of piping.

10.4.2 System Analysis

A system analysis for a pumping station will aid in the selection of the best pumping units. For the analysis, system head curves for the proposed system are calculated for critical conditions and combined with the characteristic curves of pumps that are being considered for the installation. A preliminary plan for the piping system is needed for this purpose.

A system head curve is a plot of total system head against flow rate where the total system head at any flow rate is the head to be supplied by the pump to produce the given flow rate at the discharge point of the piping system. The total system head is the static head plus the head losses in the piping system which include the pipe friction losses and the minor losses at entrance and exit and at fittings such as valves, bends, expansions, and contractions. See Figure 10.1.

FIGURE 10.1
SYSTEM-HEAD CURVES FOR A FLUCTUATING STATIC PUMPING HEAD
(Clark et al., 1977)



Pipe friction losses are calculated by the Manning's equation. For this calculation the equation may be expressed as follows:

$$h = 2.87 n^2 \frac{LV^2}{D^{4/3}} \tag{10.1}$$

where h is the head loss (ft), L is the pipe length (ft), D is the pipe diameter (ft), V is the flow velocity (ft/sec) and n is the Manning's roughness coefficient. Commonly used values of n range from 0.010 to 0.041. Wherever possible, local experience with different pipe materials should be used in choosing a value of n.

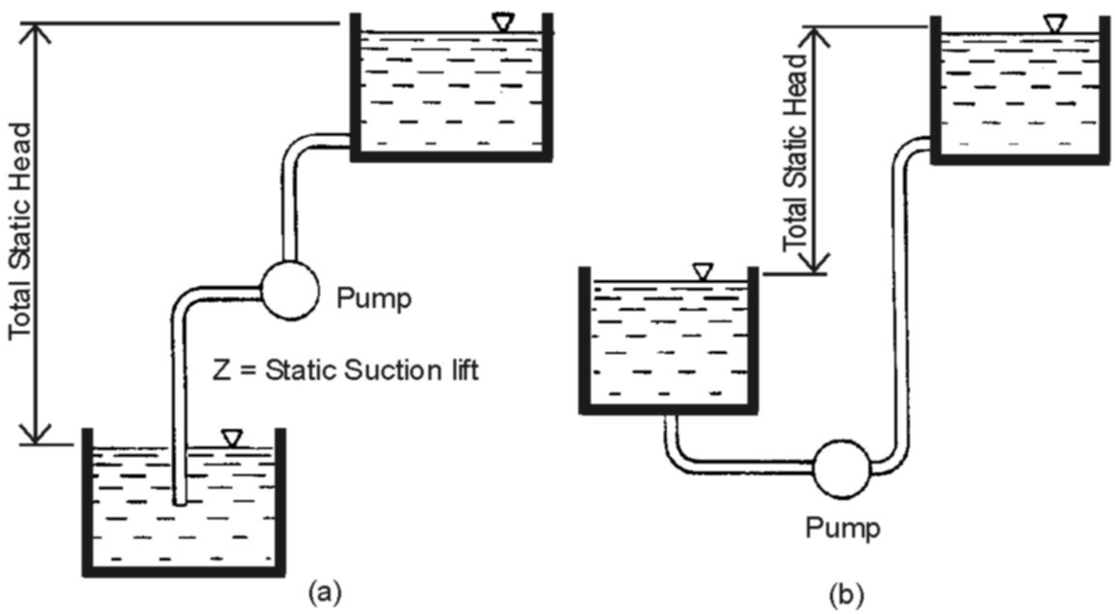
Minor losses are most often expressed as:

$$H_L = \frac{KV^2}{2g} \tag{10.2}$$

where H_L is the head loss (ft), K is a coefficient for the particular fitting and $V^2/2g$ is the velocity head (ft) in the pipe. Alternatively, minor losses may be expressed as "equivalent length of pipe."

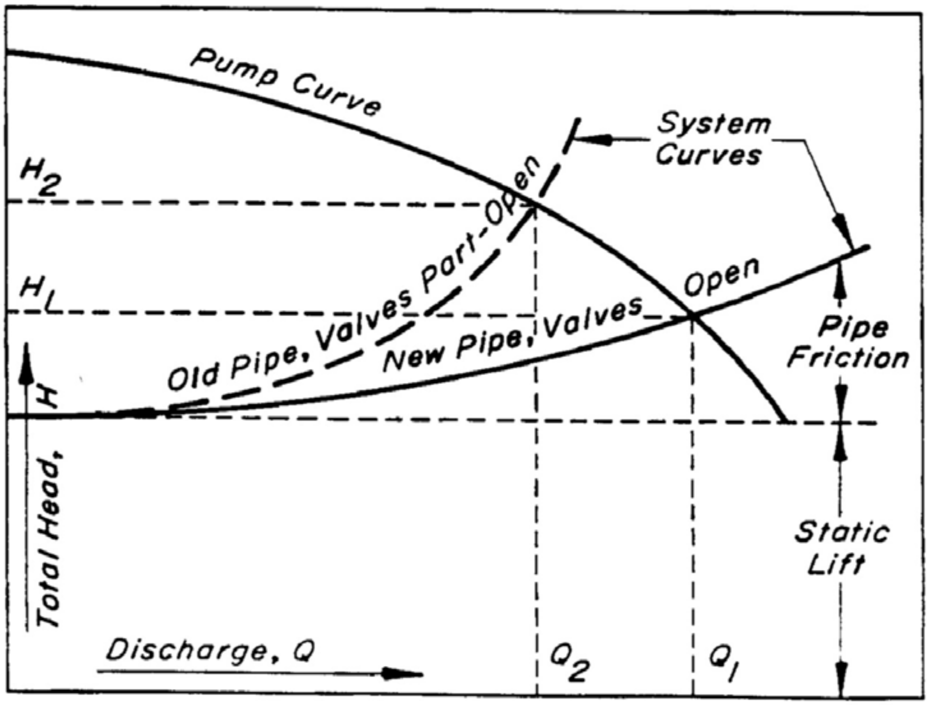
Static head is the difference in elevation between the water surface in the wet well and that at the discharge point. Storm water pumping systems are usually designed to discharge into a channel, conduit or receiving body at atmospheric pressure; in this case, the appropriate elevation to use is the centerline elevation of the effluent pipe (see Figure 10.2). Since the level in the wet well may vary, system head curves for maximum and minimum static heads are usually plotted.

FIGURE 10.2
TOTAL STATIC HEAD
(a) Intake below pump centerline; (b) intake above pump centerline
(Clark et al., 1977)



The analysis proceeds with the plotting of the head-discharge curve for the selected pump(s). Intersection of the pump curve with a system head curve represents an operating point of the pump under the assumed conditions of static head and head loss. Efficiency and input power curve should be also plotted. The operating point (see Figure 10.3) should be at or near to the best efficiency point for the selected pump(s). It is not only desirable that the pump operate efficiently from the standpoint of power consumption, but also it has been found that pumps operating at the best efficiency point do so more quietly and with less wear than otherwise, resulting in lower maintenance costs.

FIGURE 10.3
PUMP OPERATING POINT IN A GIVEN PIPING SYSTEM
(ASCE, 1975)



10.4.3 Pump Types

Stormwater pumps may be either vertical, wet-pit or dry-pit, submersible or horizontal, depending upon the individual conditions of each situation. Dry-pit pumps are usually volute pumps, while vertical wet-pit pumps generally have diffusion vanes to save space. Some advantages of the wet-pit type are no priming requirement, relatively high available NPSH and economy of space. Dry-pit pumps allow easier accessibility and require less maintenance. In small installations submersible pumps may be used; in such a pump, the motor is close-coupled to the pump and is submerged.

Stormwater pumps may be radial, axial or mixed flow types. In general, radial flow pumps are adapted to high-head, low-flow situations; and axial flow pumps are used for large flows at low heads. The specific speed can serve as a guide to choosing the proper pump type.

10.4.4 Pump Selection

It is usual in stormwater pumping stations to require at least two pumps for reliability, with either one capable of handling the station capacity. More generally, where more than one pump is used, the requirement is made that the station capacity can be met when the largest pump is out of service.

Pump speeds used in stormwater pumping are usually limited to 1,750 rpm to minimize maintenance problems and provide greater reliability. Generally, constant speed control is used.

10.4.5 Piping and Valves

Suction inlets for dry pit pumps should be designed to avoid the formation of vortices which cause air to be drawn into the pump. Flared end sections are preferable.

Vertical wet pit pumps are very sensitive to intake conditions within the wet well. Follow the recommendations of the pump manufacturer.

Suction piping should be kept as short as possible to minimize head losses and should not contain any unnecessary fittings. The line should contain a flanged gate valve and an eccentric flanged reducer with the flat side up as the suction pipe should be one or two sizes larger than the pump suction nozzle. Velocities in the suction pipe should not exceed 5 ft/sec with lower velocities desirable.

Discharge piping should be based upon velocities not exceeding 8 ft/sec with lower values preferred. The pipe size should be at least one size larger than the pump discharge nozzle with the transition made by a concentric increaser. The increaser should be followed by a check valve in a horizontal section of pipe and a gate valve. If the force main is longer than 1 mile, the possible occurrence of water hammer should be analyzed and appropriate surge control measures provided.

10.4.6 Location

The following points related to the location of a pumping station should be taken into account:

- Accessibility for maintenance and removal of equipment for repair.
- Parking space for personnel and emergency equipment such as generators.
- Local drainage and protection from flooding.
- Availability of suitable electrical power supply.
- Soil conditions at proposed site and information on groundwater levels, particularly maximum heights.
- Safety and environmental requirements of local codes, including those concerning exhaust of internal combustion engines, if used.

10.4.7 Wet Well Design

The principal purposes of the wet well are to provide a sump for the pump suction intakes, and to supply storage to minimize on-off cycling of the pumps. For small stations, a common requirement is to have a volume (in gallons) between on and off levels in the well of 2.5 times the pump flow rate in gallons per minute. This requirement is based upon the criterion that the minimum on-off cycle for a pump should be 10 minutes and for a uniform pumping rate and uniform inflow, the minimum cycling time occurs when the pumping rate is twice the inflow. The cycling time for a single pump is given by:

$$t = \frac{V_w}{P - Q} + \frac{V_w}{Q} \tag{10.3}$$

where t is the cycling time in minutes, P is the pumping rate in gallons per minute, Q is the inflow in gallons per minute and V is the wet well volume in gallons, between the levels of the on and off controls. Then, the required wet well volume for minimum cycling time, t_m is:

$$V_w = \frac{t_m P}{4} \tag{10.4}$$

or for a 10 minute minimum,

$$V_w = 2.5P \tag{10.5}$$

If two pumps are used alternately, the cycling time is increased by a factor of two.

Other details of wet well design which should be considered are the following:

- The wet well floor design should minimize deposition of solids.
- Access to the wet well should be from the outside and means of ventilation should be provided.
- Divided wet wells should be provided for larger pumping stations so that one part may be used while the other is shut down.
- A high water alarm should be provided.
- Screening may be required if the stormwater is expected to be carrying debris which would damage the pumps.

10.4.8 Pump Room or Drywell

Besides space for pumps, motors and control equipment, the pump room must provide space and facilities so that maintenance work on all equipment can be done effectively and safely. One rule for spacing between pumps is at least 3 feet from each outside pump to the wall and 4 feet between each pump discharge casing. The pump room should be adequately and safely lighted and ventilated. Floor drainage should be provided. Openings for removing equipment should provide ample space for so doing.

10.4.9 Pump Settings

The pump setting is the location in elevation of the impeller or propeller centerline with respect to the water level. The setting needed is related to the following requirements:

- Provision of sufficient available NPSH at extreme operating conditions
- Sufficient submergence of the suction inlet so that air is not drawn into the pump by vortexes
- Priming or the need for a centrifugal pump to be filled with water when started

For dry pit pumps it is recommended that the high point of the casing be set below the minimum water level in the wet well to ensure proper priming.

10.4.10 Pump Controls

Stormwater pump controls include elements to sense the well level, on-off switches to operate one or more constant speed pumps, step or stepless variable speed control units and mechanical or electrical alternators to change the order of operation of two or more pumps.

Sensing elements for maximum and minimum water levels should be separated by at least 3 feet, and individual controls should be at least 12 inches apart.

Besides the on-off controls, a high water alarm and a low water alarm and cut-out switch should be installed. The maximum water level should be such that undesirable surcharging of the incoming sewers is prevented and the high water alarm should be 6 inches above this level. The low water alarm should be 12 inches below the minimum wet well level.

10.4.11 Electrical Power Supply

The power authority should be consulted to determine what voltage and type of electrical power is available at the station site. For small stations 240 or 480 volt three phase is usually used. For large motors (over 400 hp), higher voltages may be preferred, if available, for economy.

For reliability in large pumping stations, two independent incoming power lines should be available at the pumping station and provisions for supplying emergency generator power to the pumps should be considered.

10.4.12 Emergency Power Supply

In situations where electrical failure would result in prolonged inundation and cause undue hardship, such as arterial roadway dip sections under bridges, an emergency back up generator is necessary. Siting of the generator above flood levels or proper flood proofing is mandatory. This applies to fuel storage as well. Proper exhaust ventilation must be planned. The need for emergency power supply will be considered on a case by case basis and will be dependent upon the threat to public health and welfare.

10.5 AIDS

TABLE 10.1
DESIGN CHECKLIST FOR PUMP STATIONS

General
Initial Data
Contributing Drainage Basin
Location of Outfall
Capacity of Outfall
Probable Growth in the Contributing Basin
Inflow Hydrographs
Possible Components
Source of Power (primary and emergency)
Pumps
Intakes and Catch Basins
Controls
Storage
Debris Handling
Potable Water Supply
Testing
Hoisting Equipment
Ventilation
Control of Hazardous Materials
Hydrology
Economic and Alternative Analysis
Designation of Significantly Different Concepts
Hydrologic and Hydraulic Detailing of Alternatives
Cost Evaluation
Extreme Event Evaluation of Components and Alternatives
Environmental Considerations
Documentation and Comprehensive Evaluation

TABLE 10.1
DESIGN CHECKLIST FOR PUMP STATIONS (CONTINUED)

Hydraulic Analysis
Mass Curve Routing
Outflow Hydrograph
Pump Characteristics
Pipe Losses
Miscellaneous Losses
Sediment Transport
Additional Considerations

10.6 REFERENCES

American Society of Civil Engineers (ASCE), 1975, *Pipeline Design For Water and Wastewater.*

Clark, John W., Viessman, Warren, and Hammer, Mark J., 1977, *Water Supply and Pollution Control*, Third Edition, Harper & Row.

U.S. Department of Transportation (USDOT), Federal Highway Administration, 2001, *Highway Stormwater Pump Station Design, Hydraulic Engineering Circular No. 24*, Publication No. FHWA-NHI-01-007, Washington, DC. [USDOT Hydraulics WEB Site]

11 SEDIMENTATION

TABLE OF CONTENTS

11.1 DEFINITIONS

Aggradation	General and progressive buildup of the longitudinal profile of a channel bed due to sediment deposition.
Antidune	Alluvial channel bed form in upper regime flow that are often in trains. Antidunes are often called standing waves and exhibit surface waves that are in phase with the antidunes. The Manning's n for antidunes ranges from 0.01 to 0.02 and the sediment concentration ranges from 2,000 to 5,000 mg/liter.
Avulsion	A rapid change in the course or position of a stream channel that usually occurs when a stream breaks through its banks. It is usually associated with a flood or a catastrophic event.
Bar	A large volume of sediment deposited over a relatively large area on the channel bed. Bars are plane bed forms having lengths of the same magnitude as the channel width or greater, and heights comparable to the flow mean depth. It is not permanently vegetated.
Bed load	The sediment load transported close to the bed where particles move intermittently by rolling, sliding, or jumping.
Bed material	Material found in and on the bed of a stream (may be transported as bed load or in suspension).
Bed material load (discharge)	Sediment transport load or sediment transport discharge for those sediment particles sizes that are readily apparent on the surface of the streambed. It can be divided into suspended bed material load and bed load.
Bed or streambed	The bottom of a watercourse.
Bulk specific weight	Specific weight for a sediment deposit. It is defined as the dry weight of the sediment deposit divided by its bulk volume. It should be much smaller than the sediment particle specific weight.
Channel Migration	Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank.
Chute and Pool	Upper flow regime bed forms at very steep slopes with high velocities and high sediment discharge. Large elongated mounds of sediment create chutes where the flow is rapid and followed by a hydraulic jump and a pool. This bed form consists of chutes connected by pools.
Degradation (bed)	A general and progressive (long-term) lowering of the channel bed due to erosion, over a relatively long channel length.

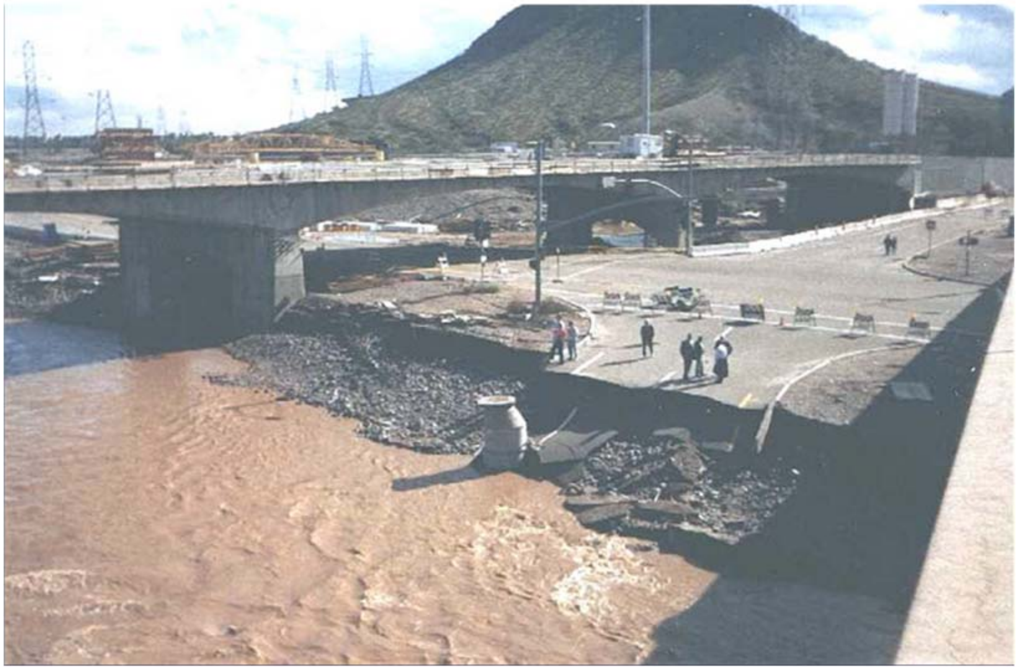
Deposition	A process where sediment is deposited to a channel due to the loss of kinetic energy in the conveying fluid.
Dune	Lower flow regime bed forms that are larger than ripples and smaller than bars. Dunes are large triangular-shaped sand elements similar to ripples with a length from 2 feet to hundreds of feet. The size of a dune is related to the flow depth. It indicates higher sediment transport rates than ripples.
Erosion	Displacement of soil particles due to water or wind action.
Fluvial sediment	Fragmentary material that originates from weathering of rocks and is transported by, suspended in, or deposited from water.
Ripple	Lower flow regime bed forms that are smaller than dunes. Ripples are small triangular-shaped sand elements similar to dunes. Ripples have gentle long upstream slopes and steep short downstream slopes. The ripples' length ranges from 0.4 feet to 2 feet, and the height from 0.03 ft to 0.2 ft.
Scour	Erosion of streambed or bank material due to flowing water.
Sediment	Rock and mineral particles subject to movement by rainfall, runoff, streamflow and wind forces.
Sediment concentration	Ratio of the weight of the sediment to the weight of the water-sediment mixture; or ratio of dry weight of sediment divided by the weight of the water-sediment mixture and multiplied by 1 million for the convenience in lab (in parts per million, ppm); or ratio of the weight of dry sediment to the volume of water-sediment mixture; or ratio of the volume of sediment to the volume of water-sediment mixture.
Sediment discharge (or sediment load)	Sediment transport rate, the quantity of sediment by either weight or volume that is conveyed past any cross section of a watercourse in a given unit of time. The discharge may be limited to certain sizes of sediment or to discharge through a specific part of the cross section.
Sediment yield	A measure of the watershed sediment outflow by weight or volume at a point of interest in the drainage network for a particular return period storm event or a specified period of time.
Sedimentation	Process of erosion, entrainment, transportation, deposition and compaction of sediment.
Suspended load	Sediment load for the sediment particles that are supported by the turbulent motion in the stream flow.

Total load (or total sediment load or total sediment discharge)	Sediment load for an entire cross section. It is bed load plus suspended load from the viewpoint of the movement mechanism. It is bed-material load plus wash load from the view point of streambed availability.
Wash load	That part of the total sediment load that is composed of fine particles (generally clays and colloids) in the suspended load that are continuously maintained in suspension by flow turbulence. Wash load originates primarily from erosion on the land slopes of the drainage area and is present to a negligible degree in the bed itself. D_{10} or D_5 from the bed material size distribution can be used to separate the wash load from the bed material load.
Watercourse	A general term for all open water conveyances typically called rivers, streams, creeks, washes, arroyos, etc., in their natural state, but also includes constructed conveyances such as channels, canals, ditches, etc.

11.2 INTRODUCTION

Sedimentation embodies the processes of erosion (Photograph 11.1), entrainment, transportation, deposition (Photograph 11.2), and the compaction of sediment (ASCE, 1975, 2006). Sedimentation engineering deals with analyses, modeling, design, and mitigation measures to analyze and solve problems associated with sedimentation. This chapter provides the basic concepts of sedimentation engineering and analytical methods and design procedures for sediment yield and scour estimation. Specific references are provided throughout this chapter. Some useful general references in the topic of erosion and sedimentation include ASCE (1975, 2006, 2008), ADWR (1996), USDOT (2001d), Simons and Senturk (1992), Simons, Li and Associates (1985), Guy (1989), Henderson (1966), Julien (2002), Schumm (1977), SCS (1977), and USACE.

PHOTOGRAPH 11.1
ERODED ROAD SEGMENT



PHOTOGRAPH 11.2
CULVERT CONVEYANCE CAPACITY REDUCED BY SEDIMENT DEPOSITION

11.3 SEDIMENTATION PROBLEMS

The sedimentation problems for erosion, sediment transport, and sediment deposition are summarized in ASCE (1975, 2006). ASCE (2008) gives a more updated discussion on various issues and problems related to sedimentation. The major erosion problems are related to geologic erosion, accelerated erosion by man's activities, and water quality. As indicated by Hooke (1994), "humans are arguably the most important geomorphic agent currently shaping the surface of the Earth." The accelerated erosion by man's activities include agricultural practices, urbanization, road and highway construction, mining operations, altering runoff conditions, and stream and river control works. Sediment in water is often viewed as an undesired element for municipal and industrial water.

The major sediment transport problems include sediment movement, sediment impingement (especially large size bed-load rocks) on structures and mechanical devices, and sediment in suspension. The major sediment deposition problems are deposits at the breaks of eroding slopes, floodplain deposits, deposits in channels (drainage ditches, irrigation canals, navigation and natural streams), and deposits in lakes and reservoirs.

As discussed in USACE (1989, 1995), the sedimentation problems associated with flood protection channels are likely to start at the following locations:

1. *Braided channels.*
2. *Changes in channel width.*
3. *Bridge or other structures built across the stream.*
4. *Channel bends.*
5. *Abrupt changes in channel bottom slope.*
6. *Long, straight reaches.*
7. *Tributary and local inflow points.*
8. *Diversion points.*
9. *Upstream from reservoirs or grade control structures.*
10. *Downstream from dams.*
11. *The downstream end of tributaries.*
12. *The approach channel to a project reach.*
13. *The exit channel from a project reach.*

The sedimentation problems associated with reservoirs include:

1. *Volume of deposition.*
2. *Location of deposits.*
3. *Rise in water surface elevations.*
4. *Aesthetics of deposited sediment.*
5. *Turbidity.*
6. *Density current.*
7. *Water quality aspects of sedimentation.*
8. *Shoreline erosion.*
9. *Shifting location of channels.*
10. *Downstream degradation.*
11. *Changes in downstream channel capacity.*
12. *Local scour at the dam, spillway and stilling basin.*

Since Maricopa County is in a semi-arid region, the dryland landscapes and "flashy" storms produce a large amount of sediment entrainment and transportation in a way very different from a humid environment. The major sedimentation problems in Maricopa County include:

1. *Channel bed erosion.*
2. *Channel bank erosion.*
3. *Scour at bridge piers, abutments, and guide banks.*
4. *Scour at power line poles in channels.*
5. *Scour for flood control structures such as bank protection and spur dikes.*
6. *Scour at downstream end of grade control and drop structures.*
7. *Scour at downstream end of culverts.*
8. *Scour at utility crossings (natural gas lines, sewer lines, etc.).*
9. *Lateral shift of channel thalweg alignment.*
10. *Degradation downstream of dams and flood retarding structures (FRS).*
11. *Headcut and tailcut caused by channel sand and gravel mining.*
12. *Deposition at retention basins, detention basins, FRS, dams.*
13. *Earth spillway erosion for FRS and dams.*
14. *Deposition at alluvial fans.*

11.4 SOLUTIONS TO SEDIMENTATION PROBLEMS

The structural or non-structural solutions to sedimentation problems are based on a good understanding and analysis of the sediment source and transport. In order to measure the short-term and long-term results of a proposed solution, it is important to define and evaluate the base condition as the basis for comparing with the proposed solution. Since channels are always in the natural sedimentation processes of degradation, aggradation, bank erosion, and thalweg lateral shifting, the base condition should not be the current existing bed elevation condition. Instead, it should be the "future" base condition which reflects the changes in those natural sedimentation processes as a function of time. USACE (1989, 1995) discusses this concept extensively and also defines it as the "no-action condition." For example, when using a sediment transport model to evaluate the tailcut depth and distance (erosion moving downstream from the pit's downstream end) due to a proposed sand and gravel mining pit, one should first compute the maximum scour depth at each cross section during the entire simulation time period for a regulatory flow hydrograph without the proposed pit. These scour depths without the pit are due to natural sedimentation processes and are the "future" base condition. Then, a sediment transport model with the proposed pit should be run to compute the maximum scour depth at each cross section during the entire simulation time period for the same regulatory flow hydrograph. These scour depths correspond to the "future" project condition. The comparison should be made between the "future" base condition and "future" project condition in order to measure the results of a proposed solution.

The natural sedimentation processes to be quantified for a stream's "future" base condition are summarized by USACE (1989, 1995) as:

1. *Location and rate of bank erosion.*
2. *Location and rate of bed erosion.*
3. *Location and rate of deposition.*
4. *Lowering or raising the base-level of the stream system water surface elevations.*
5. *Channel basic geometry (width, depth, and slope).*
6. *Turbidity.*
7. *Water quality aspects of sedimentation.*
8. *Shifting location of deep-water channels.*
9. *Head-cutting of the approach channel.*
10. *Headcutting up tributaries.*
11. *Aggradation of the exit channel.*
12. *Local scour at bridges and hydraulic structures.*

Once the natural sedimentation processes are quantified for the "future" base condition, different alternatives can be analyzed to mitigate the sedimentation hazards as possible solutions to the sedimentation problems. The general solutions to sedimentation problems may be classified as structural measures and non-structural measures. The structural measures for channel bank erosion protection can be direct protection, indirect protection, and grade control (USACE, 1989, 1995). Direct bank protection measures include riprap, gabions, other flexible mattresses, soil cement, and concrete. The indirect protection includes impervious or pervious dikes that are constructed away from the banks to deflect the erosive forces. Grade control structures can be used to reduce the bed slope, and thus the velocity and erosion in the channels. The structural solutions to aggradation can be debris basins, periodic removal of sediment, and upstream grade control structures that reduce the bed slope, velocity, and the sediment supply. The structural solutions to degradation are drop structures that reduce the bed slope and overall channel velocity. The structural solutions to sand and gravel mining headcut problems can be structural erosion barriers installed upstream of the pit and below the channel bed.

The non-structural measures to sedimentation problems are through scour anticipation, erosion hazard setbacks, and erosion monitoring and maintenance. If the anticipated scour depth can be estimated, the channel bank protection toe down should be extended beyond the anticipated scour depth. The anticipated scour depth is critical to bridge pier, bridge abutment, guide banks, spur dikes, culvert outlets, power line poles, and utility crossings. Erosion setback is another useful non-structural measure to manage the scour or deposition hazard. For example, if the lateral erosion distance can be estimated, a new house shall not be allowed to be built inside the lateral erosion zone without adequately designed structural measures. Another example is the setback for tailcut erosion caused by a sand and gravel mining pit. The tailcut setback must be reserved as a "no-mining" space so the erosion caused by the proposed pit is contained and mitigated within the pit owner's property line. The erosion downstream of the pit's property line should not be more than the erosion for the "future" base condition without the pit or at least the difference in erosion between the "future" base condition and the "future" project condition should be within a reasonable engineering accuracy level. Finally, erosion/deposition monitoring tied to maintenance program may be a very cost-effective non-structural measure to reduce some sedimentation problems.

One of the most important objectives for sediment engineering analyses is to develop a future base condition for sediment transport/degradation/aggradation; then perform similar analyses for proposed alternatives such that the proposed alternatives will not have adverse impacts on adjacent, downstream or upstream areas. All erosion/deposition hazards should be contained within the project owner's property lines. Therefore, these hazards should be mitigated by the property owner and no negative impact should occur on adjacent, downstream, or upstream property owners.

11.5 FUNDAMENTALS FOR ALLUVIAL CHANNELS

11.5.1 Channel Patterns and Sinuosity

An alluvial channel flows in its own deposits. These deposits are called alluviums. The patterns or plan forms of alluvial channels are classified as straight, meandering, and braided (Leopold and Wolman, 1957). In a natural river system, straight channels are rarely observed. The majority of alluvial streams and channels are meandering or a combination of straight, meandering, and braided patterns. As defined by Blench (1986), meandering is the longitudinal progression in time and space of a river's sinuous course of characteristic plan forms minus more or less periodic obvious short-circuits. A meandering reach has alternating bends and a plane form that is S-shaped. Sinuosity for a channel reach is defined as the ratio of channel length (measured along the channel) to the valley length (the length of a straight line connecting the reach's two ends). Sinuosity measures the meandering magnitude of a channel reach. For a straight channel, the sinuosity is 1.0. Leopold and Wolman (1957) classified channels with a sinuosity greater than 1.5 as meandering channels. For sub-meandering problems where there is a meandering low flow channel within the main channel, the low flow channel sinuosity measurement should be made along the thalweg.

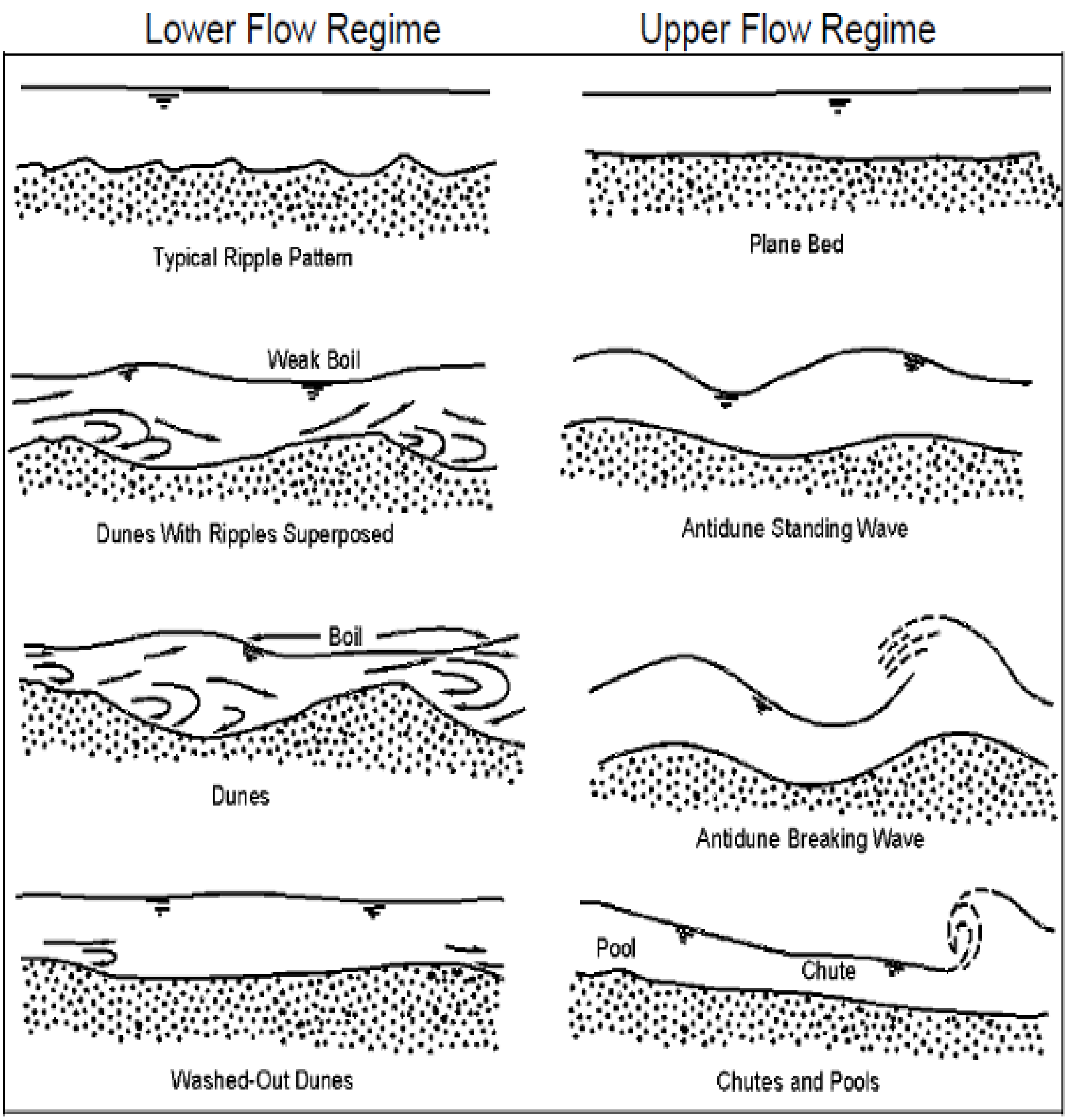
A braided channel is defined as one which flows in two or more channels around alluvial islands (Leopold and Wolman, 1957). The field observations and lab experiments by Leopold and Wolman (1957) indicate that a braided system develops when coarser material can not be transported downstream and starts to deposit. The coarser material forms a bar which may become an island. The bars or islands divide the channel into braided channels which have a smaller discharge, a larger grain size, and a steeper slope than the undivided channel. In general, meandering channels have mild slopes while braided channels have steep slopes. From the viewpoint of channel stability, meandering channels are more stable than braided channels. Braided channels are mainly bed load channels while meandering channels are suspended load channels. Although a braided channel may have some meandering reaches, its overall pattern is much less sinuous than a meandering channel. It is common to observe that the thalweg line (the line of the maximum depth) meanders within a relatively straight channel or a braided channel. Leopold and Wolman (1960) developed a relationship between the slope and bankfull discharge for a variety of natural streams. Based on this relationship, when the bankfull discharge is 1,000 cfs, if the channel slope is greater than 0.003 ft/ft, then the channel is braided; if the channel slope is less than 0.003 ft/ft, then the channel is meandering. When the bankfull discharge is 10,000 cfs, if the channel slope is greater than 0.001 ft/ft, then the channel is braided; if the channel slope is less than 0.001ft/ft, then the channel is meandering.

11.5.2 Bed Forms

Alluvial channels are either sand-bed channels, gravel-bed channels or a combination of both. Most alluvial channels are sand-bed channels. The bed forms for gravel-bed channels are relatively flat with or without bars (Shen and Julien, 1993). A bar consists of a large amount of sediment deposited over a relatively large area on the channel bed. Bars are plane bed forms having lengths of the same magnitude as the channel width or greater, and heights comparable to the mean flow depth. The longitudinal profiles for bars are approximately triangular with very long gentle upstream slopes and short downstream slopes that are approximately the same as the angle of repose (USDOT, 2001d). Bars can be classified as point bars, alternate bars, middle bars, and tributary bars. Point bars occur at the inside banks of channel bends due to sediment deposition caused by secondary flow. Alternate bars occur in straighter channel reaches and distribute periodically along the reach. Transverse bars also occur in straight channel reaches. However, they occupy nearly the full channel width while alternate bars occupy a much smaller width. Tributary bars occur immediately downstream of the lateral inflow confluence with a main channel. Middle bars occur in the middle of a channel.

The flows in sand-bed channels can be categorized into lower regime flows, transition regime flows, and upper regime flows. As indicated by Shen and Julien (1993), the dividing line between lower regime and upper regime flow is generally close to the critical flow where the flow Froude number is 1.0. For larger flumes and rivers, the shifts from lower flow regime to upper flow regime can occur at Froude numbers as low as 0.2 (USDOT, 2001d). Figure 11.1 shows the flow regimes and their typical bed forms (USDOT, 2001d).

FIGURE 11.1
BED FORMS AND FLOW REGIMES
(USDOT, 2001d)



The lower flow regime consists of a plane bed without sediment movement, ripples, dunes with ripples superposed, or dunes. The above figure shows ripples and dunes bed forms. The upper flow regime consists of plane bed, antidunes with standing waves, antidunes with breaking waves, or chutes and pools.

In the lower flow regime, resistance to flow is large and sediment transport is small. For a plane bed without sediment movement, the Manning's n is 0.014 and sediment concentration is 0 mg/ liter (Julien, 2002). When the average shear stress on the channel bed exceeds the critical shear stress, the sediment particles start to move and the plane bed starts to change to ripple bed form. Ripples are small triangular-shaped sand elements on the channel bed, with gentle long upstream slopes and steep short downstream slopes. The upstream slope is generally between 2 and 4 times as long as the downstream slope (Chien and Wan, 1998). The ripples' length ranges from 0.4 feet to 2 feet, and the height from 0.03 ft to 0.2 ft. The Manning's n ranges from 0.018 to 0.028 and the sediment concentration ranges from 10 to 200 mg/liter (Julien, 2002). The occurrence of ripples has little correlation with the water depth and is the result of the unstable viscous layer near the boundary (Chien and Wan, 1998). When the velocity increases, ripples will eventually become dunes. Dunes are large triangular-shaped sand elements similar to ripples with the length ranging from 2 feet to hundreds of feet. The size of a dune is closely related to the water depth. In the Mississippi River, 700 ft long and 40 ft high dunes are observed (USDOT, 2001d). The amplitude of dunes can increase with increasing flow depth. Like ripples, the longitudinal cross section of the dune shape is not symmetrical (longer upstream slope and shorter downstream slope). The water surface wave and dune wave are out of phase. When the water surface wave reaches crest (trough), bed dune wave reaches trough (crest). The Manning's n for dune bed channels ranges from 0.02 to 0.04 and the sediment concentration ranges from 200 to 3,000 mg/liter (Julien, 2002).

As the velocity increases further, the dunes elongate and reduce in amplitude, which results in washed out dunes, also referred as transition regime (USDOT, 2001d). The Manning's n for washed-out dunes ranges from 0.014 to 0.025 and the sediment concentration ranges from 1,000 to 4,000 mg/liter (Julien, 2002). As the velocity further increases, a plane bed form with sediment movement will develop. A large sediment transport magnitude is characteristic of this bed form. The Manning's n for a plane bed with sediment movement ranges from 0.01 to 0.013 and the sediment concentration ranges from 2,000 to 4,000 mg/liter (Julien, 2002). As the velocity further increases, antidunes will develop. Unlike ripples and dunes, the antidune sand wave is symmetrical along the longitudinal cross section of the sand waves. The water surface wave and antidune sand waves are in phase, i.e. both waves reach the crests and troughs at the same time or the water surface streamlines parallel with the river bed. While ripples and dunes can only travel in a downstream direction, antidunes can travel in both directions or remain stationary. The antidunes often form in shallow flows but moving at high velocities (Chien and Wan, 1998). It may be pointed out that although the antidunes wave profile can move in an upstream direction, the sediment particles still move in a downstream direction.

When there are antidunes, the water surface undulates. The amplitude of the water surface wave may exceed that of the antidune sand wave by a factor of 1.5 to 2 (Chien and Wan, 1998). This may have a significant impact on freeboard design for flood control channels. The antidunes with standing waves are those sand waves that gradually subside. The antidunes with breaking waves are those sand waves that grow in height until they become unstable and break like the sea surf (USDOT, 2001d). If the antidunes do not break, the resistance is about the same as that for a plane bed. If the antidunes break, resistance is larger than that for a plane bed with sediment movement. The Manning's n for antidunes ranges from 0.01 to 0.02 and the sediment concentration ranges from 2,000 to 5,000 mg/liter (Julien, 2002).

Chutes and pools form at very steep slopes with high velocities and high sediment discharge rates (USDOT, 2001d; Simons and Senturk, 1992). Large elongated mounds of sediment creates chutes where the flow is rapid and followed by a hydraulic jump and a pool. This bed form consists of chutes connected by pools. The Manning's n for chutes and pools ranges from 0.018 to 0.035 and the sediment concentration ranges from 5,000 to 50,000 mg/liter (Julien, 2002).

11.5.3 Regime Theory

British engineers working in India and Pakistan at the end of the nineteenth century and early twentieth century pioneered the stable channel design concept or dynamic equilibrium design concept. They studied many man-made irrigation canals which did not have any degradation or aggradation problems. Empirical relationships were developed for sediment size, flow discharge, velocity, cross sectional area, width, depth, slope, and other hydraulic parameters by Kennedy (1895), Lindley (1919), Lacey (1929, 1937), Blench (1951, 1957, 1964, 1969), Simons and Albertson (1963), and other investigators. When a channel does not have any degradation or aggradation problems, it is called a stable or equilibrium channel. The approach to the design of stable channels or dynamic equilibrium channels is commonly called regime theory. Those stable or equilibrium channels are often called "in regime." As defined by Blench (1986), "in regime" for a channel implies "having acquired a long-term average steady state in boundaries of its transported sediment."

The empirical relationships between various geometric variables and hydraulic parameters are often called regime equations. After studying twenty-two channels of the Upper Bari Doab Canal which did not have degradation or aggradation problems, Kennedy (1895) proposed a regime equation: $V = 0.84D^{0.64}$ where V is the mean velocity and D is the flow depth (Graf, 1984). Lindley (1919) investigated 786 observations in the Lower Chanab Canal and modified Kennedy's equation. He also proposed an equation relating mean velocity with the channel width. Later, Lacey (1929) introduced a silt factor into the regime equations by including sediment size. Chien (1955) found that the silt factor implicitly accounts for sediment load. Lacey (1937) used dimensional analysis to derive regime equations. Blench (1957, 1964, 1969) introduced the bed sediment factor and the side factor and developed more generalized regime equations based on the Indian canal data and some laboratory data.

Regime equations were also discussed by Schumm (1971), and practical guidance for selected regime equations are provided by Leopold and Maddock (1953) and Mahmood and Shen (1971). Leopold and Miller (1956) developed three regime equations to relate channel width, depth, and velocity with the flow rate by studying 20 river cross sections for rivers in the Great Plains and the Southwest semi-arid conditions. Simons and Albertson (1963) developed more generalized regime equations based on canals in both India and the United States. Their regime equations have wide applicability because their data range was much wider than that of previous equations. They also developed regime equations for different types of channel/bank material (sand and cohesive material). Regime equations for gravel-bed channels were discussed in Bray (1982) and Hey and Thorne (1986). Julien and Wargadalam (1995) refined a regime approach based on fluid mechanics principles.

It should be mentioned that the application of regime equations must be made with care since they are empirical. Regime equations should be applied to channels which have conditions similar to the channels for which the regime equations were developed.

11.5.4 Equilibrium Concept and Channel Design

As discussed above, when a channel does not experience degradation or aggradation, it is at its stable or equilibrium condition or it is "in regime." Herein, more discussion is provided about the equilibrium condition. The geomorphic equilibrium concept is shown by Lane's qualitative relationship (Lane, 1955), Equation (11.1), and is illustrated in Figure 11.2:

$$Q_w S \propto Q_s d_s \tag{11.1}$$

where:

Q_w = water discharge, units

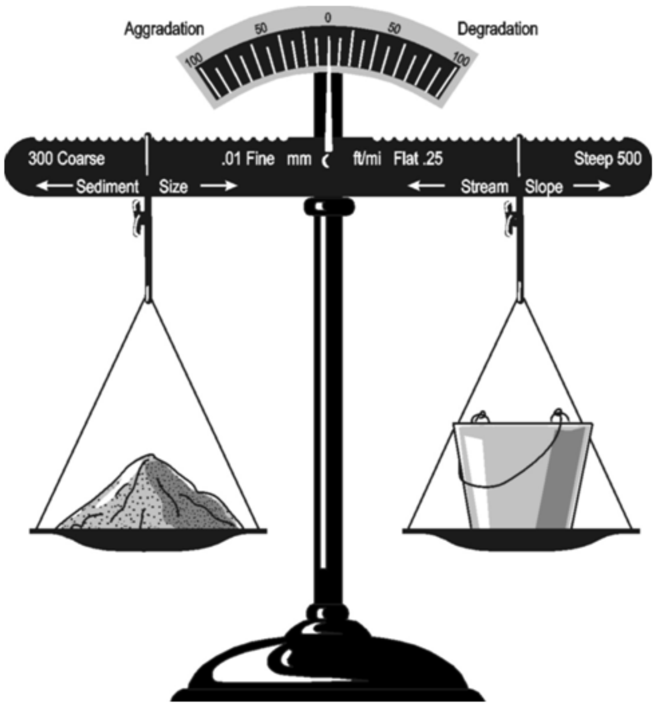
S = longitudinal slope of the watercourse, units

Q_s = bed material, discharge

d_s = characteristic bed material.

Alteration of one variable, as illustrated by the "balance" of Figure 11.2, will indicate the effect upon the others. In practice, it is often useful to consider two of the variables to remain constant while considering the quantitative effect in one variable by a change in the fourth variable.

FIGURE 11.2
LANE'S RELATIONSHIP
(Lane, 1955)



A reach of an alluvial channel is said to be in equilibrium or dynamic equilibrium if the amount of sediment inflow into this reach is equal to the sediment outflow from the reach. In other words, the channel reach's sediment transport capacity is equal to the sediment inflow. The bed elevation for this reach does not change over a long period of time. There may be some local bed elevation or cross sectional changes, but the overall reach bed elevation does not change. The channel design based on the equilibrium concept is to design a channel reach's parameters such as channel bed slope, width, and depth such that the channel sediment transport capacity is equal to the sediment inflow. Since channels "in regime" are equivalent to channels at an equilibrium condition, the regime equations can be used for equilibrium channel design.

If the inflow does not contain sediment, i.e. it is a clear-water condition, the equilibrium condition requires the sediment particles in the reach to remain stationary, so there is no sediment outflow from the reach since there is no sediment inflow. When a reach reaches equilibrium under clear-water conditions, it is said to have reached a stable condition. As defined by Lane, (1955), a stable channel is "an unlined earth channel (a) which carries water and the banks and bed of which are not scoured objectionably by moving water, and (b) in which objectionable deposits of sediment do not occur." The US Bureau of Reclamation has developed the tractive force method for a stable channel design for coarse noncohesive material carrying clear water or water with wash load (Lane, 1955). This method requires that the shear force on the sediment particle by the fluid must be less than the critical tractive force. Other methods involve using zero sediment transport capacity to determine the stable slope with width and depth design parameters. Such equations are the Schoklitsch zero bedload equation (Schoklitsch, 1932), Meyer-Peter and Muller beginning transport equation (Meyer-Peter and Muller, 1948), and the Shields diagram method (Pemberton and Lara, 1984).

Extremal hypothesis is another approach to describing alluvial channels equilibrium condition (ASCE, 1998a). It is based on the maximization or minimization of stream power, energy dissipation rate, or sediment concentration (Chang, 1980, 1988a, 1988b; White et al., 1981; Yang et al., 1981; Yang and Song, 1986; Bettess and White, 1987; Yang, 1992; Millar and Quick, 1997). As indicated in ASCE (1998a), although the theoretical justification for this approach is not entirely clear, the predictions provide global agreement with a wide range of observations.

11.5.5 Equilibrium Concept Challenges in Arid and Semiarid Regions

It should be mentioned that alluvial channels in arid and semiarid regions are different from those in humid regions because (1) the topography and landforms are more abrupt; (2) the soils are thinner, (3) the bedrock exposures are more pronounced; (4) the streams are smaller and are likely to be dry; (5) the streams carry large sediment loads from erosion by both wind and water; and (6) flash floods produce high sediment removal and transportation (Simons, Li & Associates, 1982). Although the equilibrium concept was used in early work on alluvial channels in semiarid or arid regions, its validity has been subject to debate. Slatyer and Mabbutt (1964) stated: “Despite their ephemeral character, drainage channels in arid regions show the same tendency to adjust in equilibrium with hydraulic factors….” As indicated by ASCE (1998a), because the channel geometry in arid and semiarid regions of the American West change drastically many channels cannot be considered "in regime" or equilibrium. Such channels are constantly enlarging rapidly in width due to high flows or contracting due to less than average runoff. Hooke and Mant (2002) argued: “It has long been accepted that ephemeral channels tend to be in a non-equilibrium state and are therefore unstable, with some propensity for sudden switches of characteristics….”

As discussed in Nanson et al. (2002), some dryland rivers (semiarid, arid, and extremely arid) are in equilibrium and some others are in non-equilibrium just as rivers in humid regions. In particular, the upland reaches in dryland rivers are susceptible to dramatic changes during large floods. Such reaches are characterized largely by non-equilibrium conditions. The moderate-gradient reaches can be in equilibrium for long periods of time. But they can be in non-equilibrium during extreme flood events. The low-gradient lowland reaches are more like to be in equilibrium. Richards (1982) suggested four conditions to check to verify a river is in equilibrium: (1) channels have remained essentially stable despite occasional large floods; (2) sediment transport discontinuities are essentially insignificant in terms of disrupting equilibrium channel form and process in individual reaches; (3) there are strong correlations between channel form and process variables; and (4) dryland rivers can be adjusted to maximum sediment transport efficiency under conditions of low gradient, abundant within-channel vegetation, and declining downstream discharges.

It may be mentioned that whether or not a channel is at an equilibrium condition is relative and is a function of time scale. A channel may be in equilibrium for a short time period, but may not be in equilibrium for a long time period. An equilibrium channel is also a function of the magnitude of rainfall and climate changes. A channel in Maricopa County may reach its equilibrium condition in the past 10 years with a series of nominal discharges. The equilibrium condition can be easily disrupted when a larger flood event occurs at the end of the 10-year period. To understand the equilibrium condition, it is very important to consider the applicable time scales, which could be 1,000 years, 100 years, 10 years, 1 month, 1 minute, or any other time intervals.

Another challenge is related to the use of sediment transport equations. Most sediment transport equations were developed under a steady state condition, i.e., the flow and sediment data are collected when flow and sediment movement do not change with respect to time during the observation. The steady state can be considered as a special case of an equilibrium condition for a short time scale. Under the equilibrium condition or a steady state condition, there is no degradation or aggradation in the channel reach and the sediment inflow is equal to the sediment outflow for the reach. Under these conditions, the actual sediment transport rate is the same as the sediment transport capacity rate. Therefore, most sediment transport rate equations are the sediment transport capacity rate equations. In fact, most sediment models are based on an assumption that the actual bed-load transport rate is equal to the transport capacity under the equilibrium condition at every computational time step (Wu, 2007). However, sediment transport in natural rivers is not usually in a state of equilibrium and sediment cannot reach a new equilibrium state instantaneously due to the temporal and spatial lags between flow sediment transport (Wu, 2007). In addition, flumes in laboratories usually have solid walls and therefore the bank effects are not included in the sediment transport equations, which limits the application of the equations in natural rivers because natural rivers are usually subject to bank erosion. When natural rivers are not in a steady state condition during a flood such as a flash flood, the sediment transport equations may not give accurate results because most sediment transport equations were developed under the steady state condition.

Han (1980) derived a non-equilibrium sediment transport equation based on a convective-diffusion equation considering wash load. Bell and Sutherland (1983) performed experiments to study the nonequilibrium bedload transport behavior. Nakagawa et al. (1989) used convolution-integral modeling to describe non-equilibrium bed load sediment transport. Chien and Wan (1998) extensively studied the non-equilibrium condition where the sediment concentration distribution varies in the streamwise direction. Recently, Wu and Vieira (2002) improved Bell and Sutherland's approach and added non-equilibrium total sediment transport into the CCHE1D model (Wu and

Vieira, 2002). More discussion on non-equilibrium sediment transport can be found in Wu (2007).

Non-equilibrium and unsteady state channel condition and sediment transport phenomenon pose challenges for river mechanics engineers in arid and semiarid regions. Further research is needed in this field. Application of equilibrium conditions to alluvial channels in arid and semi-arid regions such as Maricopa County is subject to great uncertainty and results should be carefully reviewed. Engineering judgment must be applied, and a safety factor should be considered to account for uncertainties and risks.

11.5.6 Channel Bed Scour and Deposition

As classified by Julien (2002), a fluvial system can be divided into three main zones: (1) an erosional zone of runoff production and sediment scour; (2) a transport zone of water and sediment conveyance; and (3) a depositional zone of runoff delivery and sedimentation. The second zone may be considered as an equilibrium condition where the inflow sediment to the zone is transported downstream and there is no net scour and deposition in the zone. The first and third zones are not in equilibrium because the inflow sediment and outflow sediment are not equal. The scour zone is where the sediment outflow is larger than the sediment inflow. The depositional zone is where the sediment outflow is less than the sediment inflow. Channel bed scour is classified by USACE (1989, 1995) as degradation and local scour. Degradation is a general lowering of the stream bed elevation for long reaches due to erosion of the bed sediments. Degradation for a channel occurs when there is a reduction in upstream sediment supply or when channel sediment capacity is greater than the sediment inflow. For example, a dam that has captured sediment and therefore reduced the sediment volume moving downstream will cause degradation in the reach downstream of the dam. Headcutting is another type of degradation that causes a rapid drop in the stream bed. It moves in the upstream direction and may cause bridge failures and bank failures. Local scour is a form of scour limited to particular locations such as bridge piers, bridge abutments, bridge guide banks, the downstream end of drop structures, the downstream end of culverts, and scour caused by sand/gravel mining pits.

Channel bed deposition is classified by USACE (1989, 1995) as aggradation and local deposition. Deposition spans long reaches of a stream similar to degradation. When the inflowing sediment load exceeds the transport capacity of the stream in a reach or equivalently the incoming sediment load is more than the outgoing sediment load, deposition will occur in that reach. Local deposition is limited to deposition in isolated areas. For example, the channel will start to deposit sediments in areas where the channel width increases or the channel bed slope decreases.

11.5.7 Channel Lateral Migration, Meandering, and Bank Erosion

Channel lateral migration is the change of the channel location along the direction normal to the channel flow direction. Examples include bank failure and lateral erosion as shown in Photograph 11.3 and Photograph 11.4. Channel meandering constitutes most of the channel migration. Many theories have been proposed to explain why channels meander. Five theories have been discussed and summarized in Garde and Raju (1985). The five theories are earth's rotation theory, disturbance theory, helicoidal-flow theory, instability theory, and excess energy theory. The earth's rotation theory assumes that the Coriolis force is able to deflect the stream. The Coriolis force is the force that applies to an object normal to its moving direction. The disturbance theory assumes that disturbances in the channel will cause the channel to meander. Tiffany (1939) found that an initially straight channel remained straight when the factors which disturbed the current were eliminated. The helicoidal-flow theory assumes that helicoidal flow is responsible for the meandering. Helicoidal flow is also called secondary flow, transverse flow, secondary circulation, or secondary current. Helicoidal flow circulates around an axis that is parallel to the channel main flow direction. Helicoidal flow exists in both bends and straight noncircular channels. It erodes the toe of the banks and causes bank failure at the outer bend. Instability theory is an analytical approach based on perturbation techniques for equations of motion and the continuity equations for water and sediment for one-dimensional models, two-dimensional models, and three-dimensional models.

PHOTOGRAPH 11.3
BANK FAILURE DUE TO EROSION AT BANK TOE

PHOTOGRAPH 11.4
VERTICAL BANKS DUE TO LATERAL EROSION

The excess energy theory assumes that a channel with excess energy tends to damp out the excess energy by increasing its length, therefore meandering. Leopold et al. (1964) stated that, "The meandering pattern approaches more closely the equilibrium condition as defined by the entropy concept than does the nonmeandering one." Under the equilibrium condition, the channel adjusts so that the rate of work expended in the system is a minimum. Leopold et al. (1964) used some examples to explain that the energy slope in a meandering channel tends to be more uniform, implying that the energy loss along the channel length is more uniform than the straight channel. In addition to the above five theories, excessive sediment load can also cause a channel to increase the bed slope and channel width therefore causing meandering or lateral migration. Channel avulsion (a channel takes a different path when the flow capacity is reduced by sediment deposition) on braided channels can be another factor to cause a channel to laterally migrate. The Kosi River on an alluvial fan in India had an annual shift rate of 1.04 kilometers from 1933 to 1950 (Garde and Raju, 1985) until levees were installed to arrest the lateral migration.

In most cases, banks must erode in order for a channel to migrate laterally or meander. Erosion along the outer bank and deposition along the inner bank have always been observed in meandering channels. The bank erosion mechanism plays a key role in studying channel migration and meandering. Bank erosion mechanisms include hydraulic forces, erosion from waves, and geotechnical failures (USACE, 1989, 1995). The hydraulic forces include tangential shear stress from the drag of the water against the channel bank and direct impingement of the water against the channel bank. Erosion from waves is generally applicable only to lake and reservoir banks. Geotechnical failures are due to bank slope instability. The channel bed erosion at the bank toe will increase the bank instability. The tangential shear stress and direct impingement play a more important role for noncohesive bank material erosion than for cohesive bank material erosion. Cohesive material is more resistant to tangential shear stress, however, it is subject to bank collapse failure due to increased pore pressure when the water levels in the channel side drop.

Ikeda et al. (1981) was among the first researchers who studied the bank erosion rate by using the linear perturbation analysis to derive an equation of bank erosion. The bank erosion rate as proposed by Ikeda et al. (1981) is assumed to be a linear function of difference between the near bank tangential velocity and the one-wavelength reach averaged velocity. Their research provided a description for the meander and migration growth. Parker et al. (1982) derived a generalized nonlinear equation for bend migration which reduced the lateral and downstream migration rates and increased the meander wavelength as compared with the linear assumption in Ikeda et al. (1981). Odgaard (1989) linked bank erosion rate to the near-bank water depth rather than the difference between the near-bank velocity and the one-wavelength reach averaged velocity.

Osman and Thorne (1988a, 1988b) provided a methodology based on a combination of lateral erosion, bed degradation, and geotechnical bank failure. Simon et al. (2000) proposed a method to include hydrostatic and pore-water pressure in the bank stability analysis. Darby and Thorne (1996) added rotational failure mechanism to the bank stability analysis of Osman and Thorne (1988a, 1988b). Hasegawa (1989) derived an equation for bank erosion and channel migration rate based on the assumption that bank erosion is proportional to the velocity difference between the near-bank velocity and centerline depth-averaged streamwise flow velocity. A universal bank erosion coefficient was proposed by Hasegawa (1989). Duan (2005) proposed a probabilistic approach to estimate the bank erosion rate for cohesive bank material of planar bank failure.

An extensive discussion on channel width adjustment and bank erosion was provided by ASCE (1998a, 1998b, 2008). It was stated in ASCE (1998a) that the width adjustment may be caused by disruption of the long-term equilibrium condition due to an extreme event or maybe a morphological response to river engineering or management. It was concluded from ASCE (1998b) that no one universal width adjustment model exists that is applicable to all circumstances and calls for further research on comprehensive field and lab data for model verification. As summarized in ASCE (1998b) and ASCE (2008), all numerical models belong to one of two general categories: those based on extremal hypotheses or those based on the geofluvial approach. The extremal hypotheses are based on minimization of stream power. The geofluvial approach is based on flow hydraulics, sediment transport, bank erosion, and bank failure modeling. A list of potential models for width adjustment is discussed in ASCE (1998b) and ASCE (2008). The examples for models based on extremal hypotheses are quasi-2D GSTARS (Yang et al., 1988) and 1D FLU-

VIAL-12 (Chang, 1988a). Examples for models based on a geofluvial approach are 1D WIDTH (Osman, 1985), 2D CCHEBank (Li and Wang, 1993), and 1D STREAM2 (Borah and Bordoloi, 1989).

Simon et al. (2000) provided a detailed discussion on the role of negative pore pressure in the unsaturated zone; the role of hydraulic forces causing bank erosion and steepening of toe profiles; effects of variability of moisture content with time on other geotechnical properties; and development of a bank-stability algorithm for layered cohesive streambanks. A bank stability algorithm was proposed to model the timing and conditions leading to mass failure with the consideration of matric suction for the portion of the failure plane, confining pressure, positive pore-water pressure, varying soil unit weight and layering within the banks (Simon, et al., 2000). Olsen (2003) developed a three-dimensional model that was successfully used to simulate the meandering streams in the laboratory, which does not require a separate bank erosion model as most other models. Olsen's model is based on finite volume that directly solves the Navier-Stokes equations with the standard $k–\varepsilon$ turbulence model. The secondary flow is directly modeled. Duan and Julien (2005) incorporated the physically based bank erosion model into the two-dimensional flow and mass conservation equations to predict bank erosion with consideration of secondary flow effects. CCHE2D and CCHE3D developed by the National Center for Computational Hydroscience and Engineering (NCCHE) in the University of Mississippi are capable of simulating channel migration and meandering (Jia and Wang, 1999; Jia and Wang, 2001). The information on CCHE3D can be obtained from http://www.ncche.olemiss.edu/index.php?page=freesoftware#cche3d.

A qualitative geomorphic analysis is also important. It should include a study of published soil maps, topography, field identification and verification of bedrock outcrops, armored channel beds, cutbanks and existing man-made bank protection structures. All available historical aerial photographs should be analyzed as a part of the geomorphic analysis in support of other estimation procedures. Comparison of historical aerial photographs should be made in a GIS framework, if possible. A discussion on some quantitative methods for evaluating the channel lateral stability can be found in USDOT (2001c). Hydraulic and geotechnical engineering principles should be applied to estimate bank stability and erosion limits. The procedure recommended by Osman and Thorne (1988a, 1988b) may be used to determine the bank erosion distance in conjunction with equilibrium slope or sediment transport modeling for channel bed scour. A simplified use of Osman and Thorne's methodology can be found in Mussetter et al. (1994). Another useful methodology developed by the National Sedimentation Laboratory can be found in Simon et al. (2000), which accounts for the effect of vegetation roots. A discussion on bank erosion mechanisms can also be found in USDOT (2001c). CCHE2D (2007 version or later), a two-dimensional hydraulic and sediment transport model, may be used to model the lateral erosion (Jia and Wang, 2001). Recently, the NCCHE has added a procedure from Osman and Thorne (1988a) to the CCHE2D model as requested by the FCDMC (Jia and Wu, 2007). The FCDMC has developed a practical engineering methodology suitable to the Maricopa County environment to estimate the lateral erosion. Detailed discussion on the methodology can be found in Section 11.9. The methodology will replace the lateral migration estimation procedures documented in Arizona State Standard 5-96 (ADWR, 1996) for Maricopa County.

In addition to quantifying the bank erosion and lateral erosion distance, mitigation through a structural solution is extremely important. The mitigation structures for bank stabilization and bend control include revetments, spurs, retardance structures, longitudinal dikes, vane dikes, bulkheads, and channel relocations (USDOT, 2001a). Detailed discussion on bank protection can be found in other chapters of this manual.

11.5.8 Headcut Migration

A headcut can be defined as a vertical drop or discontinuity in the channel bed elevation along the flow direction. A discontinuity is an abrupt break in channel bed slope. A knickpoint can be defined as the point where an abrupt break takes place. The term headcut was originally used for a vertical drop. However, herein it is used for both vertical drop and an abrupt change in channel bed slope. Headcuts are often observed in natural streams, and can move upstream to cause significant damages to roads (Photograph 11.5), bridges, banks, levees, emergency spillway for dams, and other hydraulic structures. Headcuts migrate upstream due to hydraulic stresses at the overfall, basal sapping, weathering processes, and gravitational forces on the soil mass (Hanson et al., 1999). As described in Stein and Julien (1993), headcut migration consists of two modes: 1) rotating headcuts that tend to flatten as they migrate; and 2) stepped headcuts that tend to retain nearly vertical faces.

For a channel bed made of noncohesive material, experimental studies by Leopold et al. (1964) indicate that an oversteepened slope will be flattened at a rate proportional to the rate of sediment transport. As the headcut moves upstream, the knickpoint will be obliterated by the flattening of the slope. The experimental studies by Leopold et al. (1964) indicate that in a channel bed made of cohesive material a knickpoint can move upstream, maintaining a vertical face if the following two conditions are met: 1) the material at the knickpoint has a resistance to shear stress greater than the stress provided by the flow, and 2) flow is sufficient to transport the eroded material from the base of the face. As discussed by Robinson et al. (2000), the headcut for a cohesive soils channel moves as a series of discrete cantilever mass failures and the hydraulic stresses undercut the overfall for a period of time until the headcut becomes unstable and fails. The vertical headcuts are often formed for a channel bed with two stratifying layers where the upper layer of the channel bed is resistant to erosion and the lower layer is erosive. Once the lower layer is exposed to the channel flow, it undercuts the upper layer and the upper stratum collapses, causing headcut migration upstream. Robinson and Hanson (1995) performed large-scale headcut erosion experiments in a 1.8-meter-wide and 29-meter-long flume with 2.4-meter-high sidewalls. The experiment results indicate that the underlying sand layer will significantly increase the headcut advance rate for a two-layer soil bed where the upper layer is sandy clay and the lower layer is silty sand.

A significant amount of research has been done to quantify the headcut and its migration rate. Leopold et al. (1964) indicated that when the ratio of headcut height to the flow depth is large, for example 1.5 ft/ft, the vertical drop will be maintained as it moves upstream for cohesive soil material. When the ratio is less than or equal to 1.0 ft/ft, the knickpoint is obliterated by the flattening slope. Experiments were performed by Bennett et al. (1997) for a movable bed channel. It was found that the shape of the scour hole below the knickpoint does not change as the headcut migrates upstream. Empirical equations for headcut migration rate and experimental studies can be found in De Ploey (1989), Temple (1992), Temple and Moore (1994), Robinson and Hanson (1994), Hanson et al. (1997), NRCS (1997), Temple and Moore (1997), Robinson et al. (2000), and Hanson et al. (2001). A two-dimensional model was developed by Stein and LaTray (2002) based on a cantilever mass failure model and a scour model to simulate the headcut migration rate for a two-layer soil where the upper layer is erosion-resistant soil and the lower layer is erosive soil. In 2005, another two-dimensional model was developed by Frenette and Pestov (2005) based on the vertex flow concept.

PHOTOGRAPH 11.5
SHALLOW FLOW OVER ROADWAY INITIALLY CAUSES HEADCUTTING INTO ROAD SUBGRADE AND PAVEMENT

Channel incision takes place in degradational zones where there is fine bed material Julien (2002). Rills (Photograph 11.6) are small-scale channels, and gullies (Photograph 11.7) are large-scale channels often found in upland areas. Channel incision is often found in ephemeral channels also called arroyos. Headcuts that migrate upstream are a characteristic feature of incised channels. Headcuts can also be found when a tributary channel enters into a main channel which has a lower bed elevation than that for the tributary channel. Gullies and headcuts can also be found in earthen dam spillways. A large-scale "man-made" headcut can be found in the vicinity of sand and gravel mining pits where there are abrupt changes in bed elevation.

PHOTOGRAPH 11.6
RILL EROSION

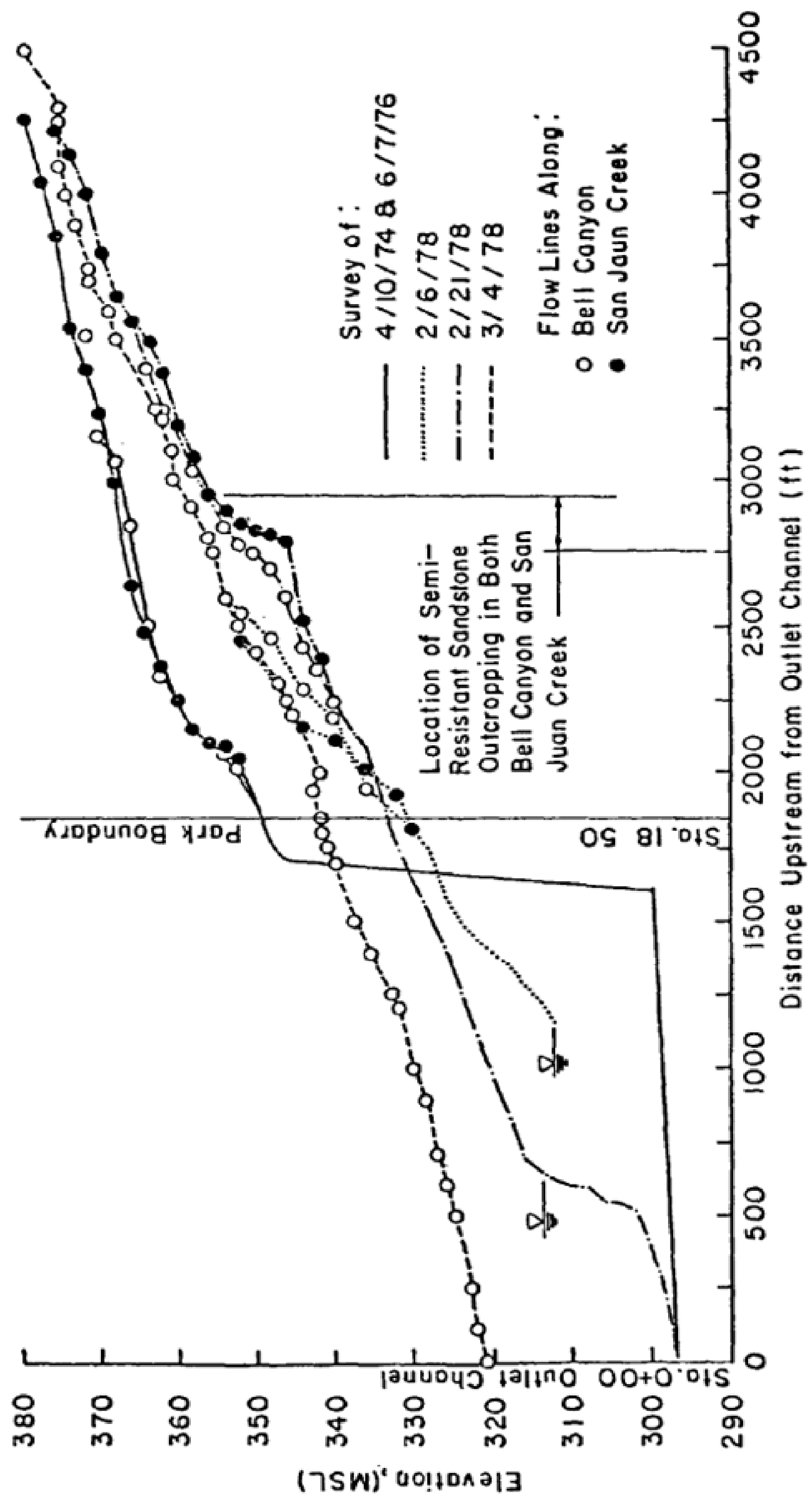
PHOTOGRAPH 11.7
GULLY EROSION

11.5.9 Sand and Gravel Mining

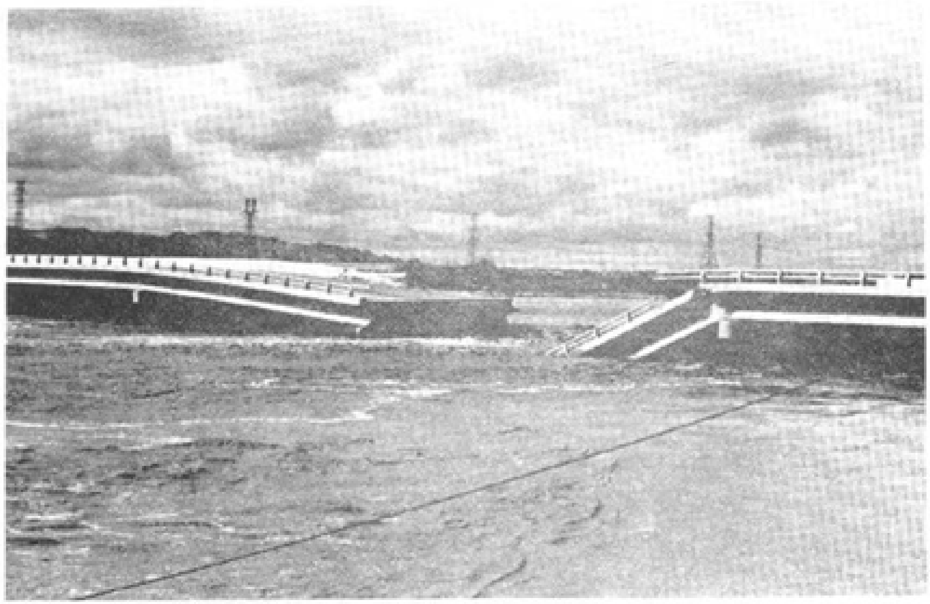
Sand and gravel mining in streambeds provides basic materials for the road and building construction industry. However, it induces significant changes to the river system. The most common effect from a sand and gravel mining pit is headcut migration. Photograph 11.8 shows a headcut that resulted from a sand and gravel pit in the Salt River downstream of 35th Avenue, Phoenix, Maricopa County, Arizona (flow direction is from east to west). The headcut and its migration distance due to sand and gravel mining pits can be very large as compared with the general headcuts found in gullies and rivers. As discussed by Kondolf (1997), the excavation of pits creates a locally steeper gradient upon entering the pit. The over-steepened knickpoint increases stream power and erodes upstream, inducing an incision or headcut that may propagate upstream for kilometers. Such headcuts or incisions can cause significant damages to bridges, buried utilities, culverts, roads, channel banks, levees, and dam spillways in the vicinity of the mining pits. For example, three bridges in Tujunga Wash in Los Angeles County, California failed due to a headcut that traveled 3,000 feet upstream from 50-75 feet deep mining pits in February 1969 (Scott, 1973). Lopez (2004) indicates that the length of channel affected by bed erosion can extend for more than 1.24 miles upstream of sand and gravel mining operations in the town of Las Tejerias, Venezuela. The riverbed has lowered 9.8 feet in the vicinity of the Las Tejerias Bridge, 0.62 miles upstream of sand and gravel mining operations. Figure 11.3 (USACE, 1989, 1995) shows channel profiles for the San Juan Creek in Orange County, California at different years. A headcut of more than 2,000 feet can be observed in Figure 11.3. Photograph 11.9 shows a bridge failure of Indian School Road Bridge at the Agua Fria River, Maricopa County, Arizona, which was in the vicinity of sand and gravel mining operations. Based on the physical modeling results for the Salt River near the Phoenix Sky Harbor International Airport (Chen, 1980), a gravel pit of 1,500-ft long, 1,000-ft wide, and 60-ft deep can have a headcut distance of about 2,750 feet and a maximum headcut depth of about 25 feet at the upstream knickpoint.

PHOTOGRAPH 11.8
SAND AND GRAVEL MINE HEADCUT

FIGURE 11.3
PROFILES SHOWING HEADCUT MIGRATION, SAN JUAN CREEK, CALIFORNIA
(USACE, 1989, 1995)
BRIDGE PIER SCOUR IN VICINITY OF MINING PITS



PHOTOGRAPH 11.9
FAILURE OF INDIAN SCHOOL ROAD BRIDGE, AGUA FRIA RIVER, ARIZONA
(USACE, 1981)



The Indian School Road bridge collapsed on the morning of February 20, 1980, the day outflows from Lake Pleasant peaked at 66,000 cubic feet per second.

Another adverse effect due to mining pits is scour downstream of the pits. This is because a sand and gravel mining pit traps most of the sediment, therefore the flow leaving the pit does not have much sediment, and the sediment-starved water removes bed material from the channel (USACE, 1989, 1995). As indicated by Kondolf (1994), the "hungry water" has excess energy and erodes its bed and banks to regain at least part of its sediment load. This erosion, often called tailcut, can also damage structures in the vicinity of mining pits. As discussed in Lee et al. (1993), Tan-Shui River, one of the major rivers in Taiwan, is subject to serious degradation problems downstream of sand and gravel mining pits. The bed elevation of the downstream reach close to the river mouth is as low as 30-40 meters below sea level, which threatens the safety of the dikes and bridges in that area. The physical modeling results by (Chen, 1980) indicate a tailcut distance of about 1,000 feet and a maximum tailcut depth of about 12 feet at the downstream knickpoint.

Other effects due to mining pits, which can damage in-stream structures, are lateral erosion, bank failure, channel widening, and low flow channel re-direction. The incision channels caused by headcuts and tailcuts can cause bank undercutting and erosion, resulting in a significant four-fold increase in the sediment load. For example, incision channeling resulted in a four-fold increase in the sediment load of Blackwood Creek, California (Kondolf, 1994). A gravel pit in San Juan Creek, California caused channel widening resulting from undercutting of banks (Kondolf, 1994). The physical modeling results by (Chen, 1980) indicate a lateral erosion of 300 feet long and 13 feet deep.

The mitigation measures for eliminating adverse impacts of sand and gravel operations consist of two approaches: set-back and structural solutions. The first approach is to estimate the headcut, tailcut, and lateral erosion distances, which are then used to establish set-back limits. The sand and gravel mining pits must be designed such that the upstream, downstream, and lateral erosion limits are confined to the mining owner's property. The estimation of headcut and tailcut for sand and gravel mining pits can be done by both empirical equations and sediment transport modeling. Lee et al. (1993) developed some empirical equations based on a series of experiments for a sand-bed flume to estimate the headcut and tailcut profiles. ADOT (Li et al., 1989) also developed empirical equations based on a computer simulation for both sand-bed and gravel-bed channels. However, there are two issues with ADOT's methodology. The first one is that ADOT's method may significantly under-estimate the headcut distance for both sand-bed and gravel-bed conditions while it may significantly over-estimate the tailcut distance for a sand-bed channel. The second issue is that ADOT's method yields a zero tailcut distance for gravel-bed channels. The FCDMC recently developed a software called PitScour to compute headcut and tailcut based on ADOT's method. The physical modeling results by Chen (1980) for the Salt River indicate that the headcut distance should be in thousands of feet for a gravel bed channel and the tailcut distance should be around a 1,000 feet for a 60-ft deep gravel pit. A field observation in the Lower Hassayampa River indicates a 0.5 mile headcut migration from a 10-foot deep sand pit upstream of the Tonopah-Salome Highway, Maricopa County, Arizona (FCDMC, 2006).

The headcut distance/depth and tailcut distance/depth for pits in Maricopa County should be estimated by using a sediment transport software such as HEC-6 (USACE, 1991), HEC-6T (MBH, 2002), Fluvial-12 (Chang, 1988a; Chang Consultants, 2006), or other FCDMC-approved sediment transport software.

When sediment transport modeling is used, both the base condition model without the pit and proposed condition model with the pit are needed. Both sediment transport models are run to obtain the maximum scour depth and deposition depth at each cross section for the entire simulation time period. It may be noted that "maximum" is with respect to the maximum value over the entire simulation time period. The scour and deposition are with respect to the channel thalweg. The scour depth at adjacent properties' boundary lines for the proposed condition model with the pit should be less than or equal to the scour depth for the base condition without the pit. If the modeling results for the base condition indicate an aggradation at adjacent properties' boundary lines, the minimum channel thalweg elevation corresponding to maximum scour depth for the proposed condition should be between the channel initial thalweg elevation and the maximum channel thalweg elevation for the base condition.

The headcut and tailcut setbacks must be reserved as "no-mining" spaces so that the scour caused by the proposed pit is contained within the pit owner's property lines unless the adjacent property owners agree with the scour depth on their properties. However, since the adjacent property owners may also plan to excavate sand and gravel pits or sell the land in the future for potential future sand and gravel activities, the sediment transport modeling for the "combined" pits should be analyzed to estimate the cumulative impact on further upstream and downstream properties. Figure 11.4 depicts the non-adverse impact scenario when the base condition is subject to natural erosion. Figure 11.5 depicts the non-adverse impact scenario when the base condition is subject to natural deposition.

It may be noted that since FLUVIAL-12 can simulate the formation of narrow headcut and tailcut channels and HEC-6 or HEC-6T's prediction is the average value over the entire erodible cross section, the scour distance and depth computed by FLUVIAL-12 may be larger than those computed by HEC-6 and HEC-6T.

The second approach to mitigation measures is through structural solutions such as grade control structures, drown-out structures, rock-filled trenches, levees, and lateral erosion control structures. It should be mentioned that grade control structures can effectively control the headcut problems, but cannot control the tailcut problems, because tailcut problems are due to the lack of sediment in the flow leaving the pits. Erosion and sediment control methods for surface mining are available in U.S. EPA (1976). Technical guidelines and procedures for analyzing sand

and gravel mining and its potential impact on watercourses are provided by USACE ((1980 and 1987), Li et al. (1989) and Urban Drainage and Flood Control District (1987).

FIGURE 11.4
NON-ADVERSE IMPACT WHEN BASE CONDITION IS SUBJECT TO NATURAL EROSION

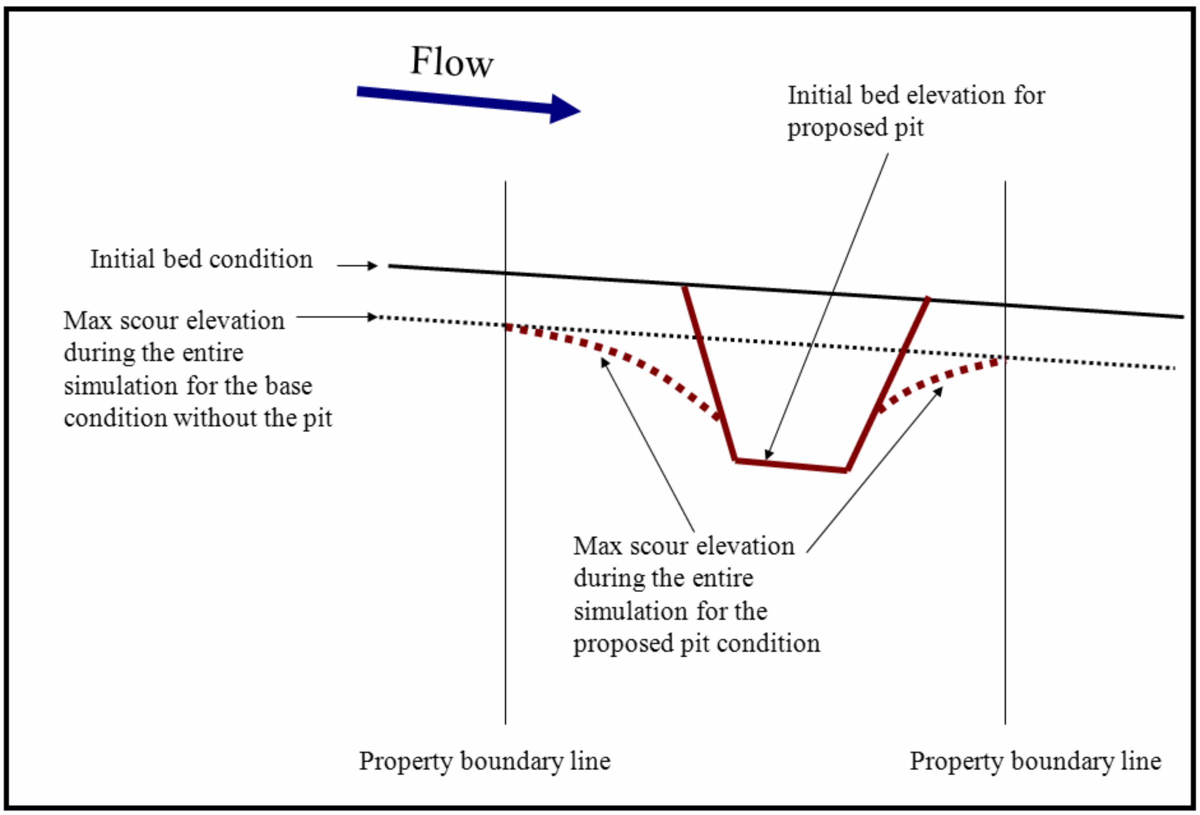
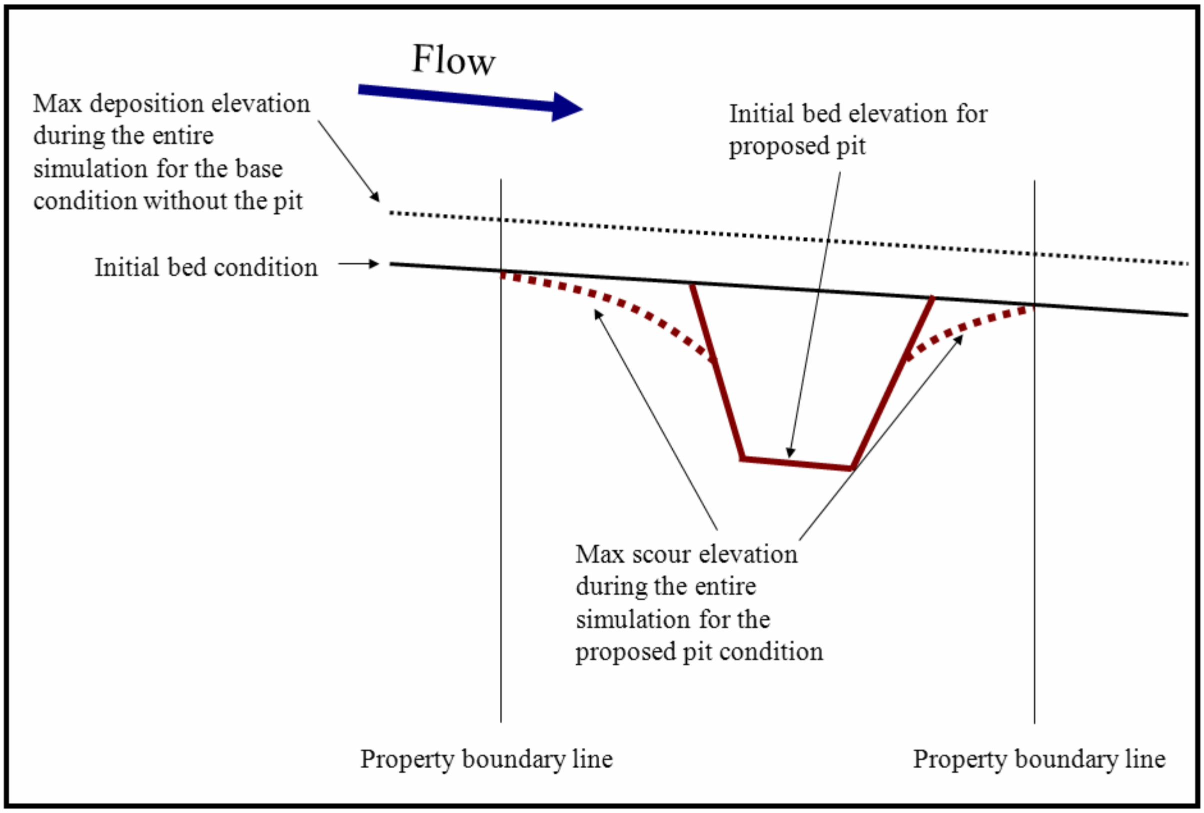


FIGURE 11.5
NON-ADVERSE IMPACT WHEN BASE CONDITION IS SUBJECT TO NATURAL DEPOSITION



11.6 FUNDAMENTALS FOR SEDIMENT TRANSPORT

11.6.1 Sediment Properties

All sediments originate from the process of weathering of rocks, where weathering can be defined as the process by which solid rocks are broken up and decayed (Garde and Raju, 1985). After the rocks are disintegrated, the sediments are transported by stream, wind, or glaciers. The sediment properties for sediment transport analysis and modeling can be classified into two categories: (a) those related to the particle itself and (b) those related to the sediment mixture or deposit (USACE, 1995). The sediment mixture or deposit consists of a group of sediment particles. The sediment properties for a group of sediment particles are also called bulk properties. Herein, the sediment properties are discussed in terms of individual sediment particles, bulk properties for sediment mixture or deposit, and a water-sediment mixture.

11.6.1.1 Properties for Individual Sediment Particles

The basic properties for individual sediment particles are particle size, particle classification, particle shape, particle specific gravity, particle specific weight, and particle fall velocity.

Particle Size

The sediment particle size is one of the most important and commonly used properties (Garde and Raju, 1985). There are four common definitions for particle size (USACE, 1995):

1. The *nominal diameter* of a particle is the diameter of a sphere that has the same volume as the particle.
2. The *sieve diameter* of a particle is the length of the side of the smallest square opening through which the given particle will pass.
3. The *sedimentation diameter* of a particle is the diameter of a sphere that has the same specific gravity and has the same terminal settling velocity as the given particle in the same fluid under the same conditions.
4. The *standard fall diameter* (or simply *fall diameter*) of a particle is the diameter of a sphere that has a specific gravity of 2.65 and has the same terminal settling velocity as the given particle in quiescent distilled water at a temperature of 24°C.

Particle Classification

Sediment particles are classified into six general categories: clay, silt, sand, gravel, cobbles, and boulders based on the particle size (USACE, 1995). There are many grading systems in the engineering and geologic literature to define the size range for each of the six categories. For sediment work and modeling, the sediment grade system proposed by the subcommittee on Sediment Terminology of the American Geophysical Union (Lane, 1947; ASCE 1975, 2006) should be used. Table 11.1 shows the sediment grade scale system.

TABLE 11.1
SEDIMENT GRADE SCALE
(Lane, 1947; ASCE 1975, 2006)

Class Name (1)	Size Range		Approximate Sieve Mesh Openings per inch	
	Millimeters (2)	Inches (3)	Tyler (4)	United States Standard (5)
Very large boulders	4,096-2,048	160-80		
Large boulders	2,048-1,024	80-40		
Medium boulders	1,024-512	40-20		

TABLE 11.1
SEDIMENT GRADE SCALE
(Lane, 1947; ASCE 1975, 2006)

Class Name	Size Range		Approximate Sieve Mesh Openings per inch	
	Millimeters	Inches	Tyler	United States Standard
Small boulders	512-256	20-10		
Large cobbles	256-128	10-5		
Small cobbles	128-64	5-2.5		
Very coarse gravel	64-32	2.5-1.3		
Coarse gravel	32-16	1.3-0.6		
Medium gravel	16-8	0.6-0.3	2-1/2	
Fine gravel	8-4	0.3-0.16	5	5
Very fine gravel	4-2	0.16-0.08	9	10
Very coarse sand	2.0-1.00		16	18
Coarse sand	1.0-0.5		32	35
Medium sand	0.5-0.25		60	60
Fine sand	0.25-0.125		115	120
Very fine sand	0.125-0.062		250	230
Coarse silt	0.062-0.031			
Medium silt	0.031-0.016			
Fine silt	0.016-0.008			
Very fine silt	0.008-0.004			
Coarse clay	0.004-0.002			
Medium clay	0.002-0.001			
Fine clay	0.001-0.0005			
Very fine clay	0.0005-0.00024			

Note: A 200 US Sieve Size (Tyler 200 Mesh) corresponds to 0.074 mm (0.0029 in) opening size.

Particle Shape

As discussed by Garde and Raju (1985), the sediment particle shape affects the bed load transport, fall velocity, and mean velocity of the flow at which the particle on the bed moves. Sediment particle shape can be defined by the shape factor, S_F, (USACE, 1995) as:

$$S_F = \frac{c}{\sqrt{ab}} \tag{11.2}$$

where:

a = the length of the longest axis,

b = the length of the intermediate axis,

c = the length of the shortest axis.

The shape factor for a sphere is 1.0. Natural sediment typically has a shape factor of about 0.7 (USACE, 1995). Other shape factors and parameters describing the sediment particle shape can be found in Chien and Wan (1998).

Particle Specific Gravity and Specific Weight

The sediment particle specific gravity is defined as the ratio of the specific weight of the sediment particle to the specific weight of water at 4°C. Sediment particle specific gravity ranges from 2.6 to 2.8 for natural soils (USACE, 1995). Since the specific weight for water is 62.4 lb/ft^3, the sediment particle specific weight ranges from 162.24 lb/ft^3 (=2.6*62.4) to 174.72 lb/ft^3 (=2.8*62.4). Quartz is the most common mineral found in sediments. Quartz has a specific gravity of 2.65 and a specific weight of 165.36 lb/ft^3. For most applications a specific gravity of 2.65 can be assumed for sediment; however, the specific gravity can be less or much higher for heavy minerals.

Particle Fall Velocity

Sediment particle fall velocity is one of the most important parameters, which describes the particle in relation to the fluid. The fall velocity is a function of Reynolds number, shape factor, proximity of the boundary, concentration, specific gravity, and turbulence (Garde and Raju, 1985). Under gravitational force's influence, a spherical particle will ultimately attain a uniform velocity. This velocity is called the fall velocity or the terminal settling velocity (Graf, 1984). The standard fall velocity can be defined as the average rate of fall that the particle would finally attain if falling alone in quiescent distilled water of infinite extent and at a temperature of 24°C (USACE, 1995). The fall velocity of a sphere is given as follows (ASCE, 1975, 2006):

$$V_s^{\ 2} = \frac{4gd}{3C_D}\left(\frac{\gamma_s - \gamma}{\gamma}\right) \tag{11.3}$$

where:

V_s = the fall velocity or terminal settling velocity,

g = the gravitational acceleration,

d = the sphere diameter,

γ_s = the specific weight of the sphere,

γ = the specific weight of the fluid,

C_D = the drag coefficient.

The drag coefficient can be approximated (ASCE, 2008) by:

$$C_D = \frac{24}{Re}(1 + 0.152Re^{0.5} + 0.0151Re) \tag{11.4}$$

where:

Re = the Reynolds number and is equal to $V_s d / \nu$,

V_s = the fall velocity or terminal settling velocity,

d = the sphere diameter,

ν = the kinematic viscosity which can be approximated by:

$$\nu = \frac{0.00000179}{1 + 0.03368T + 0.00021T^2} \tag{11.5}$$

where:

ν = the kinematic viscosity of clear water in m^2/s,

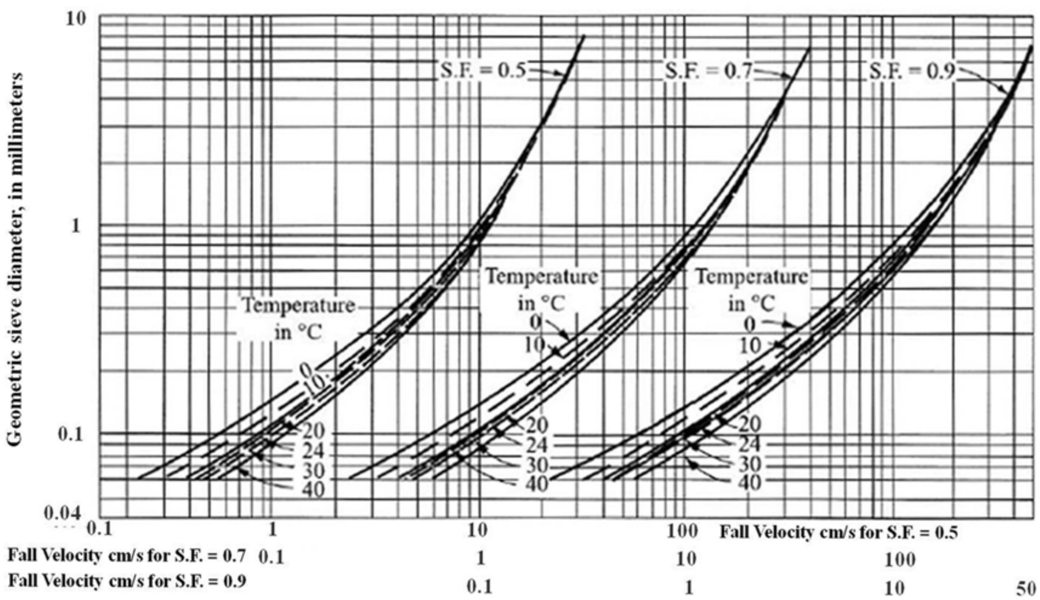
T = the temperature of the water in degrees centigrade.

When the Reynolds number is less than 0.1, C_D can be approximated by $24/Re$. Then, the settling velocity becomes the Stokes Law equation as follows:

$$V_s = \frac{gd^2}{18\nu}\left(\frac{\gamma_s - \gamma}{\gamma}\right) \tag{11.6}$$

For typical engineering sedimentation studies, the fall velocity can be estimated by Figure 11.6. In general, the drag coefficient decreases as the Reynolds number increases. When the Reynolds number for spheres is between 1,000 and 10,000, the drag coefficient is around 0.5 (Rouse, 1938). When Reynolds number is less than 0.1 (Stokes' range), the drag coefficient is independent of the thickness of the sediment particle as long as the particle is not relatively long (Garde and Raju, 1985). The shape affects the fall velocity significantly when Reynolds number is large. For the same Reynolds number, a larger shape factor will give a smaller drag coefficient. A discussion on fall velocity for nonspherical particles can also be found in ASCE (2008). The suspended sediment concentration also affects the fall velocity. Experiments show that fine sediments with a diameter less than 0.062 mm reduce the fall velocity (Simons, Richardson and Haushild, 1963). Shen and Julien (1993) indicated that heavy sediment concentrations reduce the fall velocity for a sediment particle. Open channel experiments indicate an increase in fall velocity due to turbulence (Garde and Raju, 1985). Generally speaking, the particle roughness will increase the drag coefficient. However, when the Reynolds number is below a certain-value, the influence of the roughness on the drag coefficient disappears (Graf, 1984). The effects on the fall velocity due to these and other factors are discussed in Graf (1984), Garde and Raju (1985), Shen and Julien (1993), and ASCE (1975, 2006).

FIGURE 11.6
RELATIONSHIP OF SIEVE DIAMETER AND FALL VELOCITY
FOR NATURALLY WORN QUARTZ PARTICLES
FALLING ALONE IN QUIESCENT DISTILLED WATER OF INFINITE EXTENT
(Interagency Committee, 1957)



11.6.1.2 Bulk Properties for Sediment Mixture or Deposit

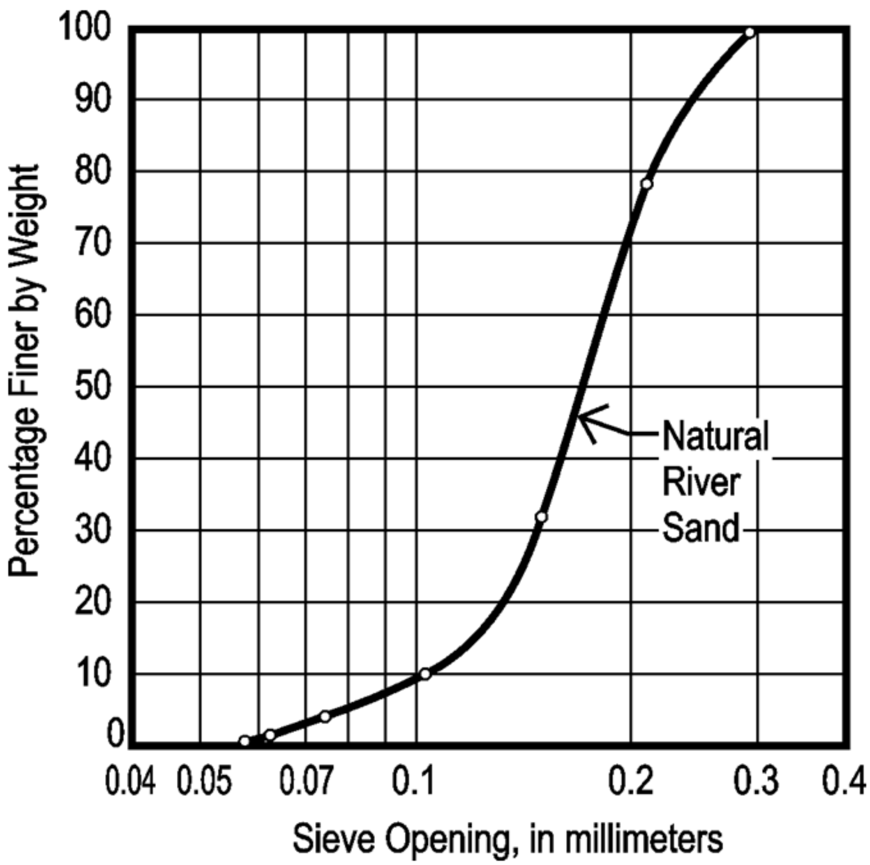
The basic bulk properties for a group of sediment particles are particle size distribution, porosity, bulk specific weight, and submerged angle of repose (Garde and Raju, 1985).

Particle Size Distribution

The size classes of sand and larger particles are typically obtained by laboratory sieve analyses of bulk samples (for fine sand to coarse gravel between 0.0625 mm and 32 mm) or by the pebble count technique (Wolman, 1954) for larger particle sizes. For silt and smaller particle sizes (less than 0.0625 mm), a fall velocity method is typically used. Size distribution or size gradation is usually presented as cumulative sediment size distribution, where the fraction or percentage by weight of sediment that is smaller than a given size is plotted against the size. This relationship is also called the sediment size gradation curve. Figure 11.7 is a typical size gradation graph. The data are typically plotted on a semi logarithmic graph. The median size, d_{50}, that is, the size for which 50 percent of the material is finer by weight, can be read from the curve. Other values of interest are d_5, d_{16}, d_{84} and d_{95}, defined similarly as d_{50}.

.

FIGURE 11.7
CUMULATIVE SEMILOGARITHMIC SIZE-GRADATION CURVE



The commonly used U.S. standard sieve sizes for gradation graph are 3 inches, 1.5 inches, ¾ inches, 3/8 inches, No. 4, No. 10, No. 20, No. 40, No. 60, No. 100, and No. 200. The sizes in mm for No. 4 through No. 200 are shown below in Table 11.2.

TABLE 11.2
U.S. STANDARD SIEVE SIZES
USACE (1970)

Commonly Used Sieve Size Number (U.S. Standard Sieve Size, ASTM E-11-6)	Size (mm)
No. 4	4.76
No. 10	2.00
No. 20	0.841

TABLE 11.2
U.S. STANDARD SIEVE SIZES
USACE (1970)

Commonly Used Sieve Size Number (U.S. Standard Sieve Size, ASTM E-11-6)	Size (mm)
No. 40	0.420
No. 60	0.25
No. 100	0.149
No. 200	0.074

The sediment arithmetic mean size d_a is the average value of the size for each sediment particle. Sediment samples from natural rivers often follow log-normal distribution (the log values of the sediment size follows the normal distribution or Gaussian distribution). When the sediment sizes follow a log-normal distribution, the sediment geometric mean, d_g, and sediment median, d_{50}, are equal, and they can be estimated (Mays, 1999) by:

$$d_g = d_{50} = (d_{84}d_{16})^{1/2} \tag{11.7}$$

While the median and geometric mean are used to measure the average size of the sediment mixture, the geometric standard deviation or gradation coefficient is used to measure the size variation of the sediment mixture. When the sediment size follows a log-normal distribution, the geometric standard deviation σ_g is estimated as (Mays, 1999):

$$\sigma_g = (d_{84}/d_{16})^{1/2} = d_{84}/d_{50} = d_{50}/d_{16} = 0.5(d_{84}/d_{50} + d_{50}/d_{16}) \tag{11.8}$$

The term $0.5(d_{84}/d_{50} + d_{50}/d_{16})$ is often called the gradation coefficient. For more discussion on other statistical parameters for a sediment mixture, see Garde and Raju (1985), Shen and Julien (1993), and ASCE (2008).

Porosity

The porosity of a sediment deposit is defined as the percentage of pore space in the total bulk volume of the sediment (Shen and Julien, 1993):

$$P = V_v/V_t \tag{11.9}$$

where:

P = the porosity,

V_v = the void volume,

V_t = the total volume of the sample.

Fine sediment particles usually have more voids than coarse particles because the surface area of fine particles is relatively larger (Chien and Wan, 1998). The porosity ranges for fine sand, medium sand, and coarse sand are 44%-49%, 41%-48%, and 39%-41%, respectively. The porosity for sandy soils with a small amount of clay may be between 50% and 54%. The porosity of a natural sediment mixture is between 25% and 50%.

Bulk Specific Weight

The bulk specific weight is the specific weight for a sediment deposit. It is defined as the dry weight of the sediment deposit divided by its bulk volume:

$$\gamma_b = W_s / V_t \tag{11.10}$$

where:

γ_b = the bulk specific weight,

W_s = the dry weight of the sediment deposit,

V_t = the bulk volume.

The bulk volume is equal to the sediment volume plus the void volume. The bulk specific weight can be derived and expressed as a function of porosity and particle specific weight by:

$$\gamma_b = (1 - P)\gamma_s \tag{11.11}$$

where:

γ_b = the bulk specific weight,

P = the porosity,

γ_s = the sediment particle specific weight = $SG * \gamma$,

where:

SG = sediment particle specific gravity,

γ = 62.4 lb/ft^3.

Since there are voids among the particles in a sediment deposit, the specific weight for sediment deposits is always less than that for a single sediment particle. This can be observed from the above equation since P is less than 1.0 and greater than or equal to 0.0. Table 11.3 lists the initial specific weights for sediments that have been in deposits for 1 year or less (ASCE, 2006). Table 11.3 also contains the approximated D_{50} values when only the data range or D_{90} was given in the original table from ASCE (2006). Figure 11.8 shows the initial specific weights as a function of D_{50} based on-values from Table 11.3. A regression equation was developed by FCDMC to fit the data points in Figure 11.8 as given below:

$$\gamma_{initial} = 100.5 + 20.44 * \log_{10}(D_{50}) \tag{11.12}$$

where:

$\gamma_{initial}$ = the initial bulk specific weight for a sediment deposit in lb/ft^3,

D_{50} = the median sediment size in mm.

The applicable range for D_{50} is between 0.0005 mm and 180 mm. The solid line in Figure 11.8 is the fitted line that corresponds to the regression equation.

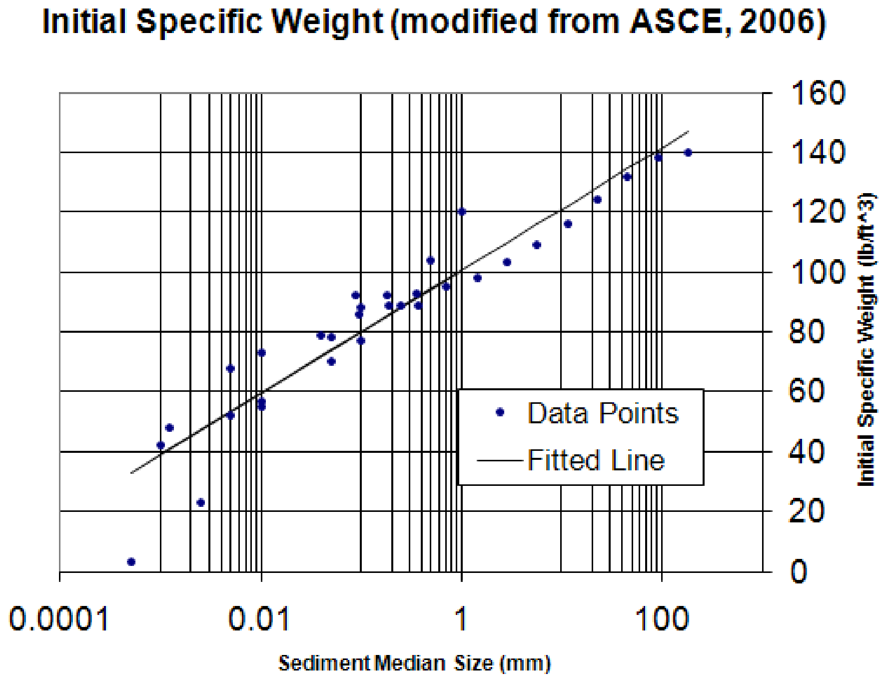
.

TABLE 11.3
INITIAL BULK SPECIFIC WEIGHTS OF SEDIMENTS OF VARIOUS GRAIN SIZE
(MODIFIED FROM ASCE, 2006)

California Division of Water Resources			Trask (Laboratory)			Hembree, Colby, Swenson, and Davis (Countrywide)		Happ (Middle Rio Grande)	
D_{90} (mm)	D_{50} (mm)	Initial Specific Weight (lb/ft^3)	Size Range (mm)	D_{50} (mm)	Initial Specific Weight (lb/ft^3)	D_{50} (mm)	Initial Specific Weight (lb/ft^3)	D_{50} (mm)	Initial Specific Weight (lb/ft^3)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
0.125	0.089	92	---	---	---	---	---	---	---
0.25	0.179	92	---	---	---	---	---	---	---
0.5	0.357	93	---	---	---	---	---	---	---
1	0.714	95	---	---	---	---	---	---	---
2	1.429	98	0.001-0	0.0005	3	0.001	42	0.0012	48
4	2.857	103	0.004-0.001	0.0025	23	0.005	52	0.005	68
8	5.714	109	0.016-0.004	0.01	55	0.01	57	0.01	73
16	11.429	116	0.064-0.016	0.04	79	0.05	70	0.05	78
32	22.857	124	0.125-0.064	0.0945	86	0.1	77	0.1	88
64	45.714	132	0.25-0.125	0.1875	89	0.25	89	---	---
128	91.429	138	0.5-0.25	0.375	89	0.5	104	---	---
256	182.857	140	---	---	---	1.0	120	---	---

Notes: Column (2) is obtained by dividing column (1) by 1.4. 1.4 is obtained by interpolating the middle values of stone size ranges for 100 and 85 based on Table 6.4 from the FCDMC Hydraulics Manual (1996). Column (5) is the averaged values of the lower and upper bounds from Column (4).

FIGURE 11.8
INITIAL BULK SPECIFIC WEIGHT FOR SEDIMENT DEPOSITS
(MODIFIED FROM ASCE, 2006)



As indicated in ASCE (1975, 2006), three factors influence the specific weight of a deposit. They are mechanical composition, the environment in which deposits are formed, and time. Coarse materials may not change with time, but fine materials such as clay and silt may have an initial specific weight which is much smaller than the specific weight for the ultimately consolidated deposit over time. The initial bulk specific weight for sediment deposits will increase as the sediment deposit is being consolidated over time. The bulk specific weight for reservoir sediment deposits after T years can be expressed (ASCE, 2006) as:

$$\gamma_T = \gamma_{res,\ initial} + B * \log_{10} T \qquad (11.13)$$

where:

γ_T	=	the consolidated bulk specific weight in lb/ft^3,
$\gamma_{res,\ initial}$	=	the reservoir sediment bulk specific weight for the initial deposit after 1 year of consolidation,
B	=	the consolidation coefficient,
T	=	time in years.

Table 11.4 lists the consolidation coefficients for sand, silt, and clay. When the sediment deposit contains sand, silt, and clay, the above equation can be used to find the individual consolidated sediment specific weight after T years for each. Then, the composite specific weight can be estimated by Colby's equation (ASCE, 2006) as:

$$\gamma_{Comp,T} = \frac{1}{\frac{F_{clay}}{\gamma_{clay,T}} + \frac{F_{silt}}{\gamma_{silt,T}} + \frac{F_{sand}}{\gamma_{sand,T}}} \quad (11.14)$$

where:

$\gamma_{Comp,T}$ = the composite specific weight after T years,

$\gamma_{clay,T}$ = the bulk specific weight for clay after T years,

$\gamma_{silt,T}$ = the bulk specific weight for silt after T years,

$\gamma_{sand,T}$ = the bulk specific weight for sand after T years,

F_{clay} = the volume of clay in percent (as a decimal),

F_{silt} = the volume of silt in percent (as a decimal),

F_{sand} = the volume of sand in percent (as a decimal).

TABLE 11.4
CONSOLIDATION COEFFICIENTS FOR SPECIFIC WEIGHT OF RESERVOIR SEDIMENTS
(MODIFIED FROM ASCE, 2006)

Reservoir Operation	Sand		Silt		Clay	
	$\gamma_{res,\ initial}$	B	$\gamma_{res,\ initial}$	B	$\gamma_{res,\ initial}$	B
Sediment always submerged or nearly submerged	93	0	65	5.7	30	16
Normally a moderate reservoir drawdown	93	0	74	2.7	46	10.7
Normally considerable reservoir drawdown	93	0	79	1	60	6
Reservoir normally empty	93	0	82	0	78	0

Submerged Angle of Repose

The submerged angle of repose is the angle beyond which the slope formed by the submerged sediment particles will start to slide. The angle is with respect to the horizontal plane. As indicated by ASCE (2008), the submerged angle of repose is an empirical quantity ranging from 30^o for sand to 40^o for gravel. Chien and Wan (1998) showed a graph of submerged angle of repose as a function of sediment size for particles ranging from very sharp edges to perfectly round shape. The submerged angle of repose ranges from 20 degrees to 42 degrees for sediment size in the range of 0.5 mm to 10 mm. Sharp edge particles have a submerged angle of repose 5-10 degrees higher than round particles. The submerged angle of repose for a group of sediment particles with the same size may be estimated (Wu, 2007) as follows:

$$\phi = 32.5 + 1.27d \tag{11.15}$$

where:

ϕ = the submerged angle of repose in degrees,

d = the sediment particle diameter in mm (the applicable range for d is between 0.2 mm and 4.4 mm).

11.6.1.3 Properties for Water-Sediment Mixtures

The basic properties for water-sediment mixtures to be discussed herein are sediment concentration, sediment discharge, and sediment load.

Sediment Concentration

There are several definitions for sediment concentration. The first definition, $C_{w/w}$, is the ratio of the weight of the sediment to the weight of the water-sediment mixture. The second definition is the first definition in parts per million, *ppm*, denoted by C_{ppm}, i.e., the dry weight of sediment divided by the weight of the water-sediment mixture multiplied by 1 million for convenience in the laboratory (Porterfield, 1972). The third definition, $C_{w/v}$, is the weight of dry sediment in a water-sediment mixture per volume of mixture. A fourth definition is the ratio of the sediment volume to the water-sediment mixture, denoted by $C_{v/v}$. The following equations show the relationships among these definitions:

$$C_{w/w} = \frac{SG_s{*}C_{v/v}}{1 + (SG_s - 1)C_{v/v}} \tag{11.16}$$

$$C_{ppm} = 10^6 C_{w/w} \tag{11.17}$$

$$C_{w/v} = \gamma_b C_{w/w} \tag{11.18}$$

where:

SG_s = the specific gravity for sediment particles,

γ_b = the bulk specific weight.

A special case of $C_{w/v}$ is milligrams per liter, $C_{mg/l}$. The relationship between $C_{mg/l}$ and C_{ppm} can be expressed as:

$$C_{mg/l} = \frac{SG_s * C_{ppm}}{SG_s + (1 - SG_s)C_{ppm}10^{-6}} \tag{11.19}$$

$$C_{ppm} = \frac{SG_s * C_{mg/l}}{SG_s + (SG_s - 1)C_{mg/l}10^{-6}} \tag{11.20}$$

The numerical values of C_{ppm} and $C_{mg/l}$ are similar when the concentration is less than 16,000 *mg/l* because the water-sediment mixture has a density close to water density, 1 *mg/ml*.

The sediment concentration can be used to classify flows (O'Brien, 1986), as illustrated in Table 11.5.

TABLE 11.5
SEDIMENT FLOW CLASSIFICATION BASED ON CONCENTRATION
(O'Brien, 1986)

Type of Flow	Concentration Range (in) (mg/l)
(1)	(2)
Water flood	0 - 410,000
Mud flood	410,000 - 650,000
Mudlflow	650,000 - 730,000
Landslide	730,000 - 880,000

Sediment Load

The term sediment load is used to denote sediment transport rate, and the dimension of sediment load can be expressed either as weight per unit time or as volume per unit time (Shen and Julien, 1993). Herein, the terms sediment load and sediment discharge are used interchangeably. Sediment load is often expressed in tons/day. The relationship between $C_{mg/l}$ and suspended sediment discharge can be expressed as:

$$Q_s = 0.0027 C_{mg/l} Q_w \tag{11.21}$$

where:

Q_s = the sediment discharge in tons/day,

$C_{mg/l}$ = the sediment concentration for the suspended sediment,

Q_w = the water discharge in cfs.

It may be noted that the above equation is based on an assumption that the weight of a cubic foot of water-sediment mixture is 62.4 pounds (Porterfield, 1972).

The sediment discharge in tons per day can be converted to cubic feet per second by:

$$Q_{s,cfs} = 0.02315 Q_s / \gamma_s \tag{11.22}$$

where:

$Q_{s,cfs}$ = the sediment discharge in cubic feet per second,

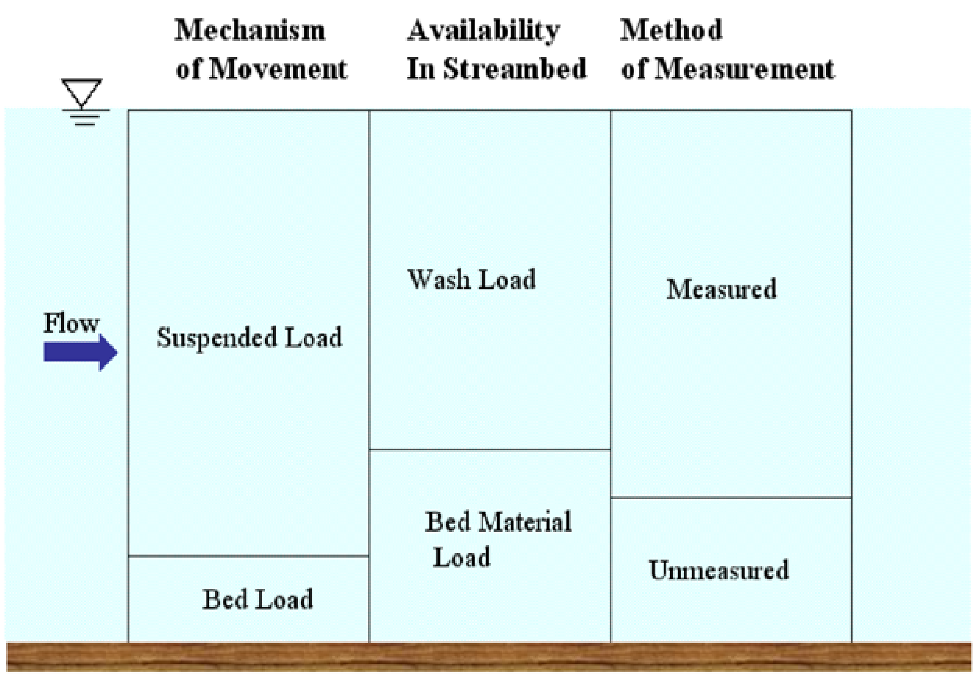
γ_s = the specific weight of the sediment particles in lb/ft^3 (USACE, 1995).

Total sediment load can be defined as the sediment transport load for an entire cross section. Based on the mechanism of movement, total sediment load can be divided into bed load and suspended load (Shen and Julien, 1993). The bed load is the sediment load transported close to the bed where particles move intermittently by rolling, sliding, or jumping (USACE, 1995). The suspended load is the sediment load for the sediment particles that are supported by the turbulent motion in the stream flow.

Based on its availability in the streambed, total sediment load can be divided into bed-material load and wash load. Bed material load is the sediment transport load for those sediment particles sizes that are readily apparent on the surface of the streambed (Shen and Julien, 1993). Particles that move as suspended load or bed load and periodically exchange with the bed are part of the bed material load (USACE, 1995). The bed material load can be computed from the composition of the streambed. Wash load consists of the finest particles in the suspended load that are continuously maintained in suspension by flow turbulence. The D_{10} or D_5 sizes from the bed material size distribution can be used to separate the wash load from the bed material load (Shen and Julien, 1993). D_{10} is defined as the sediment size on the streambed surface for which 10 percent is finer. D_5 is defined as the sediment size on the streambed surface for which 5 percent is finer. Wash load may be calculated based on upstream supply rate, watershed sediment yield analysis, or actual sediment rate measurement (Shen and Julien, 1993). According to the method of calculation, bed material load can be divided into suspended bed material load and bed load. The suspended load can be divided into wash load and suspended bed material load.

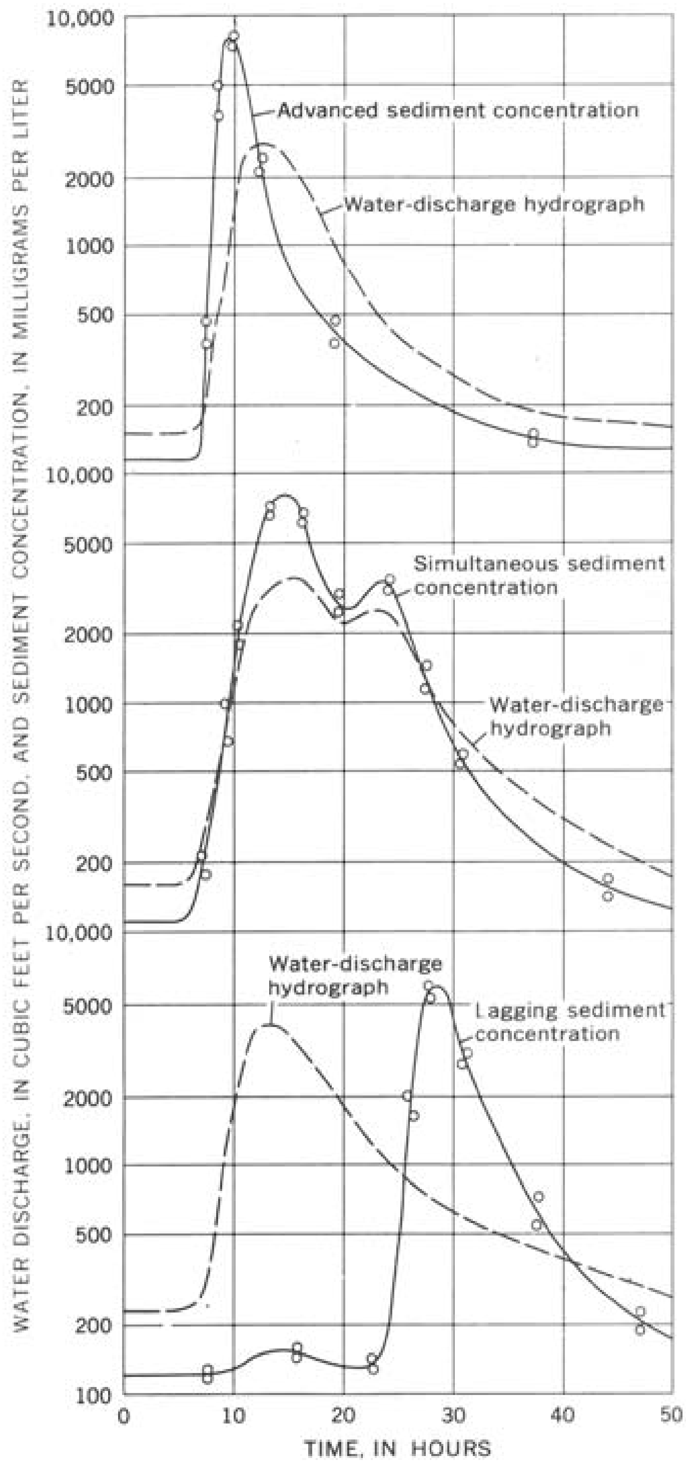
Based on the measurement method, sediment can be divided into measured and unmeasured. When depth-integrated suspended sediment samplers are used, the lower 0.5 ft of the water column is unmeasured. The unmeasured load includes some of the suspended sediment and usually all of the bed load (USACE, 1995). Figure 11.9 shows the total sediment load definitions by the mechanisms of sediment movement, sediment availability in the streambed, and the method of measurement. Procedures for measurement of fluvial sediment are provided by Guy and Norman (1970). Methods for laboratory analyses of sediment samples are provided by Guy (1969).

FIGURE 11.9
TOTAL SEDIMENT LOAD DEFINITION BY DIFFERENT CLASSIFICATIONS
Guy (1989)



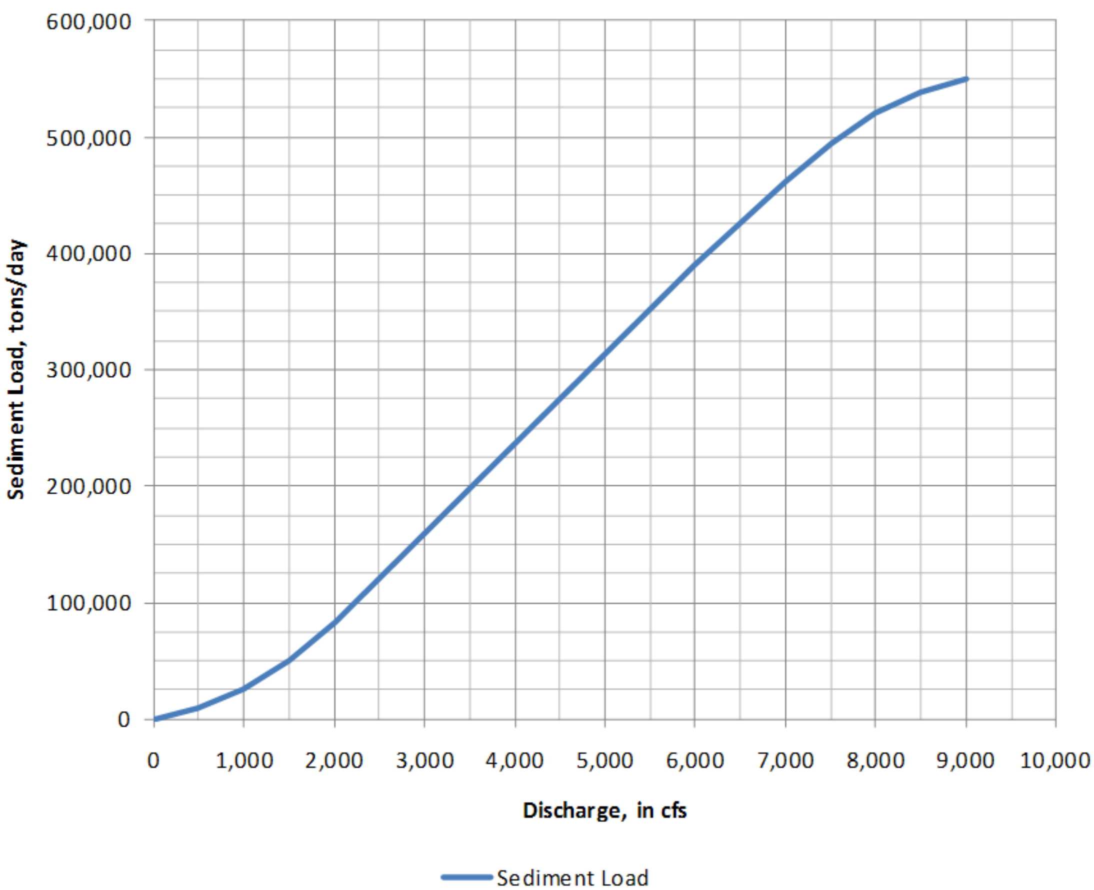
At a particular cross section of a river, the peak of the sediment concentration may coincide with the peak of the water-discharge, but it may also lag behind or advance in front of the flow peak. Generally speaking, when the travel distance from the erosion source is short, the peak sediment concentration usually coincides with the peak flow or somewhat precedes it (Guy, 1989). When the travel distance from the erosion source is large, the peak of the sediment concentration may lag behind the peak of the flow. Figure 11.10 from Guy (1989) shows these three types of relationships.

FIGURE 11.10
SEDIMENT CONCENTRATION AND WATER DISCHARGE HYDROGRAPH
(Guy, 1989)



In Maricopa County, there is a lack of adequate sediment discharge and corresponding storm runoff data. Therefore, it is often necessary to estimate sediment yield from the watershed by analytic or empirical methods. Techniques of sediment transport modeling, such as HEC-6 (USACE, 1991), are often used in such analyses. Sediment inflow relations are generally required and those are often in the form of sediment load rating curves of sediment discharge (tons per day) as a function of water discharge (cfs). Figure 11.11 gives an example of a sediment rating curve.

FIGURE 11.11
EXAMPLE OF A SEDIMENT DISCHARGE RATING CURVE



Sediment Data

Figure 11.12 shows suspended sediment concentration and sediment discharge in rivers of the conterminous United States in 1980 (Meade and Parker, 1984). The darker brown color on the map represents higher sediment concentration (*mg/l*). The size of the semicircles is proportional to the sediment volume. The numerical values next to the semicircles are the sediment load (in million tons per year) that is discharged into the coastal zone.

FIGURE 11.12
SEDIMENT CONCENTRATION AND LOAD IN US FOR 1980
(FROM Meade and Parker, 1984)

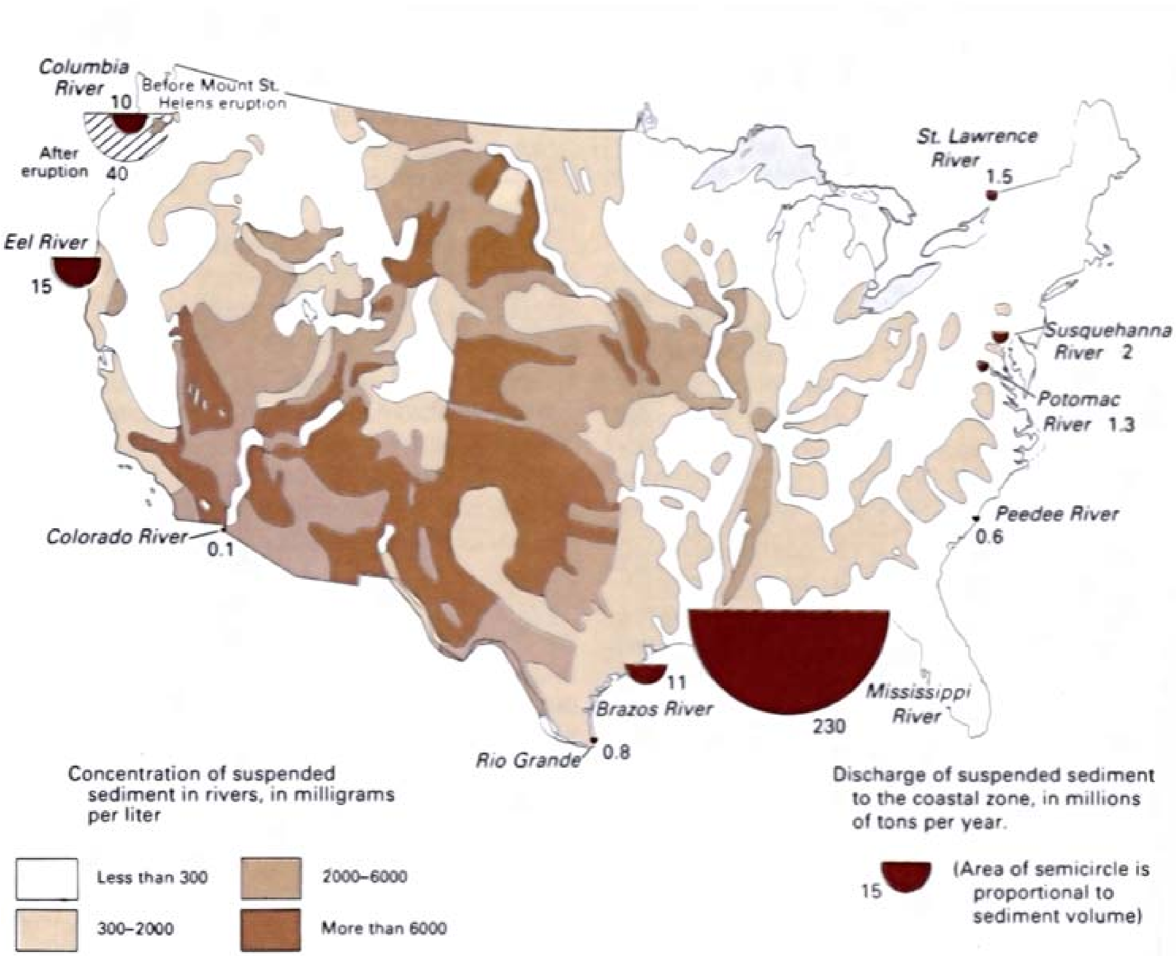


Table 11.6 lists the suspended sediment to the coastal zone by 10 major rivers of the United States for the year 1980 (Meade and Parker, 1984). As can be seen, the Mississippi River generates the largest volume of sediment. Figure 11.13, Figure 11.14, and Figure 11.15 show the historically measured suspended sediment concentration for the Mississippi River, the Colorado River, and the Gila River, respectively. Figure 11.16 and Figure 11.17 show annual suspended sediment amount in metric tons for the Colorado River and the Gila River. Figure 11.18 shows a few measured suspended sediment transport rates in tons/day for several rivers near Maricopa County, Arizona in 2005.

TABLE 11.6
SUSPENDED SEDIMENT LOAD FOR 10 MAJOR RIVERS IN US IN 1980
(Meade and Parker, 1984)

Rivers	Average annual sediment load (million ton/yr)
(1)	(2)
Mississippi	230[1]
Copper	80
Yukon	65
Susitna	25
Eel	15
Brazos	11
Columbia (before Mount St. Helens eruption)	10
Columbia (since Mount St. Helens eruption)	40
St. Lawrence	1.5
Rio Grande	0.8
Colorado	0.1

Note: [1] Includes Atchafalaya River

FIGURE 11.13
MEASURED SUSPENDED-SEDIMENT CONCENTRATION (MG/L) FOR MISSISSIPPI RIVER
(http://co.water.usgs.gov/sediment/)

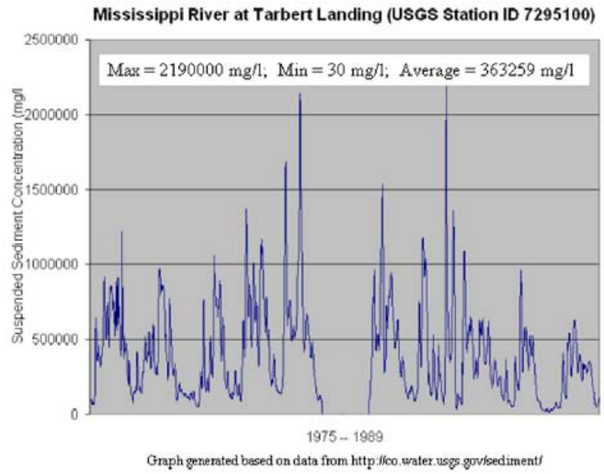


FIGURE 11.14
MEASURED SUSPENDED-SEDIMENT CONCENTRATION (MG/L) FOR COLORADO RIVER
(http://co.water.usgs.gov/sediment/)

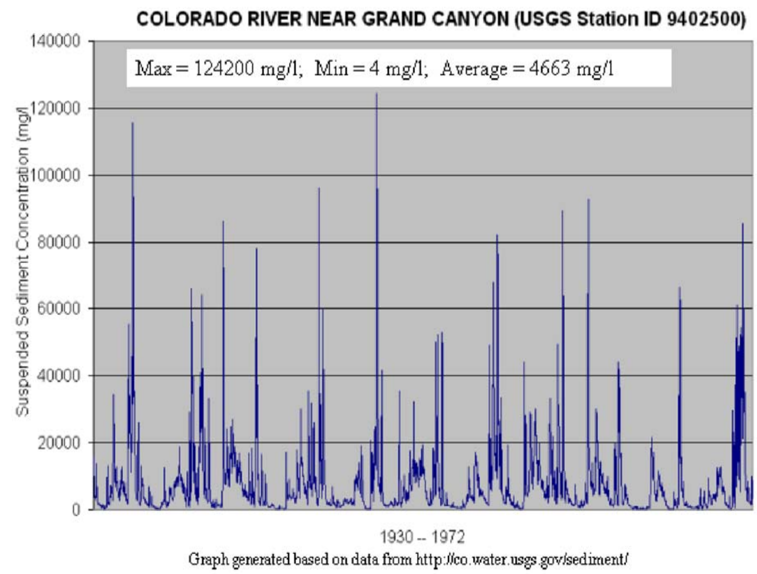


FIGURE 11.15
MEASURED SUSPENDED-SEDIMENT CONCENTRATION (MG/L) FOR GILA RIVER
(http://co.water.usgs.gov/sediment/)

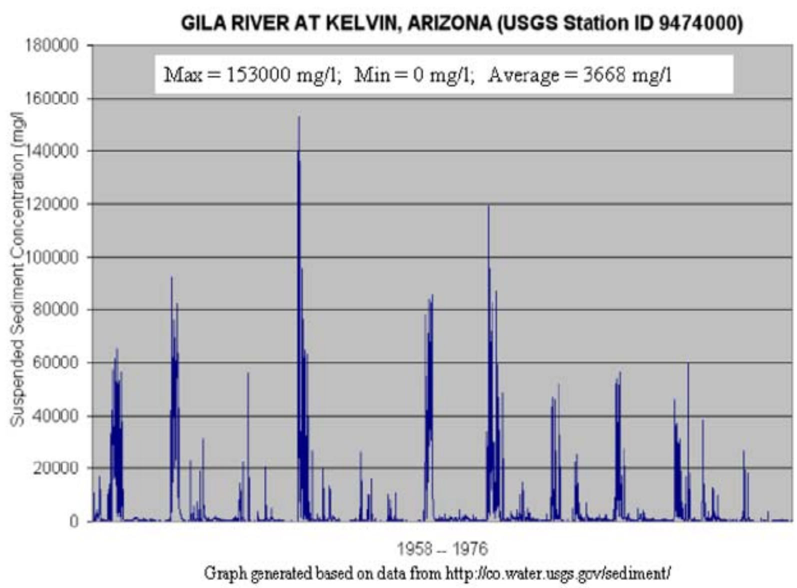


FIGURE 11.16
ANNUAL SUSPENDED-SEDIMENT DISCHARGE IN METRIC TONS FOR COLORADO RIVER
(http://co.water.usgs.gov/sediment/images/az10.gif)

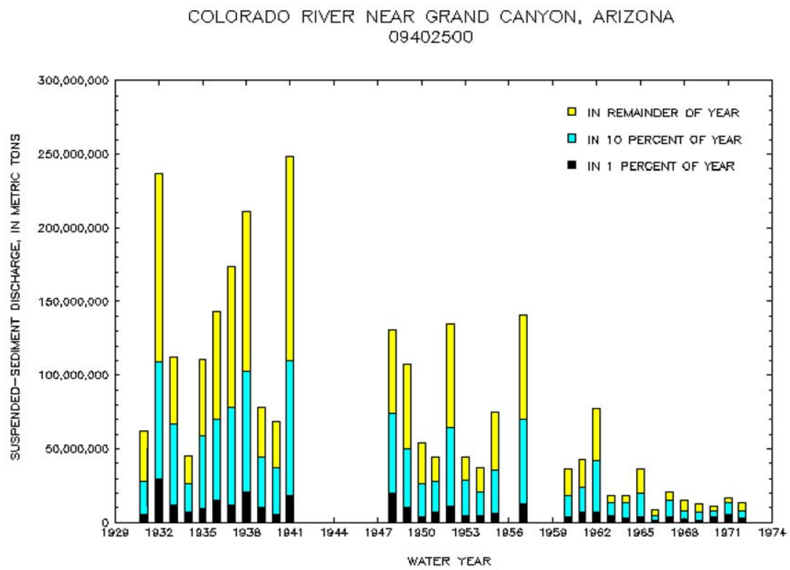


FIGURE 11.17
ANNUAL SUSPENDED-SEDIMENT DISCHARGE IN METRIC TONS FOR GILA RIVER
(http://co.water.usgs.gov/sediment/images/az20.gif)

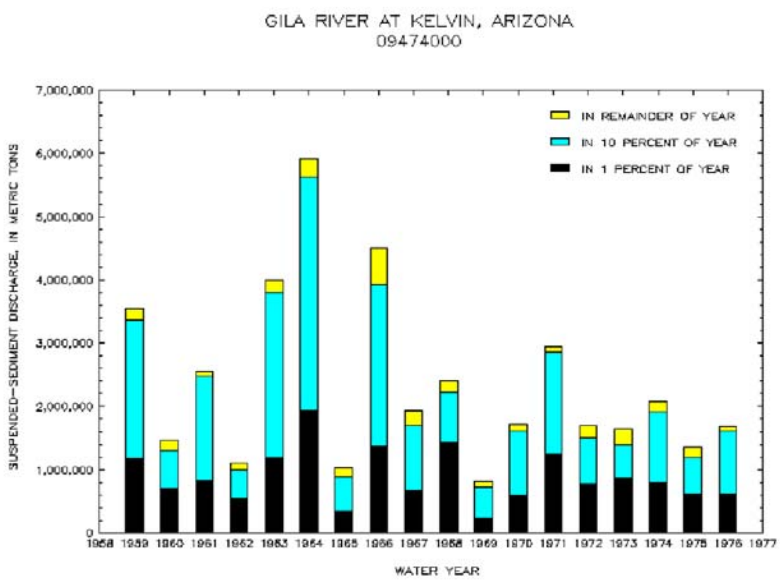
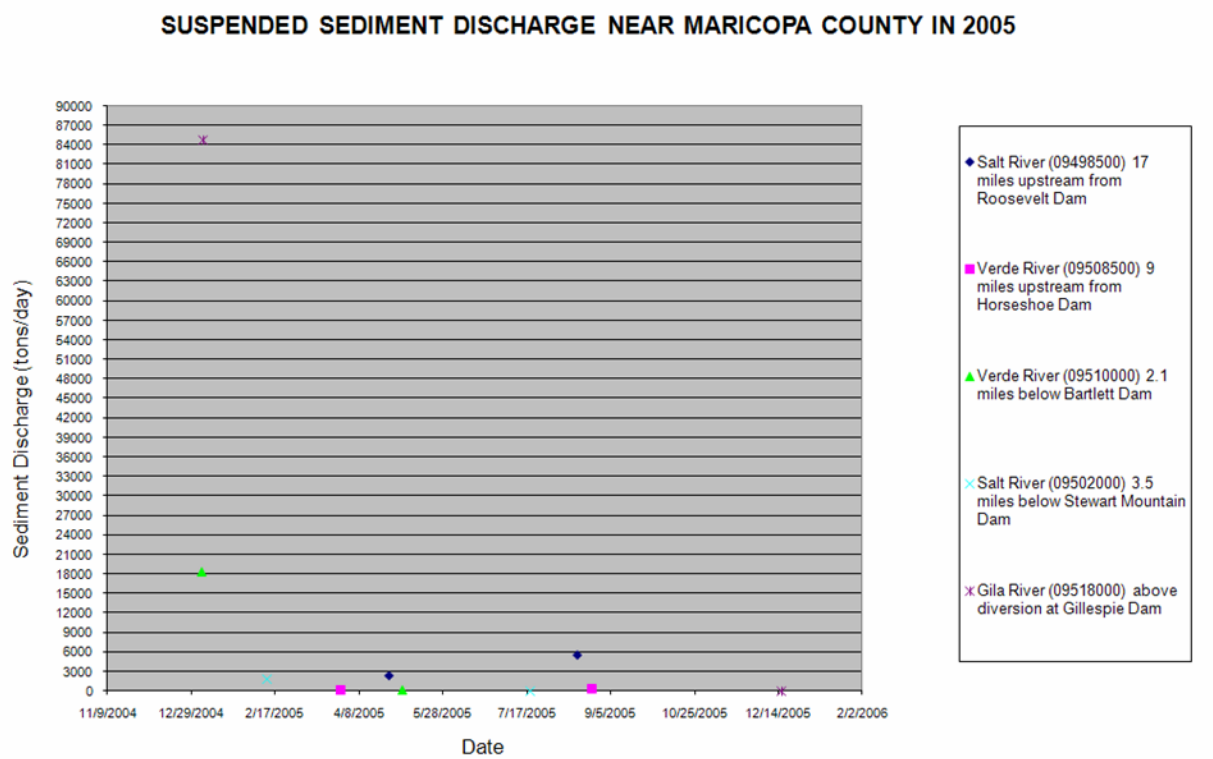


FIGURE 11.18
SUSPENDED SEDIMENT DISCHARGE NEAR MARICOPA COUNTY IN 2005
GRAPH GENERATED BASED ON DATA FROM USGS WATER-DATA REPORT AZ-05-1 (Fisk et al, 2005)



11.6.2 Incipient Motion

Incipient motion occurs when the hydrodynamic forces acting on a grain of sediment of a given size is equal to the forces resisting movement. Incipient motion is often analyzed using the Shields relation (Mussetter et al., 1994).

$$d_c = \frac{\tau_o}{F_*(\gamma_s - \gamma)} \tag{11.23}$$

where:

d_c = the sediment diameter at incipient motion, in feet,

τ_o = the bed shear stress, in pounds per square foot,

γ_s = the sediment specific weight, typically 165 pounds per cubic foot,

γ = the specific weight of water, 62.4 pounds per cubic foot,

F_* = the dimensionless shear stress, often referred to as the Shields parameter.

F_* ranges from 0.03 to 0.06. A value of 0.047 is suggested by Meyer-Peter and Muller (1948).

The bed shear stress in pounds per square foot, is calculated by:

$$\tau_o = \gamma RS \tag{11.24}$$

where

γ = the specific weight of water, 62.4 pounds per cubic foot,

R = the hydraulic radius, in feet,

S = the channel bed slope, in ft/ft.

Incipient analysis, as presented herein, does not cover all aspects of incipient motion. For a discussion of applications, limitations and modifications see (Mussetter et al. (1994), ASCE (1975, 2006), USDOT (2001d), Simons and Senturk (1992), Yang (1973), Simons, Li and Associates (1985), Chang (1988b), and Shen (1971, 1972, 1973).

Application of incipient motion analysis may provide information on the magnitude of discharge required to move the particles lining the watercourse bed and/or the banks. These analyses are generally most reliable and useful for gravel or cobble-bed watercourses. When applied to sand-bed systems, incipient motion results usually show that the sediment particles are in motion, even at small discharges.

11.6.3 Armoring

Armoring occurs when material finer than the incipient motion size is eroded and transported away leaving a layer of coarser, immobile (for a given discharge) material on the surface. Armoring will occur in time when the channel bed downstream from a dam contains more than 10 percent coarse material which can not be transported under dominant flow conditions (Pemberton and Lara, 1984). If the watercourse is degrading, this process can continue over a range of discharge events. Each subsequent larger event removes increasingly larger particle sizes. Armoring is effective only to a given magnitude of flood event, flows exceeding that magnitude may disrupt the armor layer causing bed scour and degradation.

Armoring analysis normally requires the application of incipient motion analysis to determine the critical sediment particle size, d_c. The Shields relation can be used to estimate the sediment critical particle size. Four other methods for estimating the sediment critical particle size, d_c, are given in Pemberton and Lara (1984). It should be cautioned that these methods may give different results. Engineering judgment must be used to select the most appropriate method(s) for estimating the critical particle size. Once d_c is estimated, the decimal percentage of original bed material larger than d_c is determined based on the bed material gradation curve. The scour depth, Z_s, necessary to establish an armor layer can be estimated by Pemberton and Lara (1984) as follows:

$$Z_s = Y_a\left(\frac{1.0}{P_c} - 1.0\right) \quad (11.25)$$

where:

P_c = the decimal percentage (fraction) of bed material coarser than the critical particle size,

Y_a = the thickness of the armor layer which is either 3*d_c or 0.5 ft, whichever is smaller (Pemberton and Lara, 1984).

This scour depth may be considered as the long-term scour depth.

11.6.4 Sediment Bulking

High sediment concentrations can increase the total volume of the water and sediment discharge. This is referred to as bulking, and the total volume of the water-sediment mixture, V_m, is estimated by (Mussetter et. al., 1994):

$$V_m = B_f V_w \quad (11.26)$$

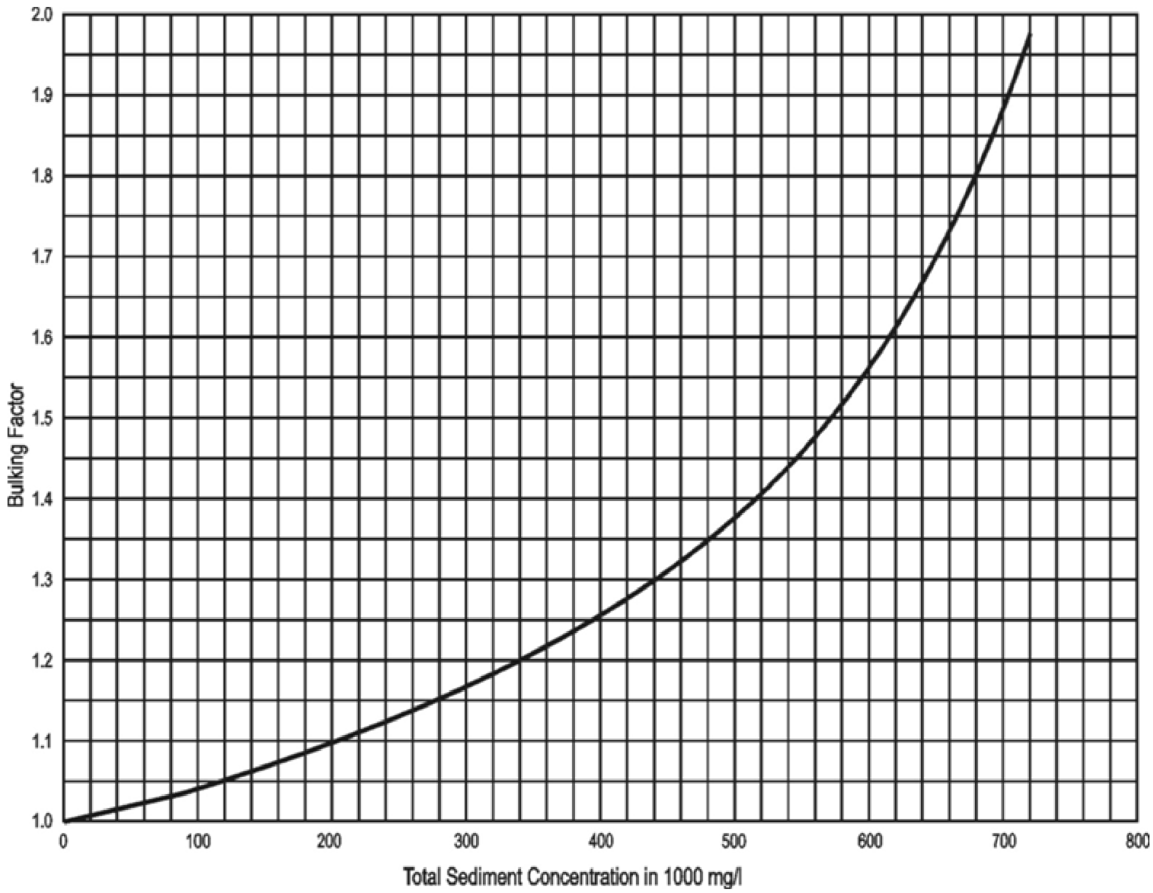
where:

B_f = the bulking factor,

V_w = the clear-water volume.

The relationship between total sediment concentration and the bulking factor is given by Figure 11.19. For example, if the sediment load concentration is 200,000 mg/l, the total water-sediment volume discharge is increased by a factor of about 1.10. For high sediment concentration discharges, design capacities should, based on engineering judgement, accommodate the bulked volumetric discharge.

FIGURE 11.19
RELATIONSHIP BETWEEN TOTAL SEDIMENT CONCENTRATION AND BULKING FACTOR
(Mussetter et al., 1994)



11.6.5 Sediment Transport Functions

The planning and design of drainage and flood control facilities usually requires the analysis of sediment transport using sediment transport functions. The sediment transport functions may be theoretical, empirical, or a combination of both. Some of the more popular sediment transport functions are the Einstein bed load function, the Meyer-Peter and Muller equation, the Yang unit stream power concept and the Colby relations. However, there are virtually dozens of sediment transport functions in the literature. One of the challenges for engineers is to select an appropriate function to solve a particular problem. When selecting a sediment transport function, the original data (sediment size, flow condition, mode of transport process, etc.) used to develop each method must be understood. The selection, however, is not straightforward and often it is not possible to determine which one is best for a particular application. Often the selection process indicates that no one function is best and two or more functions may need to be used and the respective results evaluated. The results by different functions often differ drastically. It is absolutely imperative that the application and limitation of the various transport functions be understood when using the equations to estimate sediment transport. The engineer must use experience and judgment in both the selection of the sediment transport function and in the interpretation of the results. Yang (2003) gave a detailed discussion on different sediment transport equations. Thomas et. al. (2002) gave the data range that was used to develop sediment transport equations, see (Mussetter et al. (1994), ASCE (1975, 2006), Yang (1973, 2003), Simons, Li and Associates (1985), Chang (1988b), USDOT (2001d), Shen (1971, 1972, 1973), Sheppard (1960), and Simons and Senturk (1992) for further discussions of sediment transport functions.

11.7 WATERSHED SEDIMENT YIELD

11.7.1 Introduction

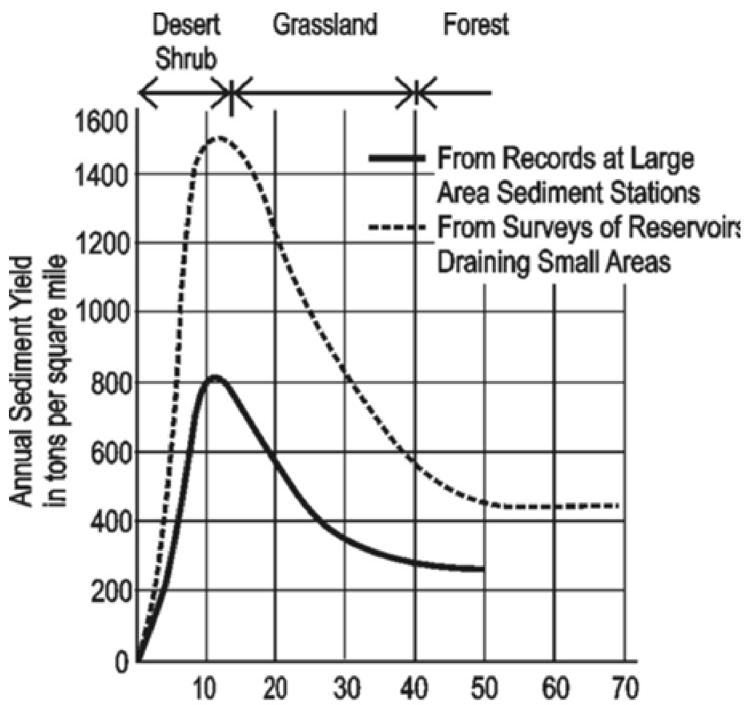
Sediment yield is a measure of the sediment production exiting a watershed at some point in the drainage network. It is usually measured in units of weight (tons), volume (acre-feet) or uniformly eroded depth of soil (inches or millimeters). The sediment yield is often expressed in terms of annual sediment yield (acre-feet per year or tons per year). When a flood for a specific flood return period is of interest, the sediment yield for that return period is estimated. Sometimes sediment yield per year per square mile is used.

Sediment yield is dependent upon the rate of total erosion within the watershed and the efficiency of transport of the eroded sediments through the drainage network. See Photograph 11.10 for an example of sediment deposition upstream of a roadway embankment that reduces the drainage network efficiency. Erosion and transport factors are widely variable; therefore, measures of sediment yield are broadly generalized.

PHOTOGRAPH 11.10
DEPOSITION UPSTREAM OF ROADWAY EMBANKMENT/CULVERT

Sediment yield is highly dependent upon vegetation cover and precipitation. Langbein and Schumm (1958) illustrate in Figure 11.20 a trend of increasing sediment yield with increasing annual (effective) precipitation, until increased precipitation results in improved vegetation cover. Beyond that point, sediment yield then decreases with increasing precipitation. Maximum sediment yield occurs in the 8 to 15 inches of annual precipitation range. Notice in Figure 11.20 that the sediment yield is considerably higher when data for small watersheds is used. Smaller watersheds typically have higher unit sediment yields because of the influence of high intensity rainfalls that can impact the entire watershed.

FIGURE 11.20
SEDIMENT YIELD, AS AFFECTED BY CLIMATE
(Langbein and Schumm, 1958)



11.7.2 Sediment Yield Data

Sediment yield data for watersheds in Arizona, New Mexico and California that may be applicable to conditions in Maricopa County are shown in Table 11.7.

TABLE 11.7
MEASURED SEDIMENT YIELD FROM REPRESENTATIVE WATERSHEDS
(REPRESENTATIVE OF CONDITIONS TO BE EXPECTED IN MARICOPA COUNTY, AZ)

No.	Location	Drainage Area (sq. miles)	Sediment Yield (ac-ft/sq. mi./year)	Reference (see footnote)
(1)	(2)	(3)	(4)	(5)
1	Cave Creek Dam, AZ	121.00	0.24	A
2	Spookhill FRS, AZ	16.40	0.15	B
3	Saddleback FRS, AZ	30.00	0.08	B
4	Davis Tank, AZ	0.21	0.96	C

TABLE 11.7
MEASURED SEDIMENT YIELD FROM REPRESENTATIVE WATERSHEDS
(REPRESENTATIVE OF CONDITIONS TO BE EXPECTED IN MARICOPA COUNTY, AZ)

No.	Location	Drainage Area (sq. miles)	Sediment Yield (ac-ft/sq. mi./year)	Reference (see foot-note)
(1)	(2)	(3)	(4)	(5)
5	Kennedy Tank, AZ	0.97	0.27	C
6	Juniper Tank, AZ	2.00	0.29	C
7	Alhambra Tank, AZ	6.61	0.03	C
8	Black Hills Tank, AZ	1.14	0.68	C
9	Black Hills Tank, AZ	1.56	0.58	D
10	Mesquite Tank, AZ	9.00	0.03	C
11	Tank 76, AZ	1.17	0.21	C
12	Camp Marston, CA	1.59	0.14	B
13	Embudo Arroyo, NM	20.68	0.07	E
14	La Cueva, NM	8.00	0.05	E
15	Baca Arroyo, NM	11.55	0.34	E
16	North Pino Arroyo, NM	2.82	0.22	E
17	South Pino Arroyo, NM	9.33	0.13	E
18	Bear Arroyo, NM	15.50	0.12	E
19	Vinyard Arroyo, NM	0.98	0.28	E
20	Hahn Arroyo, NM	5.80	0.01	E
21	N. Diversion Channel, NM	101.01	0.21	E
		Average =	0.24	
		Median	0.21	
		AZ Average =	0.32	
		AZ Median =	0.24	

TABLE 11.7
MEASURED SEDIMENT YIELD FROM REPRESENTATIVE WATERSHEDS
(REPRESENTATIVE OF CONDITIONS TO BE EXPECTED IN MARICOPA COUNTY, AZ)

No.	Location	Drainage Area (sq. miles)	Sediment Yield (ac-ft/sq. mi./year)	Reference (see foot-note)
(1)	(2)	(3)	(4)	(5)

Table footnotes:

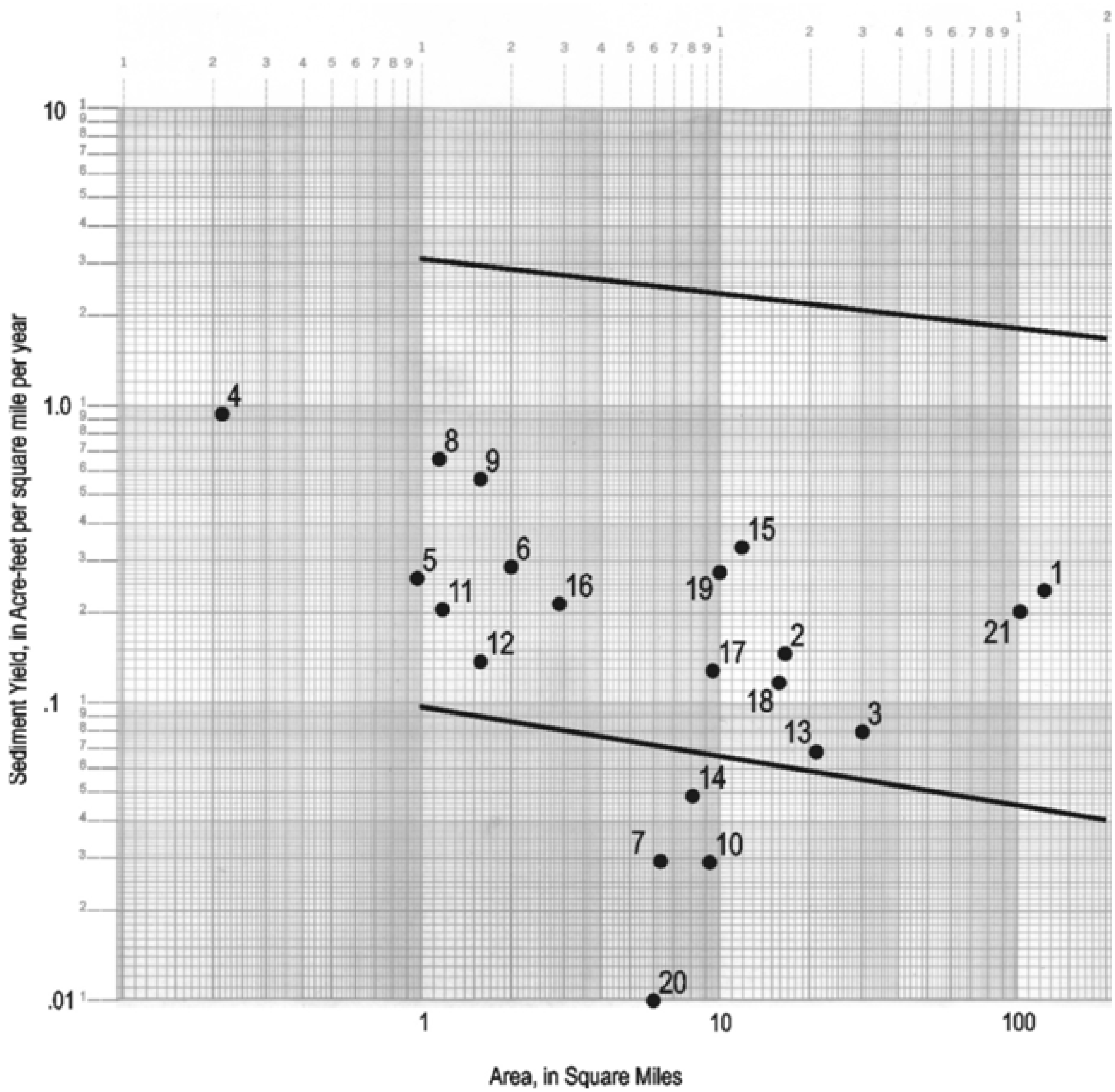
A - USACE, 1974
B - USDA, Natural Resources Conservation Service file data
C - Peterson, 1962
D - Langbein et al., 1951
E - Mussetter et al., 1994

The sediment yield data from Table 11.7 are plotted in Figure 11.21 along with an envelope of sediment yield for 51 watersheds in the United States (Glymph, 1951). It is noted that the U.S. Army Corps of Engineers used a sediment yield of 0.30 acre-feet per square mile per year for the design of Cave Buttes Dam in Maricopa County, Arizona (USACE, 1974). Although at the time (1970), sediment yield for the area immediately upstream of Cave Creek Dam was only 0.24 acre-feet per square mile per year. The larger value (0.30 acre-feet per square mile per year) was used for design purposes to account for large sediment inflow during the September 1970 flood that is not reflected in the 0.24 acre-feet per square mile per year measurement.

The USACE (1974) indicate a range of sediment yield of 0.009 to 1.33 acre-feet per square mile per year for watersheds in Arizona and New Mexico. The U.S. Department of Agriculture (Alonso, 1997), reports sediment yield of 0.12 to 0.4 acre-feet per square mile per year for the Walnut Gulch Experimental Watershed near Tombstone, Arizona.

The wide range of sediment yield is caused by watershed size, soil conditions, precipitation, watercourse conditions and other factors. For example, the relatively small yield of 0.08 acre-feet per square mile per year from the 30 square mile basin above Saddleback Flood Retarding Structure in Maricopa County, Arizona is possibly due to the land surface being covered with well-developed desert pavement. The differences in sediment yield are also related to climate variation. For example, certain watersheds in San Diego County, California reflect yields of only 0.07 and 0.13 acre-feet per square mile per year due to the low annual precipitation of 3 inches per year. Some sites with a large sediment yield, such as Davis Tank, Arizona, are known to have watercourse bed and bank erosion. Lastly, other sites with relatively high yield, such as Black Hills Tank, Arizona, may have experienced a large flood during a short period of data collection.

FIGURE 11.21
REGIONAL SEDIMENT YIELD AS A FUNCTION OF DRAINAGE AREA
LINES INDICATE ENVELOPE FOR 51 U.S. WATERSHEDS BY Glymph (1951)



Runoff and sediment yield data were collected at the Black Hills Tank, near Cave Creek, Arizona, from 1945 to 1948 (Langbein et al., 1951; Peterson, 1962). The precise location of the site is uncertain, but it was near the northern end of the McDowell Mountains on a granite pediment at an elevation of about 2,600 feet. It was possibly the Black Hills Tank located near Dixileta and N. 128th Street. Vegetation was mountain-brush type consisting mainly of snakeweed, yucca, creosote bush, and cactus, with small palo verde and mesquite trees along the channels. According to Langbein et al. (1951), the approximately 2.5 mile long drainage basin was 1.56 square miles in area, headed at 3,200 feet elevation, and was drained by a network of 0.5 to 2 feet deep watercourses at a slope of about 2 percent.

The granitic rock is capped with a thin veneer of coarse residual soil. The watershed sediment yield was 0.9 acre-feet per year or 0.58 acre-feet per square mile per year based on capacity surveys at the beginning and end of the data collection. A field examination of the 1948 flood reportedly showed coarse sediment with uprooted mesquite trees deposited in a fan at the entrance to the tank. There was no spill during the period. According to Peterson (1962), the drainage basin is only 1.14 square miles and the watershed sediment yield is 0.78 acre-feet per year or 0.68 acre-feet per square mile per year. The difference in reported sediment yield for the same watershed is not significant. However, the reported large flood in 1948 is significant because unusually large amounts of sediment were deposited in the tank. The reported average annual sediment yield in Table 11.7 for Black Hills Tank for the 4-year period probably is too high because of the 1948 flood. However, those data indicate the magnitude of sediment that can be produced from a single intense runoff event.

11.7.3 Analytic Methods to Estimate Sediment Yield

11.7.3.1 Introduction

Estimation of sediment yield for a design storm is important for designing a dam or a detention basin. Estimating sediment yield from past flood events is often very difficult due to lack of data. Estimating sediment yield by an analytical method becomes very important, particularly for ungaged watersheds. Numerous analytic methods are available for estimating sediment yield. Such methods can be found in Pacific Southwest Inter-Agency Committee (1974). A commonly used procedure for annual sediment yield is the Revised Universal Soil Loss Equation (RUSLE) (Soil and Water Conservation Society, 1995, USDA, 1997, and Toy and Osterkamp, 1995). Flaxman (1972 and 1974) provides a procedure more applicable to the Western United States. The Modified Universal Soil Loss Equation (MUSLE) can be used to estimate the sediment yield for a single flood event, for example a 100-year flood. The methodology for MUSLE can be found in Appendix B of Simons, Li & Associates, Inc. (1985). The annual sediment yield can also be estimated by a probability-based weighted method based on the MUSLE method for several flood events with different return periods. The detailed procedures can be found in Mussetter et al. (1994).

Equations of mean annual soil loss like RUSLE do not account for climate changes that may produce episodic changes in channel processes such as gullies. For example, in southeastern Arizona there is geologically recent headcutting of the San Pedro River and its tributaries. The sediment yield from gullies and channel enlargement is more than 30 times the sediment yield from rill and inter-rill processes estimated by RUSLE (Toy and Osterkamp, 1995). Renard and

Stone (1981) report sediment yield increases of nearly four times that estimated by the universal soil loss equation (USLE) as a result of channel and bank erosion at two small watersheds in the San Pedro Basin.

Headcutting and gully erosion, and their influence on sediment yield, is discussed by Leopold et al. (1966). Recent headcutting is apparent in the Cave Creek basin especially near the main channel of Cave Creek. Channel incision also is apparent in the Indian Bend Wash basin such as Lost Dog Wash at the southern end of the McDowell Mountains. For watersheds larger than a few acres that have defined channels, mean annual soil loss may or may not be a large part of the sediment yield. The proportion of sediment yielded from the soil and from watercourse beds and banks is difficult to estimate.

The above examples indicate that large amounts of sediment can be derived from the watercourses of small desert watersheds. Large amounts of sediment can be derived from rill development, gully formation and watercourse bed and bank erosion where concentrated runoff from urban development crosses unprotected soil.

Sediment yield/transport numerical simulation models may be used to estimate the sediment yield. The models include, but are not limited to, KINEROS (Smith et al., 1995), HEM (Lane et al., 1995), AGWA (Semmens et al, 2001), GSSHA (Downer and Ogden, 2002), and CCHE1D (Vieira and Wu, 2002). The justification for the selection of a numerical simulation model shall be discussed with FCDMC staff for approval before it is used.

For estimating the total sediment yield by empirical equations in Maricopa County, it is recommended to use Zeller-Fullerton's total bed material equation for bed material load and MUSLE for wash load for single events and then to use the probability-based averaging method for annual sediment yield. The following section will focus on this method.

11.7.3.2 Sediment Yield Estimation by Empirical Equations

The sediment yield at a point of interest is the sum of the total bed material load and wash load delivered to the point of interest. The total bed material load or discharge is calculated with the Zeller-Fullerton equation (Zeller and Fullerton, 1983) which is based on the assumption that the reach is at an equilibrium condition. The wash load is calculated with the MUSLE method. The sediment yield for a particular frequency (return period) can be defined as $BedL + SDR*WashL$ where $BedL$ is the total bed material load, SDR is the sediment delivery ratio for wash load, and $WashL$ is the wash load. The sediment delivery ratio, SDR, measures the ratio of sediment yield for wash load at the watershed outlet (point of interest) to gross erosion in the entire watershed.

If the annual sediment yield is desired, it can be computed as $BedL\_P + SDR*Wash\_P$ where $BedL\_P$ is a probability-weighted average value for MUSLE over floods of different return periods, SDR is the sediment delivery ratio, and $BedL\_P$ is a probability-weighted average value for the Zeller-Fullerton equation over floods of different return periods. The probability-weighted value over different return period floods can be computed as follows (Mussetter et al., 1994):

$$\begin{aligned} Wash\_P = {} & 0.015Wash\_P_{100} + 0.015Wash\_P_{50} + 0.04Wash\_P_{25} + \\ & 0.08Wash\_P_{10} + 0.2Wash\_P_5 + 0.4Wash\_P_2 \end{aligned} \tag{11.27}$$

$$\begin{aligned} BedL\_P = {} & 0.015BedL\_P_{100} + 0.015BedL\_P_{50} + 0.04BedL\_P_{25} + \\ & 0.08BedL\_P_{10} + 0.2BedL\_P_5 + 0.4BedL\_P_2 \end{aligned} \tag{11.28}$$

where:

$Wash\_P$ = annual eroded wash load;

$Wash\_P_i$ = eroded sediment for i = 2-, 5-, 10-, 25-, 50-, and 100-year return periods (from using MUSLE);

$BedL\_P$ = annual total bed material load; and

$BedL\_P_i$ = total bed material load for i = 2-, 5-, 10-, 25-, 50-, and 100-year return periods (from using the Zeller-Fullerton equation).

The above equations require six return periods. Mussetter et al. (1994) also developed a simplified equation that is based on three return periods.

11.7.3.3 Total Bed Material Load

If the reach is assumed to be in an equilibrium condition, the Zeller-Fullerton (Zeller and Fullerton, 1983) total bed material load equation can be used to estimate the total bed material load. It has the form:

$$q_s = 0.0064\left(\frac{n^{1.77}V_a^{4.32}G^{0.45}}{Y_h^{0.3}D_{50}^{0.61}}\right) \tag{11.29}$$

where:

q_s = bed-material discharge in cfs per unit width (which is defined as the bed-material discharge divided by the average flow width; the average width is wetted cross section area divided by flow depth where flow depth can be Manning's equation-based normal depth or maximum flow depth from HEC-RAS);

n = Manning's roughness coefficient;

V_a = average velocity, ft/s;

Y_h = hydraulic depth, ft;

D_{50} = median diameter, also defined as the diameter where 50% is finer by weight, mm;

G = gradation coefficient,

where:

$$G = \frac{1}{2}\left(\frac{D_{84.1}}{D_{50}} + \frac{D_{50}}{D_{15.9}}\right) \qquad (11.30)$$

$D_{84.1}$, D_{50} & $D_{15.9}$ = sediment diameters based on a percent finer (by dry weight), mm.

11.7.3.4 Volume Estimation for Total Bed Material Load

When the peak discharge is used for the Zeller-Fullerton equation, the total bed material discharge is given as the peak bed material discharge of the storm in cubic feet per second per unit width (cfs/ft). However, sediment bed material in volume is more useful for detention basin or reservoir design. The following methodology converts the peak bed material discharge to volume in cubic feet or acre-feet.

First, the bed material discharge per unit width must be multiplied by the channel average width (average width can be defined as wetted cross section area divided by the maximum channel depth or depth computed by Manning's equation) to obtain the result in cubic feet per second. The assumption that the sediment discharge hydrograph follows the same shape as the water flow hydrograph is made to obtain the total bed discharge volume for the total storm duration. This assumption has the form:

$$\frac{Q_s}{V_s} = \frac{Q}{V_w} \qquad (11.31)$$

where:

Q_s = the sediment discharge hydrograph ordinates, cfs,

V_s = total volume of sediment under the sediment discharge hydrograph, ft^3,

Q = the water flow discharge hydrograph ordinates, cfs, and

V_w = total volume of water under the water flow hydrograph, ft^3.

Therefore, the total bed material volume for a flood event is:

$$V_s = V_w\left(\frac{Q_s}{Q}\right) \tag{11.32}$$

The sediment volume, V_s, can be found by using the peak sediment discharge, Q_s, and peak Q, and V_w for the specified return periods (2-yr, 10-yr, etc.). Peak Q_s is equal to q_s multiplied by the average channel flow width. The annual bed material volume (given in cubic feet) can be found by using the probability-weighted equation (Equation (11.28)).

11.7.3.5 Volume Estimation for Wash Load

As discussed above, the result for the total bed material load can be converted to volume in cubic feet or acre-feet. The result for wash load by MUSLE is in tons. It is more useful to have the result in volume rather than weight because a volume gives the required storage for the sediment which can be easily integrated into reservoir and detention basin design. The specific weight for wash load is required to convert the wash load from weight (tons) to volume (cubic feet). Herein, it is assumed that the specific weight for converting wash load from tons to cubic feet is the same as the specific weight for the sediment deposit. It should be noted that the specific weight for a sediment deposit is normally much less than that for a sediment particle because of void space in the sediment deposit. The specific weight for a sediment deposit is often called bulk specific weight.

There are two available methods to determine the bulk specific weight. The first method is to use an empirical equation for initial specific weight discussed in Section 11.6.1.2 (Equation (11.12)). This equation was developed by the FCDMC by fitting a line to the data from ASCE (2006) based on the least squares method and is repeated here for convenience:

$$\gamma_{initial} = 100.5 + 20.44\log_{10}(D_{50}) \tag{11.33}$$

where:

$\gamma_{initial}$ = bulk specific weight for sediment deposits, lb/ft^3, and

D_{50} = median sediment size of the wash load, also defined as the diameter where 50% is finer weight, mm.

Note that D_{50} is the median sediment size of the wash load and not the median sediment size for the soil sample. It is rather difficult to estimate D_{50} of the wash load. Based on Garde and Raju (1985), the limiting size for the wash load may be D_{10} for the soil sample on a river bed. Therefore, D_{50} of the wash load may be approximated by D_{10} of the sediment sample where D_{10} is defined as the diameter of which 10% is finer by weight.

A second method can be used when sediment sampling is not available. In this method, the FCDMC developed a procedure for estimating specific weight for each Natural Resource Conservation Service (NRCS) soil map unit in Maricopa County. It is automated in the latest version of the DDMSW software. More detailed information can be found in the River Mechanics Manual for DDMSW (FCDMC, 2010).

This methodology is based on a relationship between the specific weight and sand percentage (ASCE, 2006) as follows:

$$\gamma = 51(P + 2)^{0.13} \tag{11.34}$$

where:

γ = specific weight for each soil type, lb/ft^3, and

P = the percentage of material larger than 0.05 mm; For actual application when the NRCS soil survey books can be used, P may be assumed to be the percentage of material larger than 0.074 mm but smaller than 4.75 mm (percent).

Based on the Unified Soil Classification System (USCS) and the Bureau of Reclamation (USBR, 1977), sand is defined as that material which does not pass the No. 200 sieve size (0.074 mm) but passes the No. 4 sieve size (4.75 mm).

If multiple layers (vertical direction) were present in a specific map unit, only the top layer was considered. If a map unit was comprised of multiple soil types on the top layer, the map unit percentage of sand was an average of the soil types percentage of sand (e.g., if two soils are present, the total percentage would be the sand percentage of soil 1 plus the sand percentage of soil 2 divided by 2).

However, there were some map units (e.g., Lakes, ponds, reservoirs and rocks) for which a specific weight could not be developed with the above methodology. For lakes, ponds and reservoirs, the specific weight was assumed to be the conservative value of 50 lb/ft^3 in the DDMSW software. For rocks, the specific weight was assumed to be 165 lb/ft^3 in DDMSW, which is the specific weight of quartz.

An average specific weight for the watershed of interest needs to be developed from the specific weights of each map unit. To find this watershed specific weight, an area-weighted, soil erodibility-weighted average is automatically calculated with the following equation. The equation has the form:

$$\gamma_{tot} = \frac{\left(\sum_{i=1}^{n}(K_i A_i \gamma_i)\right)}{\left(\sum_{i=1}^{n}(K_i A_i)\right)} \tag{11.35}$$

where:

K_i = ith soil erodibility factor,

A_i = ith area of the soil,

γ_i = ith specific weight for each soil, lb/ft^3, and

γ_{tot} = bulk specific weight for the entire watershed, lb/ft^3.

The watershed specific weight can then be used to convert the wash load result (from the MUSLE method) from tons to cubic feet. As a note, for lakes, ponds and reservoirs, the erodibility is zero, and these map units do not contribute to the bulk specific weight of the watershed.

11.7.3.6 Sediment Delivery Ratio (SDR) for Wash Load

Only a certain percentage of the eroded sediment particles will reach the watershed outlet. Typically, the larger the watershed is, the lower the percentage will be able to reach the outlet. More sediment particles will be deposited before they reach the outlet. The sediment delivery ratio provides an estimated measure of the percentage that will reach the outlet. The USDA has published a sediment delivery ratio, SDR, curve as a function of drainage area (USDA, 1972). This SDR curve indicates that the value of the sediment delivery ratio decreases as the drainage area increases. However, in the development of this curve by the USDA, data from the southwest arid/semiarid regions were not considered. Lane et al. (2000) developed a sediment delivery ratio (0.41) for the Walnut Gulch watershed (57.53 square miles) near Tucson, Arizona based on measured sediment yield data. Therefore, to develop a curve that is more suitable to arid/semiarid regions, the FCDMC assumed that the adjusted SDR curve follows the same shape as the original USDA curve, but is shifted upwards such that the sediment delivery ratio is 0.41 for a drainage area of 57.53 square miles. Nevertheless, if there is measured data for a specific area, a different SDR value may be used based on engineering judgment.

The FCDMC-recommended curve is shown in Figure 11.22 which may be used for studies in Maricopa County. However, engineering judgment should be exercised. Table 11.8 shows the numerical values that were used to plot Figure 11.22. The following equation also represents Figure 11.22 and Table 11.8:

$$SDR = -14.08(\log_{10}A_D) + 2.44(\log_{10}A_D)^2 - 0.45(\log_{10}A_D)^3 + 60.85 \quad (11.36)$$

where SDR is the sediment delivery ratio (percent), and A_D is the drainage area (square miles). The regression equation should only be used for drainage areas larger than 0.04 square miles and smaller than 500 square miles.

FIGURE 11.22
RELATIONSHIP BETWEEN DRAINAGE AREA AND SEDIMENT DELIVERY RATIO
(MODIFIED FROM USDA, 1972)

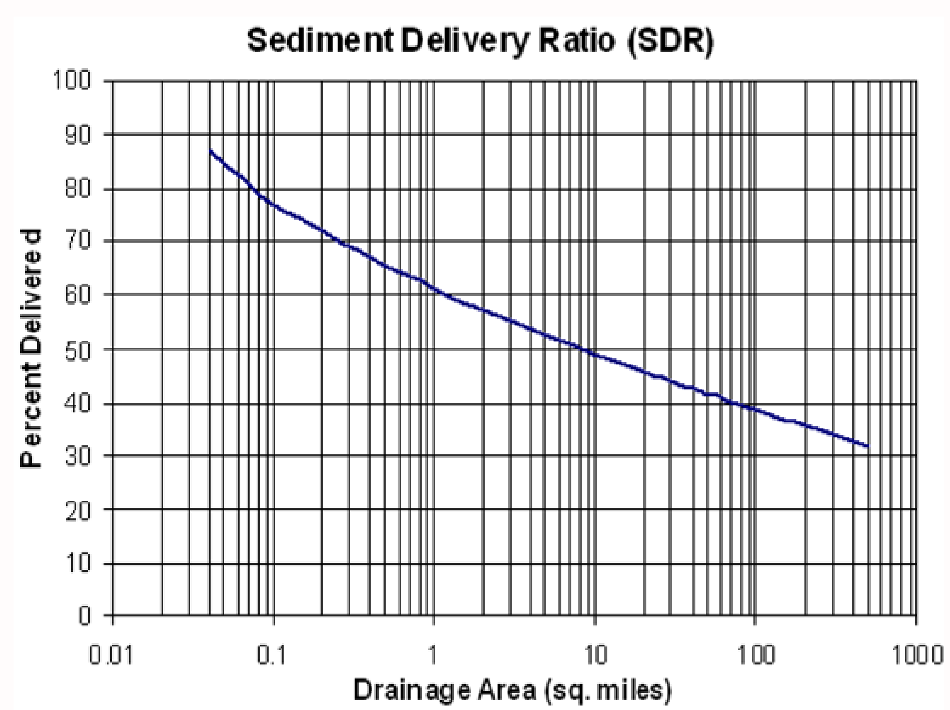


TABLE 11.8
RELATIONSHIP BETWEEN DRAINAGE AREA AND SEDIMENT DELIVERY RATIO
(MODIFIED FROM USDA, 1972)

Drainage Area (square miles)	FCDMC Recommended (percent)
0.04	87
0.1	77
0.2	72

TABLE 11.8
RELATIONSHIP BETWEEN DRAINAGE AREA AND SEDIMENT DELIVERY RATIO
(MODIFIED FROM USDA, 1972)

Drainage Area (square miles)	FCDMC Recommended (percent)
0.4	67
1	61
2	57
15	47
30	44
57.53	41
200	35.5
500	32

11.7.3.7 Wash Load by Modified Universal Soil Loss Equation

The Modified Universal Soil Loss Equation (MUSLE) equation is given by (Simons, Li and Associates, 1985) to compute watershed soil erosion in tons:

$$Y_s = \alpha(Vq_p)^{\beta}KLSCP \tag{11.37}$$

where:

Y_s = watershed soil erosion from a storm of a particular return period, in tons;

V = runoff volume from a storm event of a particular return period, in acre-feet;

q_p = peak flow rate from a storm event of a particular return period, in cfs;

α = 95;

β = 0.56;

K = the soil erodibility factor which can be obtained from Figure 11.23. An Arcview shape file of NRCS soil map units containing default tabulated values for K may be obtained from FCDMC; the FCDMC has developed preliminary values for K values for all soil map units in Maricopa County based on the NRCS Web Soil Survey data http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm (this allows automation of estimating averaged K value for a watershed); more discussion on K values is provided in Section 11.7.3.8.

LS = the topographic factor, defined as:

$$LS = \left(\frac{\lambda}{72.6}\right)^n (0.065 + 0.0454S + 0.0065S^2) \tag{11.38}$$

where:

λ = slope length in feet,

S = the percent slope (e.g. 30 for 30 percent), and

n = an exponent depending upon slope ($n = 0.3$ for $S <$ 3 percent; $n = 0.4$ for $S = 4$ percent; $n = 0.5$ for $S > 5$ percent).

C = the cover and management factor. Arcview shape files for land use from Maricopa Association of Governments (MAG) may be used in conjunction with a land use table to automatically compute the C factor; the FCDMC has developed preliminary values of C for all land use types in Maricopa County based on the MAG's land use GIS shape file; these values serve as a good starting point and allow the automatic estimation of C value for a watershed; more discussion can be found in Section 11.7.3.8.

P = the erosion control practice factor (usually 1.0 for wild land areas). P values for erosion control methods in an agricultural field can be obtained from Wischmeier and Smith (1978).

Slope length, λ, is defined as the distance from the point of origin of overland flow to the point where either the slope gradient decreases enough such that deposition begins, or the runoff water enters a well-defined channel that may be part of a drainage network or a constructed channel. Percent slope, S, is the slope for the slope length. It is in percent, for example, 30 for 30%. It may be approximated by the average watershed slope (Simons, Li and Associates, 1985). The MUSLE equation is best used for slope lengths of less than 400 ft and gradients of 3 to 8 percent.

The cover and management factor, C, can be divided into three distinct types of effects as follows:

$$C = C_I C_{II} C_{III} \tag{11.39}$$

where: (11.40)

C_{I} = Type I, the effects of canopy cover as shown in Figure 11.24,

C_{II} = Type II, the effects of mulch or close-growing vegetation in direct contact with the soil surface as shown in Figure 11.25,

C_{III} = Type III, the tillage and residual effects of the land use as shown in Figure 11.26 (Simons, Li and Associates, 1985),

FIGURE 11.23
SOIL ERODIBILITY FACTOR (K)
(Wischmeier and Smith, 1978)

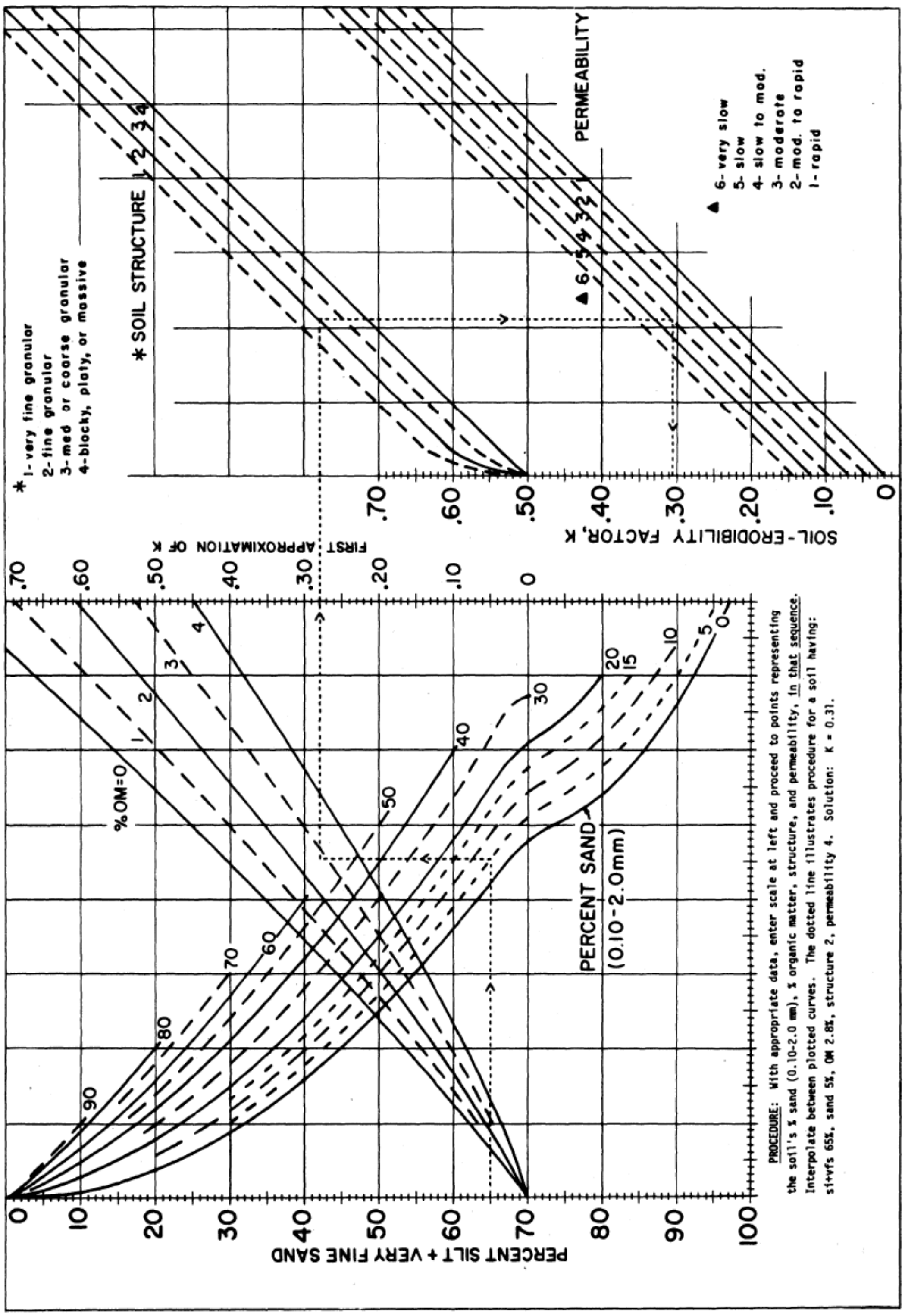


FIGURE 3.—The soil-erodibility nomograph. Where the silt fraction does not exceed 70 percent, the equation is $100\ \mathbf{K} = 2.1\ \mathbf{M}^{1.14}\ (10^{-4})\ (12 - \mathbf{a}) + 3.25\ (\mathbf{b} - 2) + 2.5\ (\mathbf{c} - 3)$ where $\mathbf{M}$ = (percent si + vfs) (100 — percent c), $\mathbf{a}$ = percent organic matter, $\mathbf{b}$ = structure code, and $\mathbf{c}$ = profile permeability class.

FIGURE 11.24
INFLUENCE OF VEGETAL CANOPY ON EFFECTIVE RAINFALL (C_i)
(Wischmeier and Smith, 1978)

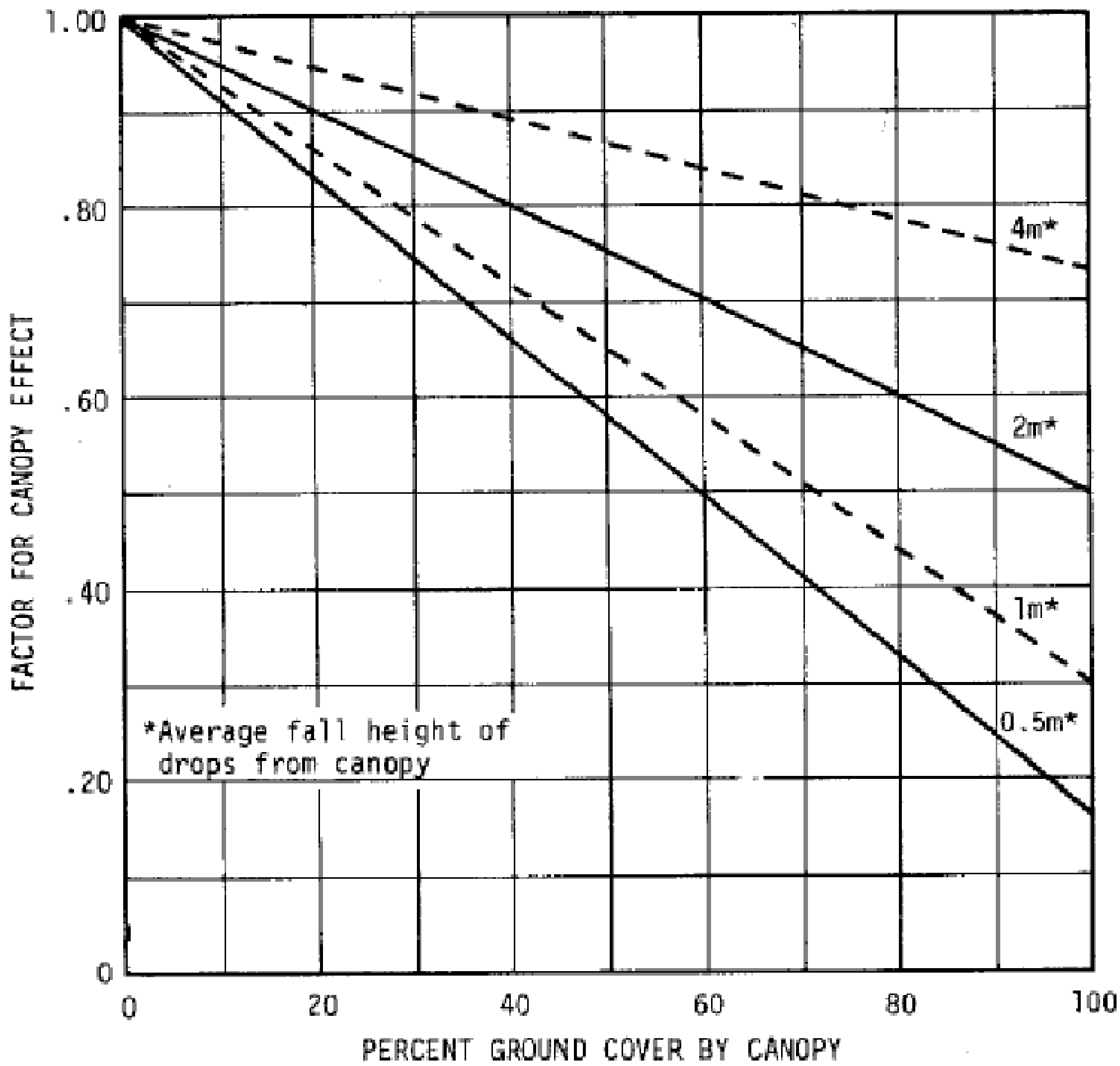


FIGURE 11.25
EFFECT OF PLANT RESIDUES OR CLOSE-GROWING STEMS AT SOIL SURFACE (C_{ii})
(Wischmeier, 1972)

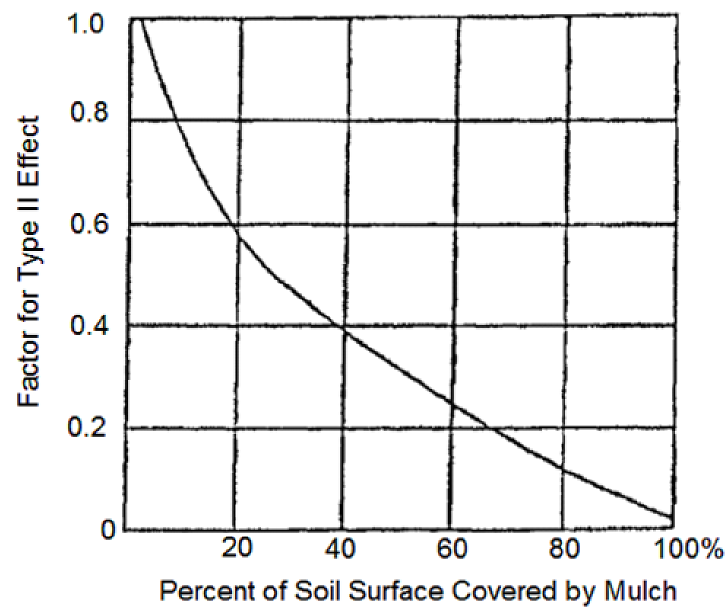
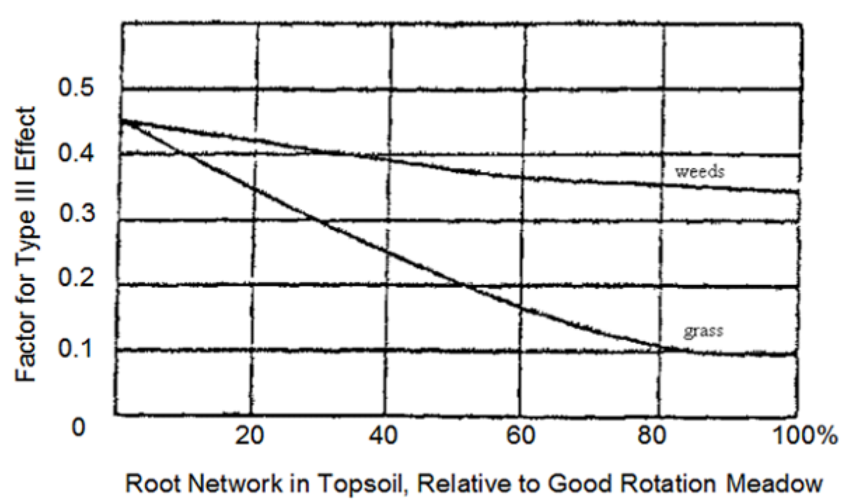


FIGURE 11.26
TYPE III EFFECTS ON UNDISTURBED LAND AREAS (C_{iii})
(Wischmeier, 1972)



11.7.3.8 Soil Erodibility Factor and Cover Management Factor

In the MUSLE method for the calculation of soil erosion, the soil erodibility factor K and the cover and management factor C must be determined. The FCDMC has developed preliminary figures for both of these factors as a rough estimate (Figure 11.23, Figure 11.24, Figure 11.25, and Figure 11.26). These tables serve as a good starting point and allow the automatic estimation of K and C values for a watershed. The automatic estimation of K and C values is implemented in the FCDMC's drainage design software (DDMSW). However, users should review the values in the tables and modify them based on more detailed information. The discussion on automation of K and C values and sediment yield can be found in the River Mechanics Manual for DDMSW (FCDMC, 2010). It should be mentioned that when there is impervious area in the watershed, the wash load should be estimated by multiplying the result with 1 minus the percentage of impervious area.

11.8 ESTIMATION OF SCOUR

11.8.1 Introduction

Scour is the lowering of the bed elevation of a watercourse, either locally or over some defined reach length of the watercourse, due to the hydraulics of flowing water. Scour is estimated as the sum of independent scour components that are due to factors along a defined reach of a watercourse, plus local scour at a specific location in a watercourse.

Scour estimates are often needed for the following drainage and flood control related purposes:

1. Estimation of the response of a watercourse due to altered management in the watershed. For example, scour in a natural watercourse may need to be evaluated due to urbanization that would alter the natural flood magnitude-frequency relations.
2. Estimation of the response of a watercourse due to alterations of the hydraulic conditions in the watercourse. Examples in this regard include floodplain encroachment, flood control modifications such as bank protection, and instream mining of sand and gravel.
3. Estimation of depth of toe-down for structural bank lining.
4. Estimation of depth of scour immediately at or downstream of hydraulic structures.
5. Estimation of potential scour depth for buried utility crossings of watercourses.
6. Estimation of scour depth for bridge piers, embankments, guide banks, and spur dikes.

The estimation of scour is critical to the evaluation of the watercourse stability at and near highway structures. Procedures to investigate watercourse stability are provided in HEC-20 (USDOT 2001c). Procedures to provide bridge scour countermeasures are provided in HEC-23 (USDOT, 2001a). The estimation of scour is an engineering application that requires both specific expertise and experience. Every application of scour technology is unique because of the wide variability of hydrologic, hydraulic and geologic/geomorphic factors. It is not possible to compile a comprehensive methodology in a drainage design manual that would be adequate to address all aspects of scour estimation. In addition, the knowledge of erosion and sedimentation is continually expanding because of the need to provide better technology in this field of engineering. Often, newer methodologies are presented in the engineering literature that should be considered and used, if appropriate. The following are general guidelines for estimating scour along with currently used methodologies that are considered applicable in Maricopa County.

11.8.2 Total Scour

Total scour, for a given application, should consider the following components of scour:

1. Long-term degradation of the bed of the watercourse.
2. General scour through a specific reach of the watercourse.
3. Scour induced due to a bend in the watercourse.
4. Scour associated with bedform movement through the watercourse.
5. Scour due to low-flow incisement.
6. Local scour due to bridge pier, bridge abutment, guide bank, etc.

Total scour, Z_t, is the sum of each of these individual components, Z_i, of scour. Total scour can be expressed as:

$$Z_t = FS(Z_{long\text{-}term} + Z_{general} + Z_{bend} + Z_{bedform} + Z_{low\text{-}flow}) + FS_{local}{}^{*}Z_{local} \qquad (11.41)$$

where:

FS = the factor of safety (safety factor) for the long-term, general, bend, bedform, and low-flow incisement scour components, and

FS_{local} = the factor of safety for local scour such as pier scour, downstream scour for drop structure/grade control structures and other local scour components.

The factor of safety is often used for hydraulic engineering design to account for uncertainties in hydraulic engineering analyses. In general, a factor of safety of 1.3 for long-term, general, bend, bedform and low flow incisement scour should be used for the design of toe-down for bank protection. However, a lower value of the safety factor may be used under special circumstances with prior approval from the FCDMC and other jurisdictional agencies. The use of a higher safety factor, such as 1.5, may be justified where underestimation of scour could cause catastrophic failure that may result in loss of life or unacceptable economic consequences. The local scour safety factor, FS_{local}, may be less than 1.3 under special conditions, such as in the calculation of pier local scour when the debris width is added to the pier diameter or in the calculation of drop structure downstream scour when the result shows unreasonably large local scour. A safety factor of less than 1.3 may also be used for abutment scour because the abutment scour methodology often over-estimates the value.

In general, the recommended toe-down depth or utility burial depth is the total scour depth below the channel thalweg for areas inside the FEMA floodway or main channel banks (Figure 11.27). The thickened dash line in Figure 11.27 represents the toe-down or utility burial depth. The channel thalweg is the lowest point on a channel cross section. The velocity for computing the total scour depth within the floodway or main channel should be the main channel velocity or floodway velocity, whichever is larger. The toe-down depth or utility burial depth is the total scour depth below the adjacent ground for areas on the lateral-erosion line (Figure 11.27). The procedure for estimating the lateral-erosion line can be found in Section 11.9. For areas between the FEMA floodway line and the lateral erosion line, the toe-down depth or utility burial depth can be linearly interpolated as shown in Figure 11.27. For areas between the lateral erosion line and the FEMA floodplain line, the toe-down depth or utility burial depth can be linearly interpolated between the toe-down/burial depths at the lateral erosion line and the FEMA floodplain line. The toe-down/ burial depth at the FEMA floodplain line can be the total scour depth based on overbank velocity and flow depth. The overbank velocity may be obtained from an existing HEC-RAS model.

Figure 11.28 illustrates the recommended toe-down depth or utility burial depth for a very erosive condition where very erosive material is found in the channel bank and bed and large erosion and channel migration were observed in the past. Figure 11.29 and Figure 11.30 illustrate the recommended toe-down depth or utility burial depth for a situation where the lateral erosion line is outside the FEMA floodway line. It may be noted that areas outside the FEMA floodplain may be beyond the floodplain administrators' jurisdiction. Engineering judgment is highly recommended about the toe-down or burial depth for areas outside the FEMA floodplain but within the lateral erosion line.

The following is a discussion of each component of scour that should normally be considered when estimating total scour. FCDMC's DDMSW software can be used to estimate the scour components. The software can be downloaded from FCDMC's web site at (http://www.fcd.maricopa.gov).

.

FIGURE 11.27
TOE-DOWN AND UTILITY BURIAL DEPTH (EROSION, INSIDE FLOODPLAIN)
(LATERAL EROSION LINE IS INSIDE FLOODPLAIN LINE)

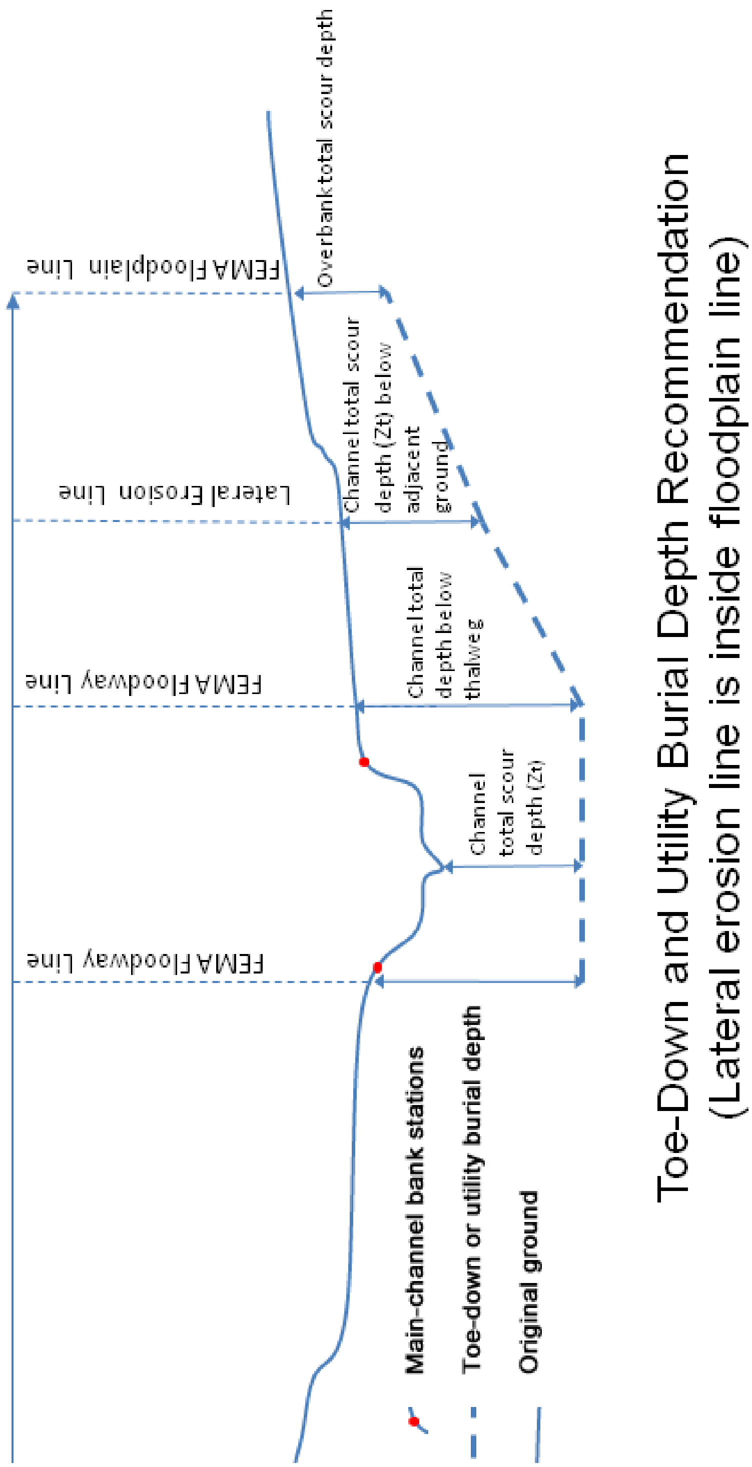


FIGURE 11.28

TOE-DOWN AND UTILITY BURIAL DEPTH (VERY EROSIVE, INSIDE FLOODPLAIN)

(A VERY EROSIVE CONDITION; LATERAL EROSION LINE IS INSIDE FLOODPLAIN LINE)

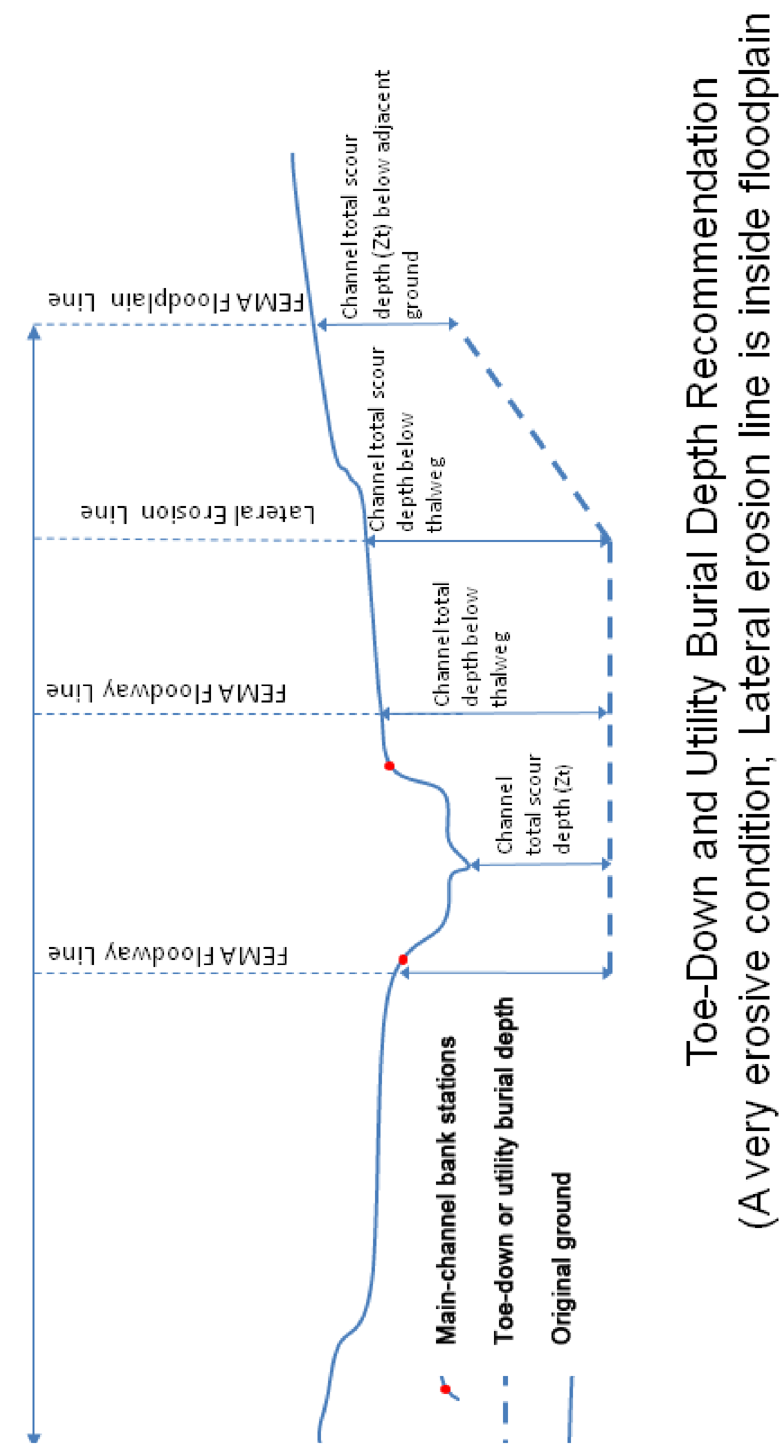


FIGURE 11.29
TOE-DOWN AND UTILITY BURIAL DEPTH (EROSION, OUTSIDE FLOODPLAIN)
(LATERAL EROSION LINE IS OUTSIDE FLOODPLAIN LINE)

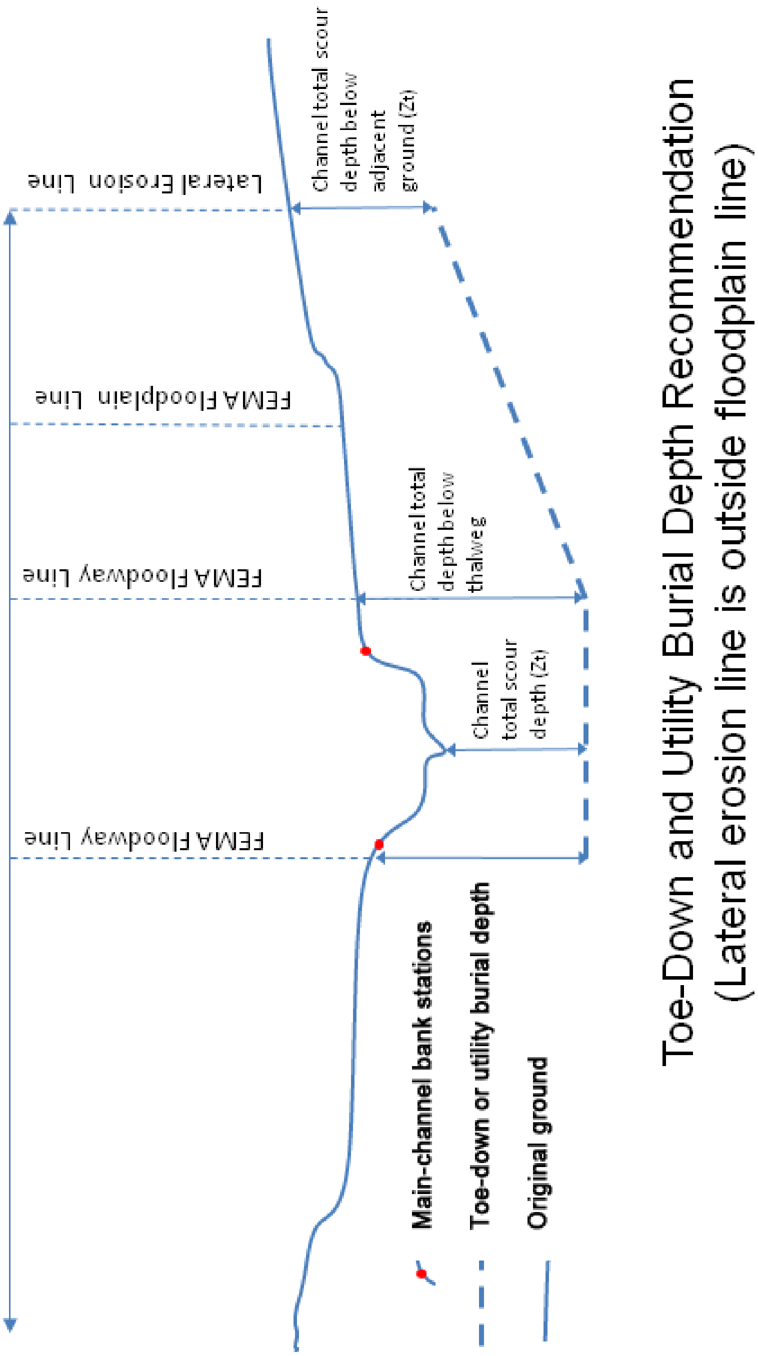
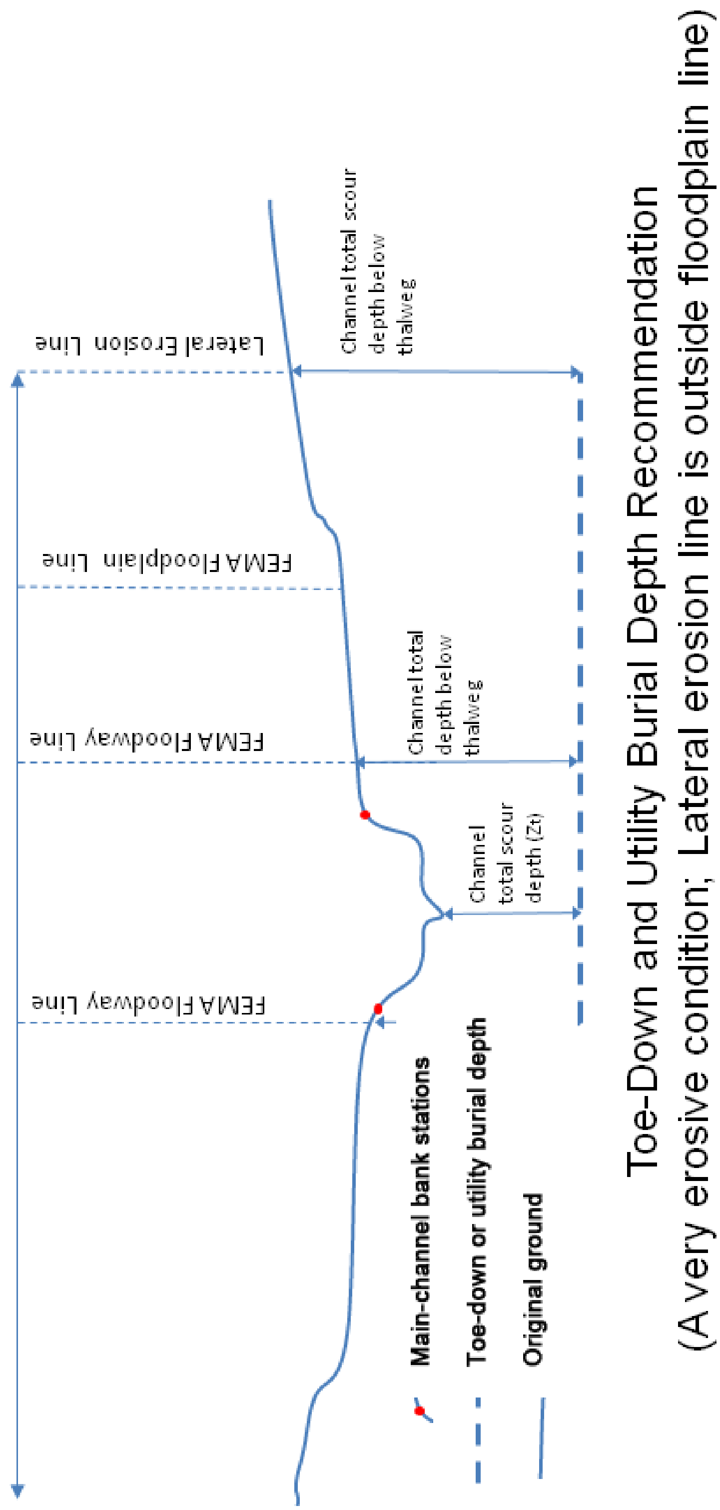


FIGURE 11.30
TOE-DOWN AND UTILITY BURIAL DEPTH (VERY EROSIVE, OUTSIDE FLOODPLAIN)
(A VERY EROSIVE CONDITION; LATERAL EROSION LINE IS OUTSIDE FLOODPLAIN LINE)



11.8.2.1 Long-Term Scour (Degradation)

Long-term degradation can be estimated by the following methods:

1. A trend analysis of historic elevation data.
2. Simulation by use of sediment transport modeling such as HEC-6 (USACE, 1991), HEC-6T (MBH, 2002), FLUVIAL-12 (Chang, 2006), or other sediment transport modeling software subject to the FCDMC's approval prior to the modeling.
3. Application of equilibrium slope analyses.
4. Level I Analysis from Arizona State Standard 5-96 (ADWR, 1996).

Trend Analysis

A trend analysis of historic bed elevation data is limited by the availability of adequate, long-term data for the watercourse. Therefore, such an analysis may be possible only for some of the major watercourses in Maricopa County. In addition, factors such as instream gravel mining and channelization of the watercourse may complicate such historic analyses.

Sediment Transport Modeling

Simulation modeling such as HEC-6, HEC-6T, FLUVIAL-12 or other FCDMC-approved software may provide useful results. The simulation results may be highly sensitive to hydrologic input (flood magnitude-frequency relations, flow duration, shape of hydrograph, etc.). Simulation modeling may only be appropriate for regional studies of major watercourses, especially those for which structural flood control alternatives are being considered. Whenever data are available, site-specific calibration should be performed to determine the parameters in the model such that it can reproduce the historical scour/deposition. Sensitivity analyses should be performed to analyze how the results respond to different input parameters. When applying the modeling approach to evaluating the scour/deposition hazard, the maximum scour or deposition during the entire simulation time period must be used for the basis of design. The flow hydrograph can be generated from historical flow records if they are available. Ideally, records for a period of one hundred years should be used. When there is no historical flow record, a synthetic long-term hydrograph can be generated. As indicated by Chang (2006), “In the time span of 100 years, one may expect statistically one flood event exceeding the 100-year flood, two events exceeding the 50-year flood, four events exceeding the 25-year flood, ten events exceeding the 10-yr flood, etc.” Therefore, the following is a potential group of events that may be used for 100-year time span long-term simulation: one 100-year flood hydrograph, one 50-year flood hydrograph, two 25-year flood hydrographs, and six 10-year flood hydrographs. The sequence of flood events is subject to engineering judgment. A sensitivity analysis may be needed to help select the sequence of these events. The FCDMC, or jurisdictional agency, must approve the proposed synthetic long-term hydrograph before it is used in the model.

Site-specific bed material sediment data is needed for sediment transport modeling. The bed material sediment sampling approval should be based on Pemberton and Lara (1984).

The inflowing sediment load for various discharges for the study reach may be estimated by (1) field measurement at the study reach upstream end (very unlikely in Maricopa County), (2) use of an appropriate sediment transport equation if the upstream supply reach is in an equilibrium condition, and (3) an iterative sediment transport modeling approach for the upstream supply reach if the supply reach is in equilibrium (trial of different sediment inflow loads for supply reach until the sediment outflow for the supply reach is equal to the sediment inflow). However, when HEC-6T is used, the recirculation option may be used to automatically determine the inflowing sediment load.

An equilibrium condition for a channel reach is where the inflowing sediment load (volume or peak sediment discharge) for a channel reach is the same as the sediment outflow load (volume or peak sediment discharge). It corresponds to the case where there is no overall channel degradation or aggradation. If no significant channel degradation or aggradation is observed from aerial photos, topographic data comparison, and field visits, the channel may be considered in an equilibrium condition. When using the sediment transport modeling approach, if the immediate upstream supply reach is not in equilibrium, one should look further upstream until an equilibrium condition reach is located.

Equilibrium Slope Analysis

Equilibrium slope analysis is a method that can often be applied to estimate long-term degradation without extensive data or modeling effort. The equilibrium slope is the channel bed slope when the sediment inflow load and outflow load for the study reach are the same. It is the slope that corresponds to the equilibrium condition. When a channel reaches the equilibrium condition, there is no channel aggradation or degradation for the study reach. The application of this method requires that the study reach is not armored. It also requires the identification of a downstream bed elevation control (pivot point) at which the bed elevation is not expected to change. Such a control can be bedrock, caliche, a reach of armored channel bed, or a constructed facility such as a diversion dam, roadway crossing, and so forth. The dominant discharge should be used for equilibrium slope analysis. In Maricopa County, either a 5-year event or a 10-year event can be considered as the dominant discharge. A bankfull discharge may also be considered as the dominant discharge, or can be used as a basis for selection of either the 5-year or 10-year storm. Selection of appropriate bank stations is very important and should be carefully considered. Refer to Cruff (1999) for guidance in selection of bank stations for determining the dominant discharge, but keep in mind that the bank station positions may need to be adjusted for sediment transport numerical computation purposes.

Long-term degradation using equilibrium slope analysis (Simons, Li and Associates, 1985) is estimated by:

$$Z_{long\text{-}term} = L\,\Delta S \qquad (11.42)$$

where:

$Z_{long\text{-}term}$ = the long-term scour, in feet,

L = the distance upstream of the pivot point in feet, and

ΔS = S_0-S_{eq}

where:

S_0 = the channel bed existing slope

S_{eq} = the channel bed equilibrium slope.

When the equilibrium slope is larger than the existing bed slope upstream from the pivot point, it would indicate aggradation rather than degradation. When it is an aggradation zone, the long-term scour depth may be considered zero as part of the total scour depth for structures design, because the long-term equilibrium status is dynamic and simply deducting aggradation depth from the total scour depth may under-estimate the total scour depth.

Application of long-term degradation is illustrated by the following:

> A natural watercourse has a slope of 22 feet per mile (0.0042 ft/ft). Proposed channelization of the watercourse will increase the unit discharge and the equilibrium slope is estimated to decrease to 15 feet per mile (0.0028 ft/ft). A drop structure is proposed at a distance of 2,000 feet upstream of a pivot point (armored channel cross section). The long-term degradation at the toe of the drop structure is estimated by:

$$\begin{aligned} Z_{long\text{-}term} &= (2000\,ft)(0.0042 - 0.0028\,ft/ft) \\ &= 2.8\,feet \end{aligned}$$

The key to long-term degradation by equilibrium slope analysis is the estimation of the equilibrium slope. The selection of an appropriate equilibrium slope equation depends upon the study reach's sediment flow condition. A clear water sediment flow condition occurs when there are upstream reservoirs, sand and gravel pits, or hydraulic structures that significantly reduce the sediment supply to the study reach. A sediment-laden condition occurs when there is no reservoir, sand or gravel pits, or hydraulic structures that significantly reduce the sediment supply. For a clear water condition in the study reach, the Schoklitsch bedload equation (Shulits, 1935; Pemberton and Lara, 1984) for zero bedload transport is recommended to estimate the equilibrium bed slope or the limiting bed slope. The Schoklitsch bedload equation is used to find the clear water condition equilibrium bed slope as follows:

$$S_{eq} = 0.00174\left(\frac{D*B}{Q}\right)^{3/4} \tag{11.43}$$

where:

S_{eq} = equilibrium slope for clear water conditions, ft/ft;

Q = dominant discharge (usually a 10-year event), cfs;

D = mean particle size, which may be assumed to be the median particle size, D_{50}, mm;

D_{50} = particle size in a mixture in which 50% are smaller, mm;

B = channel bed width, ft.

For the sediment-laden condition where there is no upstream reservoir, sand or gravel pits, or hydraulic structures that will significantly reduce the sediment load to the study reach, the iterative method should be used based on Section 5.3.7 in Design Manual for Engineering Analysis of Fluvial Systems (Simons, Li and Associates, 1985). The requirement of this method is that the immediate upstream supply reach must be in an equilibrium condition where the inflowing sediment load and outflowing sediment load for the upstream supply reach are the same. This can be checked by historical and recent aerial photos, topographic maps, and field visits. If no significant aggradation or degradation is found in the supply reach, then the supply reach may be considered in equilibrium and an appropriate total bed material load equation such as the Zeller-Fullerton equation, Equation (11.44), (Zeller and Fullerton, 1983) can be used to estimate the sediment load from the supply reach. When the immediate upstream supply reach is not in equilibrium, one should consider a longer supply reach where the channel may reach equilibrium or look for an equilibrium segment further upstream. Once the supply reach has been verified that it is in an equilibrium condition, the Zeller-Fullerton (Zeller and Fullerton, 1983) total bed material load equation, Equation (11.44), can be used to estimate the sediment load for the supply reach, which is the sediment inflow to the study reach. The total bed material sediment discharge based on Zeller-Fullerton equation is:

$$Q_s = q_s W = 0.0064\left(\frac{n^{1.77} V_a^{4.32} G^{0.45}}{Y_h^{0.3} D_{50}^{0.61}}\right) W \tag{11.44}$$

where:

Q_s = total bed material discharge in cfs;

q_s = total bed material discharge in cfs per unit width;

W = *flow average width*	=	average width of flow, defined as the wetted area divided by flow depth (the flow depth can be the Manning's equation-based normal depth or maximum flow depth from HEC-RAS);
n	=	Manning's roughness coefficient;
V_a	=	average velocity, ft/s;
Y_h	=	hydraulic depth, ft;
D_{50}	=	median diameter, also defined as the diameter where 50% is finer by weight, mm;
G	=	gradation coefficient, where:

$$G = \frac{1}{2}\left(\frac{D_{84.1}}{D_{50}} + \frac{D_{50}}{D_{15.9}}\right) \tag{11.45}$$

and $D_{84.1}$, D_{50} and $D_{15.9}$ are sediment diameters based on a percent finer by dry weight, mm.

After the supply total bed material sediment discharge is computed, one uses the Zeller-Fullerton equation to compute the total bed material sediment discharge for the study reach. If the sediment discharge for the study reach is equal to the supply sediment discharge, then the current channel bed slope for the study reach is the equilibrium slope. If not equal, one should vary the channel bed slope such that the sediment discharge for the study reach is equal to the supply reach sediment discharge. Once the sediment discharges are equal, the computed bed slope is the equilibrium slope. During this iteration process, Manning's equation or HEC-RAS may be used to compute the hydraulic variables.

When the immediate upstream supply reach is not in equilibrium, one should consider a longer upstream supply reach where the channel may reach equilibrium or look for an equilibrium segment further upstream.

Level I Analysis from Arizona State Standard 5-96

The equilibrium slope method requires locating an appropriate downstream pivotal point. When such a pivotal point does not exist, a simplified method based on ADWR (1996) may be used to estimate the long-term degradation as the last resort. The long-term degradation by ADWR (1996) Level I analysis is $0.02Q_{100}^{0.6}$ where Q_{100} is the 100-year peak flow in cubic feet per second. The long-term degradation is in feet. This equation should only be used when no downstream control structures exist.

Limits to Long-term Scour from Armoring

When computing the long-term scour, the potential for armoring should be considered. Armoring is the process in an alluvial watercourse where sediment transport removes bed material smaller than a certain size thus leaving a bed that is armored by the larger bed particle material. All alluvial channels experience the mechanics of armoring through the selective transport of finer bed material and leaving the coarser bed material. However, watercourses that continually receive inflow of bed material load in excess of transport capacity, or do not contain adequate quantities of the larger, armoring-size bed material, will not experience armoring. Also, armoring is flood magnitude dependent; that is, an armoring layer can develop over time due to a sequence of flood events, but a flood event sufficiently larger than those that formed the armor layer can penetrate the armor layer resulting in additional scour depth.

When the channel bed surface for a channel reach is entirely covered with cobbles/rocks, it is possible that this segment of the channel reach is already armored for the storm event under consideration and the long-term scour may be assumed to be zero. This armored bed may serve as the pivot point for upstream equilibrium slope analysis for clear-water long-term scour analysis. To verify if the surface cobbles/rocks have armored the river bed, one needs to compute the sediment critical particle size, d_c, by using Shields relationship. The channel bed surface may be considered armored or equivalently the long-term scour depth is taken as zero if the following two criteria are met. The first one is that the particle size for the majority of the bed surface is greater than d_c or $d_{10} > d_c$. The second criteria is that the median particle size of the bed surface material is greater than the required d_{50} computed by riprap design for a stable channel bed. The required d_{50} can be estimated by using the modified Isbash equation as set forth in Chapter 6, Loose Angular Riprap Sizing (d_{50}):

$$d_{50} = kV_a^2\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right) \qquad (11.46)$$

where:

d_{50}	=	the required median sediment particle diameter for which 50% of the material (by weight) finer, ft;
k	=	0.0191 for a straight channel (bend angle less than 30 degrees; assuming low turbulent flows);
k	=	0.0372 for a curved channel (bend angle more than 30 degrees; assuming high turbulent flows);
V_a	=	average velocity, ft/s;
γ_s	=	specific weight of stone, lb/ft^3; and
γ_w	=	specific weight of water, lb/ft^3.

If the entire channel bed is not covered with large cobbles/rocks, but a large amount of rocks are observed on the surface, it may be reasonable to assume that an armored layer may eventually develop at a certain depth below the bed surface. When the channel bed surface and sub-surface contains more than 10% coarse material, which can not be transported under dominant flow conditions, $d_{90}>d_c$, armoring will eventually develop at a certain depth below the surface (Pemberton and Lara, 1984). This depth may be assumed to be the long-term scour depth, Z_s, which can be estimated by Equation (11.25).

11.8.2.2 General Scour

General scour is one component of total scour that would occur during the passage of a design flood. The design flood may be a 100-year flood or other design events such as the Standard Project Flood (SPF) depending on the design purposes. This type of scour involves the removal of material from the bed and banks across all or most of the width of a channel. The scour is caused by increased velocities and shear stresses dictated by the local area geometry, such as at constrictions, and water surface controls. General scour can be estimated by using the empirical equations (Pemberton and Lara, 1984). It may be estimated by using a sediment transport model such as HEC-6 (USACE, 1991), HEC-6T or other FCDMC-approved models. When a sediment transport model such as HEC-6 or HEC-6T is used, the design flood hydrograph should be used. Since most one-dimensional sediment transport models are based on cross section averaged values and tend to under-estimate the general scour for the alluvial channels in the semi-arid areas, engineering judgement must be exercised to choose the most appropriate approach.

The empirical equations by the Bureau of Reclamation (page 29 - page 37 in Pemberton and Lara, 1984) for general scour due to passage of a design flood are the Neill equation, Lacey equation, and the Blench equation for zero-bed-transport.

In general, each equation should be applied as follows:

> Neill Equation. Neill's equation is applicable to areas of channel constriction, such as bridges or contraction structures. This approach also accounts for scour where bends are present in the contracted zone.
>
> Lacey Equation. This method is more applicable to a natural river system where there is not an upstream structure that captures sediment.
>
> Blench's Equation. This method is applicable to streams where the upstream sediment inflow is intercepted by basins or dams, creating clear water flow.
>
> Each of these approaches is discussed in detail in the following sections.

Neill Equation and HEC-18 Contraction Scour

For a bridge general scour estimate, the higher value between Neill's general scour equation, Equation (11.48), (Neill, 1973) and the HEC-18 contraction scour equation (USDOT, 2001b) should be used. If there is a bend, then the higher value between Neill's equation with an appropriate bend coefficient, Z, and the HEC-18 contraction scour equation with Zeller's bend scour equation, Equation (11.60), should be used.

The bend scour should be computed for the areas both at the bend and downstream of the bend because the secondary currents will still cause scour downstream of the bend. The distance from the bend at which the secondary currents will have decayed to a negligible magnitude can be found in Section 11.8.2.3.

The Neill equation is applicable to channel constriction cases where there is a bridge or contraction structure (Neill, 1973). Neill's equation is as follows:

$$Z_{general} = Zd_i\left[\frac{q_f}{q_i}\right]^m \tag{11.47}$$

where:

$Z_{general}$ = general scour depth, ft;

d_i = average depth at bankfull discharge in incised reach, ft (= hydraulic depth for bankfull discharge or dominant discharge);

q_f = design flood discharge per unit width (width can be defined as wetted cross sectional area divided by flow depth where flow depth can be the Manning’s equation-based normal depth or maximum flow depth from HEC-RAS), cfs/ft;

q_i = bankfull discharge in incised reach per unit width (bankfull discharge can be from HEC-RAS main channel flow discharge between appropriate bank stations or taken as the 10-year event with the same definition as dominant discharge (Simons, Li and Associates, 1985); width can be defined as wetted cross section area divided by depth where flow depth can be the Manning’s equation-based normal depth maximum flow depth from HEC-RAS, cfs/ft;

m = exponent varying from 0.67 for sand to 0.85 for coarse gravel; and

Z = multiplying factor (0.5 for a straight reach, 0.6 for a moderate bend, and 0.7 for a severe bend).

The HEC-18 contraction scour equations (USDOT, 2001b) are used to predict the depth of the contraction scour component in a contracted section. The equations for the clear-water condition and the live-bed condition are different. The following equation for critical velocity can be used to determine if the flow upstream of the bridge is clear-water or live-bed (USDOT, 2001b). The equation has the form:

$$V_c = 11.17 y_a^{1/6} D_{50}^{1/3} \tag{11.48}$$

where:

V_c = critical velocity, ft/s;

y_a = average depth of flow upstream of the bridge, ft (= hydraulic depth); and

D_{50} = particle size in a mixture in which 50% are smaller, ft.

The D_{50} is taken as an average of the bed material size in the reach of the stream upstream of the contraction.

When V_c < mean velocity, the live-bed equation should be used. Conversely, when $V_c \geq$ mean velocity, use the clear-water equation.

Live-bed Contraction Scour

The live-bed contraction scour equation is the modified Laursen equation (USDOT, 2001b) given as:

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \tag{11.49}$$

and

$$y_s = y_2 - y_0 \tag{11.50}$$

where:

y_s = average contraction scour depth, ft;

y_0 = existing depth of flow (the hydraulic depth) in the contracted section before scour, ft;

y_1 = average depth of flow (hydraulic depth) in the upstream main channel, ft;

y_2 = average depth of flow (hydraulic depth) in the contracted section, ft;

Q_1 = flow in the upstream channel transporting sediment, cfs;

Q_2 = flow in the contracted channel section, cfs;

W_1 = bottom width of the upstream main channel that is transporting bed material, ft;

W_2 = bottom width of the main channel in the contracted section less pier widths, ft;

k_1 = exponent determined from Table 11.9.

Please note that Q_1 may be smaller than, larger than or equal to Q_2, since there are varied flow conditions, and Q_1 is defined as the flow that is carrying sediment. This means that in some cases wide shallow overbank areas will not be counted in Q_1, but may be counted in Q_2 if the entire flow is pushed through the contracted section.

TABLE 11.9
VALUES OF k_1
(USDOT, 2001b)

$(V^*)/\omega$	k_1	Mode of Bed Material Transport
<0.5	0.59	Mostly contract bed material discharge
0.5 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

The variables for Table 11.9 are defined as follows:

V^* = shear velocity in the upstream section, ft/s given by $(gy_1S_1)^{0.5}$;

g = gravitational acceleration, 32.2 ft/s^2;

S_1 = slope of the energy grade line of main channel, ft/ft;

ω = fall velocity in m/s of bed material from Figure 11.31 based on using D_{50} as D_s in mm, or in ft/s from regression equations developed by FCDMC as follows:

$$\omega = 3.28 * 10^a$$

where:

for 40^o C:

$$a = -0.82901 + 0.74363(\log_{10}D_{50}) - 0.30037(\log_{10}D_{50})^2 + 0.049991(\log_{10}D_{50})^3 \quad (11.51)$$

for 20^o C:

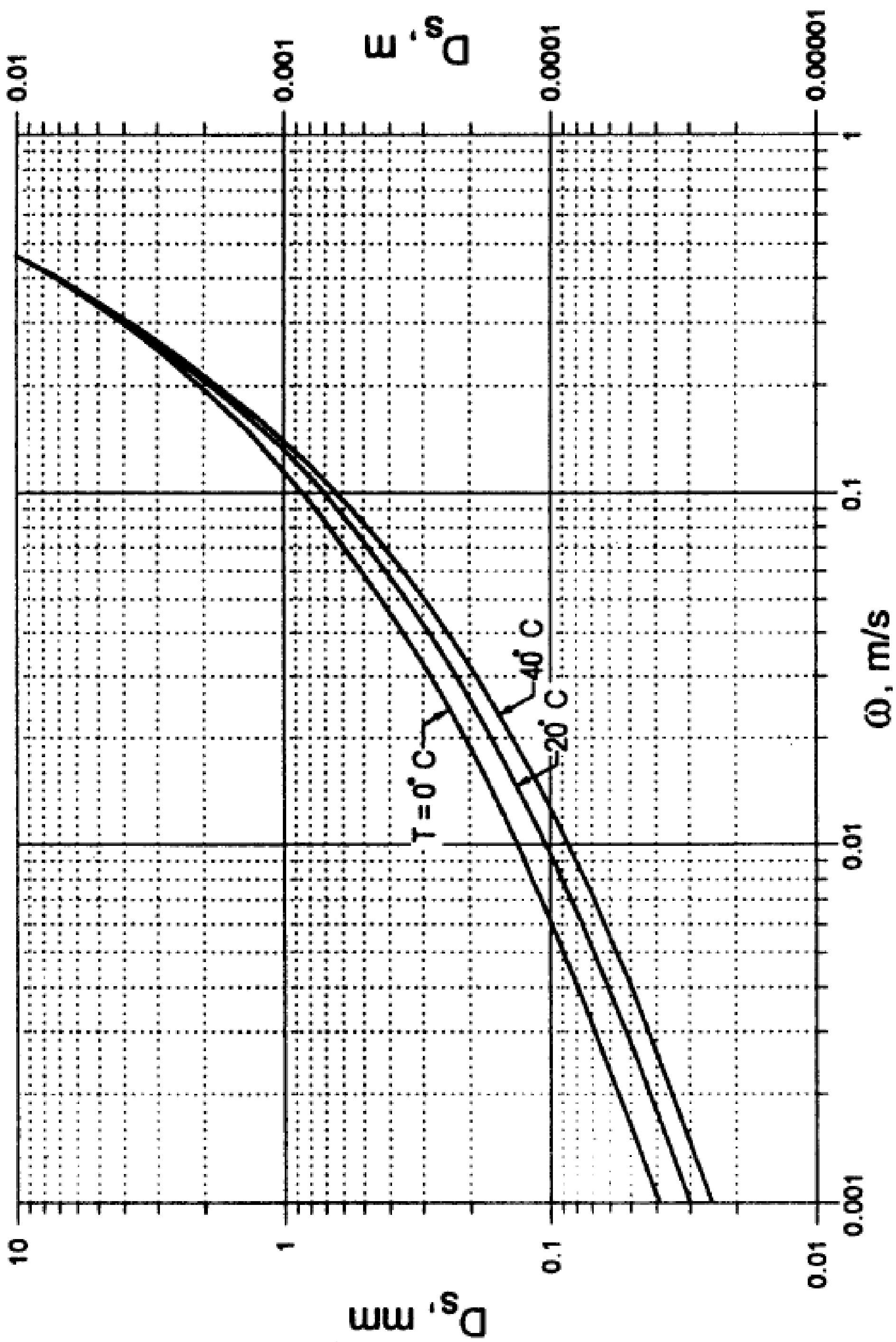
$$a = -0.84779 + 0.785215(\log_{10}D_{50}) - 0.33025(\log_{10}D_{50})^2 + 0.052387(\log_{10}D_{50})^3 \quad (11.52)$$

for 0^o C:

$$a = -0.90682 + 0.936036(\log_{10}D_{50}) - 0.38413(\log_{10}D_{50})^2 + 0.012187(\log_{10}D_{50})^3 \quad (11.53)$$

D_{50} = particle size in a mixture in which 50% are smaller, mm.

FIGURE 11.31
FALL VELOCITY
(USDOT, 2001b)



where: D_S is representative of sand-sized particles.

Clear-water Contraction Scour

The clear-water contraction scour per USDOT, 2001b, is defined by:

$$y_s = y_2 - y_0 \tag{11.54}$$

$$y_2 = \left(\frac{0.0077Q^2}{D_m^{2/3}W^2}\right)^{3/7} \tag{11.55}$$

where:

y_s	=	average contraction scour depth, ft;
y_2	=	average equilibrium depth (hydraulic depth) in the contracted section after contraction scour, ft;
Q	=	discharge through the contraction or on the set-back overbank area at the contraction associated with the width W, cfs;
D_m	=	diameter of the smallest nontransportable particle in the bed material ($1.25D_{50}$) in the contracted section, ft;
D_{50}	=	median diameter of bed material, also defined as the diameter where 50% is finer by weight, ft;
W	=	bottom width of the contracted section less pier width, ft; and
y_0	=	average existing depth (hydraulic depth) in the contracted section, ft.

Lacey Equation

The Lacey equation is more applicable to a natural river system (Blench, 1969) where there are no upstream structures that capture sediment:

$$Z_{general} = Z\left(0.47\left[\frac{Q}{f}\right]^{1/3}\right) \tag{11.56}$$

where:

$Z_{general}$	=	general scour depth, ft;
Q	=	design discharge, cfs;
f	=	Lacey's silt factor = $1.76(D_m)^{1/2}$;
D_m	=	mean grain size, which may be approximated by D_{50}, (diameter where 50% is finer by dry weight) mm; and

Z = multiplying factor (0.25 for a straight reach, 0.5 for a moderate bend, 0.75 for a severe bend, 1.0 for right angle bends, and 1.25 for a vertical rock bank or wall).

The bend scour should be computed for the areas both at the bend and downstream of the bend because the secondary currents will still cause scour downstream of the bend. The distance from the bend at which the secondary currents will have decayed to a negligible magnitude can be found in Section 11.8.2.3.

Blench's Equation

The Blench equation, as presented in Pemberton and Lara (1984), is more applicable to clear-water flow conditions when there is a reservoir, sand and gravel pit, or hydraulic structure upstream that will significantly reduce the sediment supply.

Blench's equation is as follows:

$$Z_{general} = Z\frac{q_f^{2/3}}{F_{b0}^{1/3}} \qquad (11.57)$$

where:

$Z_{general}$ = general scour depth, ft;

q_f = design flood discharge per unit width (= design flood discharge divided by flow average width; the flow average width can be defined as wetted cross sectional area divided by flow depth where flow depth can be the Manning's equation-based normal depth or maximum flow depth from HEC-RAS), cfs/ft;

Z = multiplying factor (0.6 for a straight channel reach and 1.25 for a vertical rock bank or wall. If there is a bend, the bend scour equation by Zeller should be used to compute the bend scour; see Bend Scour Section 11.8.2.3); and

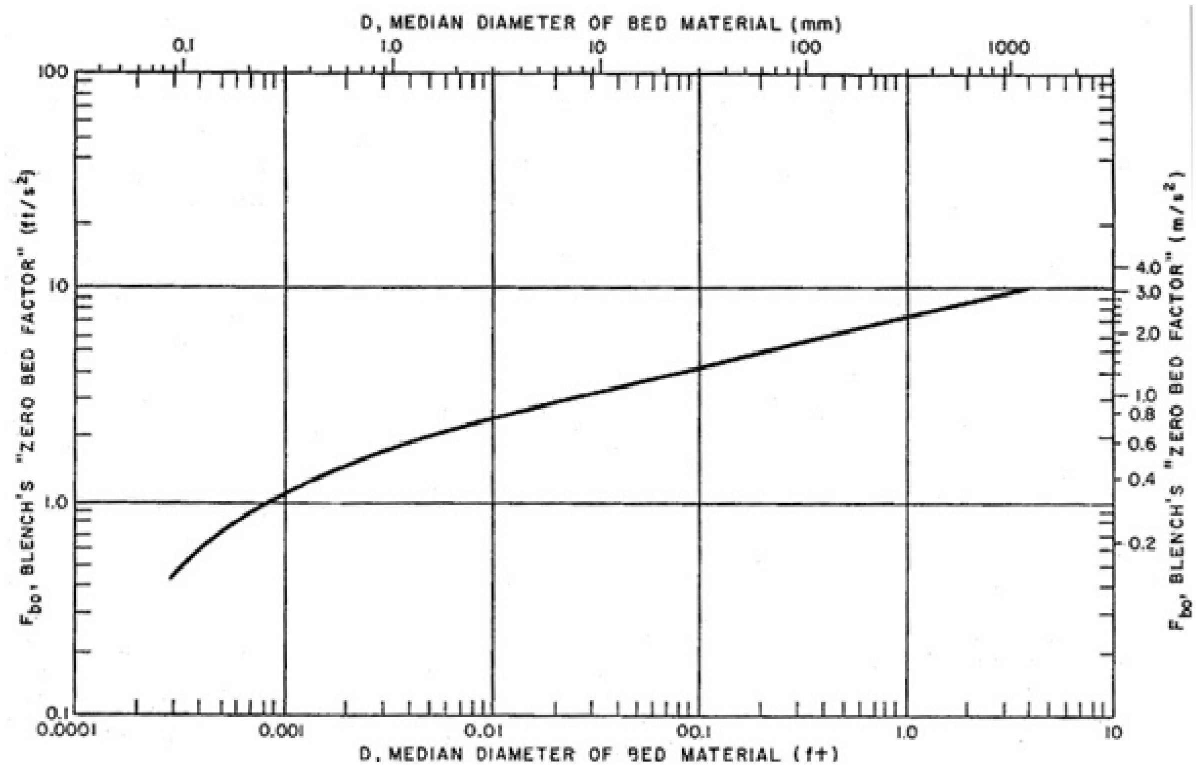
F_{b0} = Blench's zero bed factor from Figure 11.32 or from the equation developed by the FCDMC, which is:

$$F_{b0} = \begin{cases} 0.5672\ln(D_{50}) + 5.0302 & if\, D_{50} \le 0.0411 ft \\ 1.3698\ln(D_{50}) + 7.589 & if\, D_{50} > 0.0411 ft \end{cases} \qquad (11.58)$$

where:

D_{50} = median diameter, also defined as the diameter where 50% is finer by weight, ft.

FIGURE 11.32
CHART FOR ESTIMATING F_{b0} FOR THE BLENCH EQUATION
(Pemberton and Lara, 1984)



11.8.2.3 Bend Scour

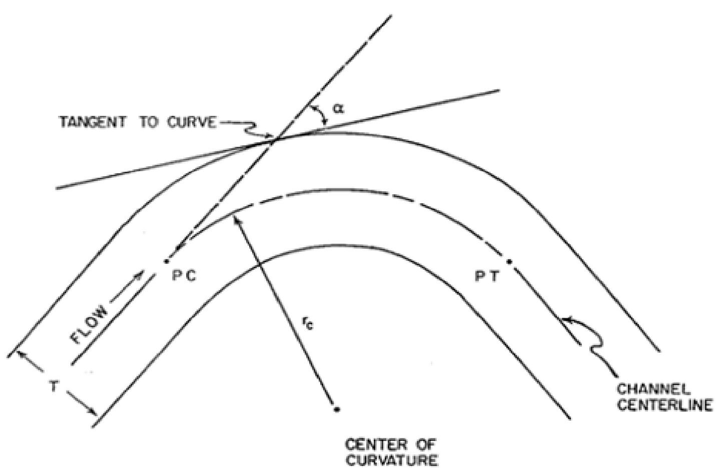
Bend scour may need to be estimated if it is not included as a component of general scour. For sand-bed watercourses, Zeller (1981) presents a bend scour equation. General scour by the Neil equation, or Lacey equation may include a bend scour component if certain coefficients (factor Z) are selected (Pemberton and Lara, 1984). However, if the Blench general equation is used, Zeller's bend scour equation should be applied. The higher value between Neill's equation (with an appropriate bend coefficient Z) and the HEC-18 contraction scour equation with Zeller's bend scour equation should be used. Zeller's bend scour equation for sand-bed watercourses (Simons, Li and Associates, 1985) is:

$$Z_{bend} = \frac{0.0685 y V_a^{0.8}}{y_h^{0.4} S_e^{0.3}}\left[2.1\left(\frac{\sin^2(\alpha/2)}{\cos(\alpha)}\right)^{0.2} - 1\right] \tag{11.59}$$

where:

Z_{bend}	=	bend scour depth, ft;
	=	0 when $r_c/T \geq 10.0$ or $\alpha \leq 17.8^o$;
	=	computed value when $0.5 < r_c/T < 10.0$ or $17.8^o < \alpha < 60^o$;
	=	computed value at $\alpha = 60^o$ when $r_c/T \leq 0.5$ or $\alpha \geq 60^o$;
r_c	=	centerline of channel radius of curvature, ft;
T	=	channel top width, ft;
V_a	=	average velocity of flow immediately upstream of bend, ft/s;
y	=	maximum depth of flow immediately upstream of bend, (ft) (= normal depth from Manning's or maximum channel depth from HEC-RAS);
y_h	=	hydraulic depth of flow immediately upstream of bend, ft;
S_e	=	energy slope immediately upstream of bend, ft/ft; and
α	=	angle formed by the projection of the channel centerline from the point of curvature to a point which meets a line tangent to the outer bank of the channel (see Figure 11.33), degrees.

FIGURE 11.33
SKETCH OF CHANNEL BEND
(Simons, Li and Associates, 1985)



PT = Downstream point of tangency to the centerline radius of curvature.
PC = Upstream point of curvature at the centerline radius of curvature.

The bend scour equation should be applied to the entire channel reach through the bend. It should also be applied to a certain distance downstream from the end of the bend because secondary currents will still cause scour in the downstream reach. The distance from the end of a bend at which the secondary currents will have decayed to a negligible magnitude can be estimated per Simons, Li and Associates, 1985, as:

$$X = 2.3\left(\frac{C}{\sqrt{g}}\right)Y \tag{11.60}$$

where:

X = distance from the end of channel curvature (point of tangency, PT) to the downstream point at which secondary currents have dissipated feet;

C = Chezy coefficient = $\frac{1.486}{n}R^{1/6}$
where *R* is the hydraulic radius;

g = gravitational acceleration, 32.2 feet/second^2; and

Y = depth of flow (to be conservative, use maximum depth of flow, including superelevation and exclusive of scour, within the bend), feet.

This equation may also be used to determine the distance when the Neill equation and the Lacey equation include the bend scour.

11.8.2.4 Bedform Scour

Bedforms develop in alluvial channels in response to the hydraulics of the flowing water and they are part of the mechanics of sediment transport. Bedforms are of various configurations and typically they consist of alternating "mounds" and "troughs," and being mobile, they move longitudinally along the bed of the watercourse. A bedform trough is a component of total scour and should be accounted for under appropriate conditions. The component of scour that is associated with bedforms is equal to one-half of the bedform amplitude (vertical distance from top of mound to bottom of trough) as shown in the following equation:

$$Z_{bedform} = 0.5d_h \tag{11.61}$$

where:

$Z_{bedform}$ = bedform scour depth, ft;

d_h = dune or antidune height (measured from mound top to trough bottom), ft.

Dunes form during lower regime flow, typically at Froude Numbers, F_r, less than 0.7. The Froude Number is defined as $V_a / \sqrt{g y_h}$ where V_a is the average channel velocity, g is gravitational acceleration (32.2 feet/second^2); and y_h is the hydraulic depth. Antidunes form during the upper regime flow where F_r is greater than or equal to 1.0 and may form during the transition from lower to upper regime flows. In the transition region where F_r is between 0.7 and 1.0, the larger of either dune or antidune height should be used.

The dune height equation for lower regime flow where $F_r < 0.7$ is shown per Gyr and Hoyer, 2006 and Zanke, 1976, as:

$$0.15 < \frac{d_h}{y_h} < 0.3 \tag{11.62}$$

where:

d_h = dune measured from mound top to trough bottom, ft;

y_h = hydraulic depth of flow, ft.

Since a range is given for dune height in the above equation, engineering judgment should be exercised to judiciously select a dune height within the given range.

The anti-dune height equation (based on Kennedy, 1961) for upper regime flow where $F_r > 1.0$ is shown per Simons, Li & Associates, 1985, as:

$$d_h = 0.027 V_a^2 \tag{11.63}$$

where:

d_h = antidune height measured from mound top to trough bottom, ft;

V_a = average channel velocity, ft/s.

When $1.0 \geq F_r \geq 0.7$, the higher value between the dune height equation and anti-dune height equation should be used.

11.8.2.5 Low-Flow Incisement Scour

The normal irregularities in the bed of a watercourse (both natural and man-made) result in the formation of a low-flow channel. The channel is formed by the predominance of a low-flow condition or due to low-flows that persist after a flood. The magnitude of low-flow incisement may best be estimated by a representative field assessment. In the absence of field data, or for planning and design purposes, low-flow incisement should be estimated as no less than 1 foot and possibly in excess of 2 feet. A lower value can be used for small and minor watercourses and a higher value should be used for regional watercourses. When there is channelization where the channel bed is graded, the low flow channel depth may be estimated by assuming a small peak discharge (2-year event) for a simple chart that relates the depth to the channel-forming discharge (Figures 5-10 in USACE, 1994). If the low-flow channel is very stable and the toe-down or total scour is measured from the channel thalweg (lowest elevation in the entire cross section), this scour component may be ignored in the total scour computation. However, engineering judgment must be carefully exercised to avoid over-estimation or under-estimation of low-flow channel depths.

11.8.2.6 Local Scour

Local scour is a component of total scour that is caused by flow acceleration and vortices due to flow obstruction and impingement. Most local scour and deposition is caused by man-made structures such as culvert outlets (Photograph 11.11 and Photograph 11.12), bridge piers/abutments, bridge guide banks, grade controls, drop structures, sand/gravel mining pits, and other structures.

PHOTOGRAPH 11.11
LOCAL SCOUR AT UNPROTECTED CULVERT OUTLET.

Generally, local scour depths are much larger than long-term degradation or general scour. However, if there are major changes in watercourse conditions, such as a water storage facility built upstream or downstream or severe straightening of the watercourse, long term bed elevation changes can be the larger element in the total scour estimate.

Bridge local scour and culvert outlet local scour are discussed in the following sections. The estimation of bridge guide bank scour is similar to the bridge abutment scour estimation procedure (USDOT, 2001b). When estimating the bank protection scour or abutment scour and if the bank protection or abutment is close to piers, scour hole influence zones should be computed. The scour due to influence zones at the bank protection or abutment should be added to the total scour. A certain minimum distance, based on engineering judgment, between the piers and abutments should be preserved to minimize the scour impact to each other.

Local scour downstream of a hydraulic structure can be estimated by empirical equations from Schoklitsch (1932), Veronese (1937), and Zimmerman and Maniak (1967). Pemberton and Lara (1984) or the original references should be consulted and engineering judgment should be exercised when selecting or applying any of these equations.

PHOTOGRAPH 11.12
CULVERT CAUSES BACKWATER RESULTING IN UPSTREAM AGGRADATION.

For a submerged structure, the local scour depth can be estimated by the Simons, Li & Associates (1986) equations. These equations are a function of grade control structure face slope, drop height and other hydraulic parameters, but are independent of bed material grain size. These equations may overestimate scour depth for coarse bed material watercourses. Simons, Li & Associates (1986) should be consulted when using these equations.

In this chapter, the scour caused by sand and gravel mining operations is classified as local scour. There are two types of erosion caused by sand and gravel mining. One is erosion that starts from the pit's upstream brink point and moves upstream, which is called headcut. Another is erosion that starts from the pit's downstream brink point and moves downstream, which is called tailcut. The estimation of headcut and tailcut may be done by both empirical equations and sediment transport modeling. The available empirical equations are the methodology developed for the Arizona Department of Transportation (ADOT) by Li, et al. in 1989. However, it has been found that the methodology provides a reasonable estimate for headcut scour depth at the knickpoint but may under-estimate the headcut distance (FCDMC, 2006). Sediment transport modeling can be performed to estimate the headcut and tailcut by using HEC-6, HEC-6T, FLUVIAL-12 or other FCDMC-approved models. However, since most models are one-dimensional, when the pit width is much smaller than the river width, the model input file may need to be set up in a way that flow can be confined in a corridor that is equivalent to the pit width.

Local Scour at Bridge Piers

Local scour at bridge piers is calculated with the CSU equation (USDOT, 2001b). The basic pier scour equation is discussed here. Other equations for more complicated pier conditions can be found in USDOT (2001b). The basic pier scour equation is:

$$\frac{Z_{local}}{a} = 2.0K_1K_2K_3K_4\left(\frac{y_1}{a}\right)^{0.35}Fr^{0.43} \qquad (11.64)$$

where:

Z_{local} = local scour depth for piers, ft;

y_1 = flow depth directly upstream of the pier, ft (= normal depth from Manning's equation; maximum channel depth from HEC-RAS);

K_1 = correction factor for pier nose shape from Table 11.10 and Figure 11.34;

K_2 = correction factor for angle of attack of flow from Table 11.11 and discussion below;

K_3 = correction factor for bed condition from Table 11.12 (note: if the bed form scour is already computed based on bed form trough depth, then K_3 should be set to 1.0 to avoid double-counting of the bed condition scour);

K_4 = correction factor for armoring by bed material size, see discussion below;

a = pier width, ft;

L = length of pier, ft;

g = gravitational acceleration, 32.2 ft/s^2;

Fr = Froude Number directly upstream of the pier = $V_1/(gy_1)^{1/2}$; and

V_1 = mean velocity of flow directly upstream of the pier, ft/s.

TABLE 11.10
CORRECTION FACTOR, K_1, FOR PIER NOSE SHAPE
(USDOT, 2001b)

Shape of Pier Nose	K_1
(a) Square nose	1.1
(b) Round nose	1.0
(c) Circular cylinder	1.0
(d) Group cylinders	1.0
(e) Sharp nose	0.9

FIGURE 11.34
PIER NOSE SHAPE
(USDOT, 2001b)

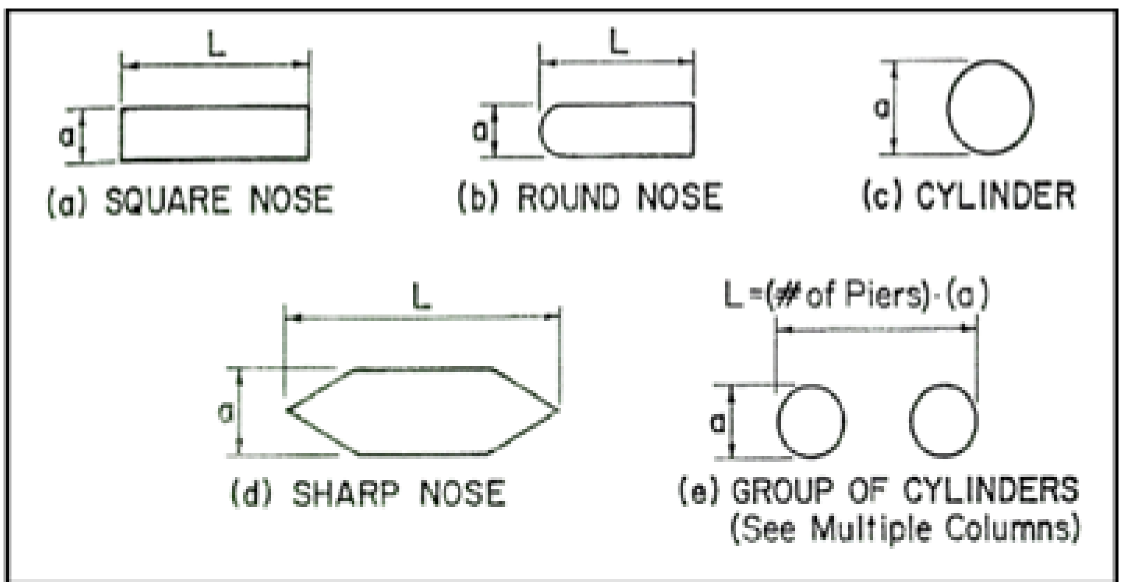


TABLE 11.11
CORRECTION FACTOR, K_2, FOR FLOW ANGLE OF ATTACK
(USDOT, 2001b)

Angle (degree)	L/a=4	L/a=8	L/a=12
1	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0
Angle = skew angle of flow L = length of pier, ft a = pier width, ft			

The following formula can also be used to estimate the K_2 factor:

$$K_2 = \left(\cos\theta + \frac{L}{a}\sin\theta\right)^{0.65} \tag{11.65}$$

where:

θ = angle of attack, degrees. If $L/a > 12$, use $L/a = 12$.

TABLE 11.12
CORRECTION FACTOR, K_3, FOR BED FORM CONDITION
(USDOT, 2001b)

Bed Condition	Dune Height (ft)	K_3
Clear-Water Scour	N/A	1.1
Plane bed and Antidune flow	N/A	1.1
Small Dunes	$3 > H \geq 0.6$	1.1
Medium Dunes	$9 > H \geq 3$	1.2 to 1.1
Large Dunes	$H \geq 9$	1.3

The following formulas are to be used to estimate the K_4 factor:

if $D_{50} < 2\ mm$ or $D_{95} < 20\ mm$, then $K_4 = 1$, or

if $D_{50} \geq 2\ mm$ and $D_{95} \geq 20\ mm$, then:

$$K_4 = 0.4(V_R)^{0.15} \tag{11.66}$$

where:

$$V_R = \frac{V_1 - V_{icD_{50}}}{V_{cD_{50}} - V_{icD_{95}}} > 0 \tag{11.67}$$

where:

V_1	=	velocity of the approach flow just upstream of the pier, ft/s;
V_{icD_x}	=	approach velocity ft/s required to initiate scour at the pier for the grain size D_x, ft;
D_x	=	grain size for which x percent of the bed material is finer, ft; and
x	=	50 or 95.

The approach velocity is calculated with the equation:

$$V_{icD_x} = 0.645\left(\frac{D_x}{a}\right)^{0.053} V_{cD_x} \tag{11.68}$$

where:

V_{cD_x}	=	critical velocity (m/s or ft/s) for incipient motion for the grain size D_x, ft; and
V_{cD_x}	=	$11.17 y_1^{1/6} D_x^{1/3}$

where:

y_1	=	depth of flow just upstream of the pier, excluding local scour, ft (normal depth from Manning's equation; maximum channel depth from HEC-RAS); and
D_x	=	grain size for which x percent of the bed material is finer, ft.

While K_4 provides a good fit with the field data, the velocity ratio terms are so formed that if D_{50} is held constant and D_{95} increases, the value of K_4 increases rather than decreases. For field data an increase in D_{95} was always accompanied with an increase in D_{50}. The minimum value of K_4 is 0.4.

Local Scour at Abutments

Froehlich's equation is used to estimate the local scour at bridge abutments when the ratio of the length of the abutment (normal to flow) to flow depth $L/y_a \le 25$ (USDOT, 2001b). It has the form:

$$\frac{Z_{local}}{y_a} = 2.27K_1K_2\left[\frac{L'}{y_a}\right]^{0.43}Fr^{0.61} + 1 \tag{11.69}$$

where:

Z_{local} = local scour depth for abutments, ft;

y_a = average depth of flow on the floodplain (A_e/L), ft;

K_1 = coefficient for abutment shape from Figure 11.35 and Table 11.13;

K_2 = coefficient factor for angle of embankment to flow;

K_2 = $(\theta/90)^{0.13}$ (see Figure 11.36 for the definition of θ);

$\theta < 90°$ if embankment points downstream;

$\theta > 90°$ if embankment points upstream;

L' = length of active flow obstructed by the embankment, ft, see Figure 11.37;

L = length of embankment projected normal to the flow, ft, see Figure 11.37;

Fr = Froude number of approach of the abutment = $V_e/(gy_a)^{1/2}$;

V_e = Q_e/A_e, ft/s;

A_e = flow area of the approach cross section obstructed by the embankment, ft^2;

Q_e = flow obstructed by the abutment and approach embankment, cfs; and

g = gravitational acceleration, 32.2 ft/s^2.

The HIRE equation is used when the ratio of the length of the abutment (normal to flow) to flow depth $L/y_a > 25$, (USDOT, 2001b). It has the form:

$$\frac{Z_{local}}{y_1} = 4Fr^{0.33}\frac{K_1}{0.55}K_2 \tag{11.70}$$

where:

Z_{local}	=	local scour depth, ft;
y_1	=	depth of flow at the abutment on the overbank or in the main channel, ft;
K_1	=	coefficient for abutment shape from Figure 11.35 and Table 11.13;
K_2	=	$(\theta/90)^{0.13}$ (see Figure 11.36 for the definition of θ);
		$\theta < 90°$ if embankment points downstream;
		$\theta > 90°$ if embankment points upstream; and
Fr	=	Froude number based on the velocity and depth adjacent to and upstream of the abutment.

FIGURE 11.35
COMMON ABUTMENT SHAPES
(USDOT, 2001b)

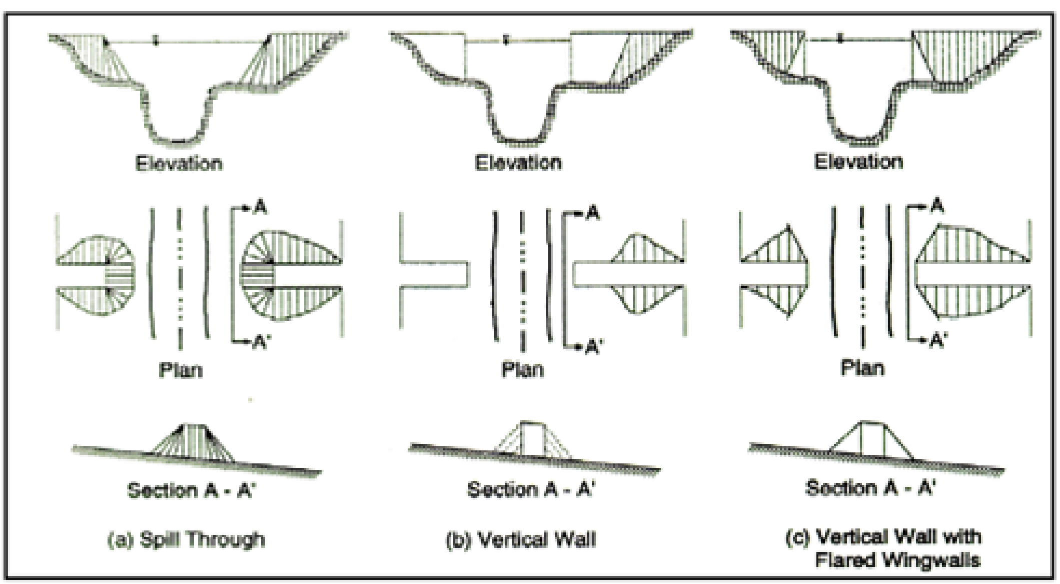


TABLE 11.13
ABUTMENT SHAPE COEFFICIENTS
(USDOT), 2001b)

Description	K_1
Vertical-wall abutment	1.00
Vertical-wall abutment with wing walls	0.82
Spill-through abutment	0.55

FIGURE 11.36
ABUTMENT SKEW; FOR ABUTMENTS ANGLES UPSTREAM
(USDOT, 2001b)

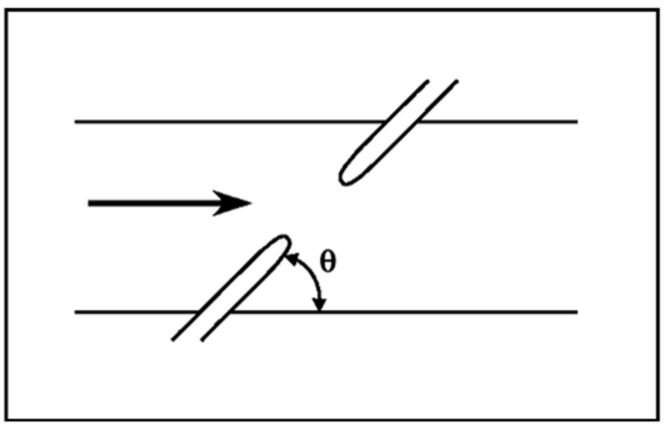
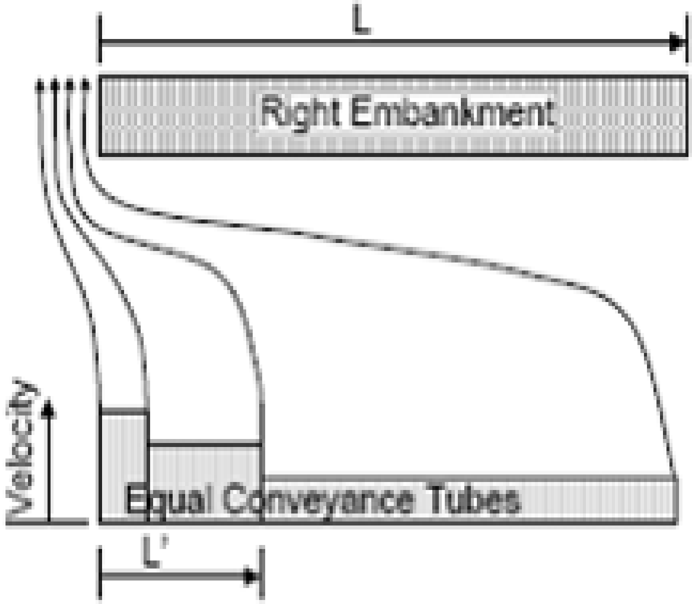


FIGURE 11.37
LENGTHS OF EMBANKMENT
(USDOT, 2001b)



Local Scour at Guide Banks

Local scour at guide banks can be estimated by the local scour at abutment equations (USDOT, 2001b). When the ratio, L/y_a, between the embankment projected length, L, normal to the flow and floodplain average depth, y_a, is less than or equal to 25, Froehlich's abutment equation can be used for guide bank local scour estimation. Assuming the guide banks are similar to spill through abutments and the abutment angle is 90 degrees, K_1 becomes 0.55 and K_2 becomes 1.0. Thus, Froehlich's abutment scour equation (if $L/y_a \le 25$) can be simplified as:

$$\frac{Z_{local}}{y_a} = 1.248\left[\frac{L'}{y_a}\right]^{0.43} Fr^{0.61} + 1 \tag{11.71}$$

where:

Z_{local} = local scour depth, ft;

y_a = average depth of flow on the floodplain (A_e/L), ft;

L' = length of active flow obstructed by the embankment, ft, see Figure 11.37;

L = length of embankment projected normal to the flow, ft, see Figure 11.37;

Fr = Froude number of approach to the abutment = $V_e/(gy_a)^{1/2}$;

V_e = Q_e/A_e ft/s;

A_e = flow area of the approach cross section obstructed by the embankment, ft^2;

Q_e = flow obstructed by the abutment and approach embankment, cfs; and

g = gravitational acceleration, 32.2 ft/s^2.

When the ratio, L/y_a, between the embankment projected length, L, normal to the flow and floodplain average depth, y_a, is greater than 25, the HIRE equation becomes:

$$\frac{Z_{local}}{y_1} = 4.0Fr^{0.33} \quad (11.72)$$

where:

Z_{local} = local scour depth, ft;

y_1 = depth of flow at the abutment on the overbank or in the main channel, ft; and

Fr = Froude number based on the velocity and depth y_1 adjacent to and upstream of the guide bank.

Local Scour at Culvert Outlets

The equation to calculate the local scour at a culvert outlet in cohesionless soil has the form (USDOT, 2006):

$$\left(\frac{Z_{local}}{R_c}, \frac{W_{local}}{R_c}, \frac{L_{local}}{R_c}, \frac{V_{local}}{R_c^3}\right) = C_s C_h \left(\frac{\alpha}{\sigma^{1/3}}\right)\left(\frac{Q}{\sqrt{g R_c^{2.5}}}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta} \quad (11.73)$$

where:

Z_{local} = depth of scour, ft;

W_{local} = width of scour, ft;

L_{local} = length of scour, ft;

V_{local}	=	volume of scour, ft;
R_c	=	hydraulic radius at the end of the culvert (assuming full flow), ft;
Q	=	discharge, cfs;
g	=	acceleration of gravity, 32.2 ft/s^2;
t	=	time of scour in minutes (30 minutes recommended);
t_o	=	base time (316 minutes);
σ	=	$(D_{84}/D_{16})^{0.5}$, material standard deviation;
D_{16}	=	median sediment particle diameter for which 16% of the material (by weight) is finer, ft;
D_{84}	=	median sediment particle diameter for which 84% of the material (by weight) is finer, ft;
α, β, θ		are coefficients, see Table 11.14;
C_s	=	slope correction coefficient, see Table 11.15; and
C_h	=	drop height adjustment coefficient, see Table 11.16.

If the soil is cohesive in nature the above equation should not be used. The equation in Section 5.2 (pages 5-6) of USDOT **(**2006) should be used.

TABLE 11.14
COEFFICIENTS FOR CULVERT OUTLET SCOUR IN COHESIONLESS SOILS
(USDOT, 2006)

	α	β	θ
Depth, Z_{local}	2.27	0.39	0.06
Width, W_{local}	6.94	0.53	0.08
Length, L_{local}	17.1	0.47	0.1
Volume, V_{local}	127.08	1.24	0.18

TABLE 11.15
COEFFICIENT, C_S, FOR CULVERT SLOPE
(USDOT, 2006)

Slope %	Depth	Width	Length	Volume
0	1	1	1	1
2	1.03	1.28	1.17	1.3
5	1.08	1.28	1.17	1.3
>7	1.12	1.28	1.17	1.3

TABLE 11.16
COEFFICIENT, C_H, FOR OUTLETS ABOVE THE BED
(USDOT, 2006)

H_d[1]	Depth, Z_{local}	Width, W_{local}	Length, L_{local}	Volume, V_{local}
0	1	1	1	1
1	1.22	1.51	0.73	1.28
2	1.26	1.54	0.73	1.47
4	1.34	1.66	0.73	1.55
[1]H_d is the height above bed in pipe diameters.				

Local Scour at Grade Controls or Drop Structures

The equations for scour below a structure are those of Schoklitsch, Veronese, and Zimmerman and Maniak (Pemberton and Lara, 1984).

Schoklitsch Equation

The Schoklitsch equation was developed to calculate scour depth below a structure with a free overfall of water on an unprotected river bed. It can also be used for evaluating local scour below a sharp-crested spillway, drop structure or grade control structure. It has the form:

$$Z_{loc} = \frac{3.15H^{0.2}q^{0.57}}{D_{90}^{0.32}} - y_m \qquad (11.74)$$

where:

Z_{local} = depth of scour, ft;

H = vertical distance between the water level upstream and downstream of the structure, ft;

q = design discharge per unit width (= design discharge divided by average flow width; the average flow width can be defined as wetted cross section area divided by flow depth where flow depth can be the Manning's equation-based normal depth or maximum flow depth from HEC-RAS), cfs/ft;

D_{90} = particle size for which 90% is finer than, mm; and

y_m = downstream mean water depth, (hydraulic depth) ft.

Veronese Equation

The Veronese equation for computing the scour depth below a low head stilling basin design is in the form:

$$Z_{local} = 1.32 H_T^{0.225} q^{0.54} - y_m \qquad (11.75)$$

where:

Z_{local} = depth of scour, ft;

H_T = the head from upstream reservoir to tailwater level, ft;

q = design discharge per unit width (= design discharge divided by average flow width; the average flow width can be defined as wetted cross section area divided by flow depth where flow depth can be the Manning's equation-based normal depth or maximum flow depth from HEC-RAS), cfs/ft; and

y_m = downstream mean water depth (hydraulic depth), ft.

Zimmerman and Maniak Equation

The Zimmerman and Maniak equation for scour depth below a stilling basin or at the end of an apron is in the form:

$$Z_{local} = 1.95\left(\frac{q^{0.82}}{D_{85}^{0.23}}\right)\left(\frac{y_m}{q^{2/3}}\right)^{0.93} - y_m \qquad (11.76)$$

where:

y_{loc} = local scour depth below streambed, ft;

q = design discharge per unit width (= design discharge divided by average flow width; the average flow width can be defined as wetted cross section area divided by flow depth where flow depth can be the Manning's equation-based normal depth or maximum flow depth from HEC-RAS), cfs/ft; and

D_{85} = particle size for which 85% is finer than, mm; and

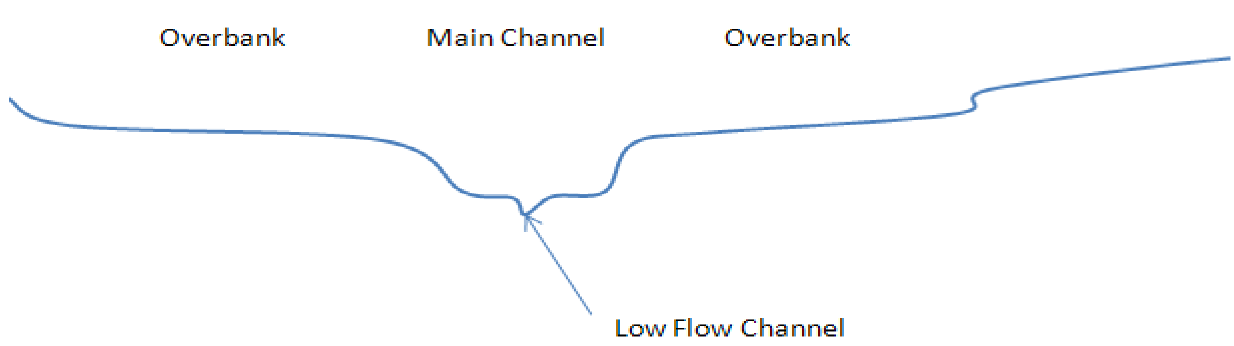
y_m = downstream mean water depth (hydraulic depth), ft.

11.9 ESTIMATING LATERAL-EROSION HAZARD ZONES

This section presents a methodology for estimating the 100-year lateral-erosion or lateral-migration hazard zones for straight or meandering natural channels in Maricopa County, Arizona. The terms lateral-erosion and lateral-migration are interchangeable in this section.

A typical natural channel has a main-channel and overbank areas (Figure 11.38). A main-channel usually has well-defined channel banks. There may be low flow channels within the main-channel. Both the main-channel and any low-flow channels may be meandering. The methodology should be applied to the main-channel instead of the low-flow channel.

FIGURE 11.38
TYPICAL CROSS SECTION FOR A NATURAL CHANNEL

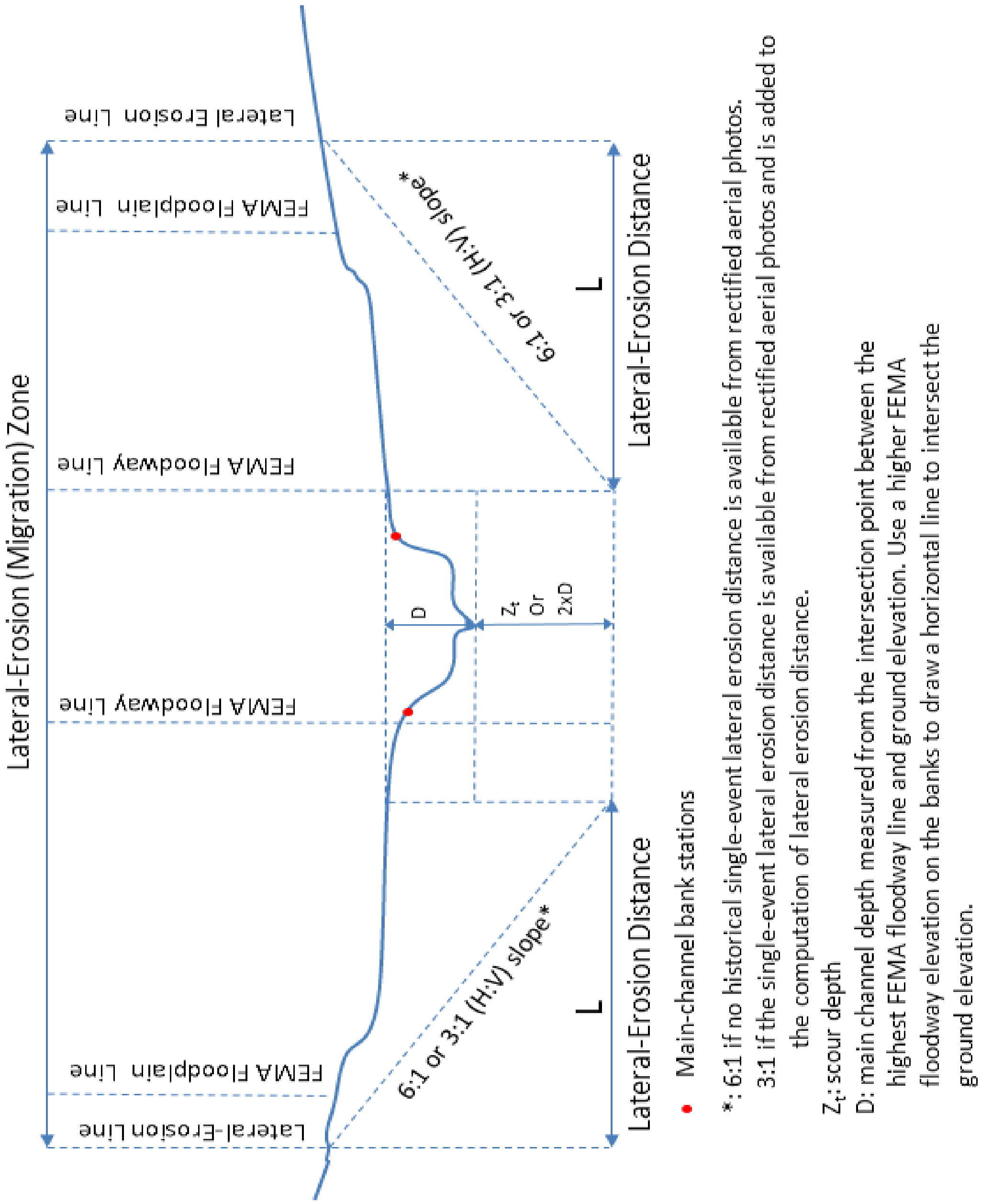


As illustrated in Figure 11.39, the lateral-erosion zone is determined by the lateral-erosion distances measured from the FEMA floodway lines, or the main-channel bank stations whichever gives a larger value. If a FEMA floodway has not been delineated, a HEC-RAS model should be developed to estimate the 100-year floodway line. The main-channel bank locations can be determined by the standard approach for selecting main-channel bank stations in HEC-RAS modeling. If a FCDMC-approved HEC-RAS model exists, the bank stations in the hydraulic model should be used as the main-channel bank stations. Under some special conditions, the main-channel bank stations may be used instead of the floodway lines with a prior approval by FCDMC staff. One special condition may be a sand and gravel mining pit that is located near an outdated FEMA floodway line and the pit owner owns the property adjacent to the lateral-erosion zone.

When the FEMA floodway elevations and bank station elevations on both banks are not the same, the highest elevation among floodway elevations and bank station elevations should be used to determine the starting point for measuring the lateral-erosion distance. For example, Figure 11.39 shows that the FEMA floodway elevation on the right bank (looking into the paper) is higher than the left FEMA floodway elevation and the two bank station elevations. It should be used as the starting point for measuring the lateral-erosion distance for the right bank. For the left bank, a horizontal line should be drawn to intersect the left bank. The intersection point should be used as the starting point for measuring the left lateral-erosion distance.

.

FIGURE 11.39
LATERAL EROSION ZONES FOR TYPICAL STRAIGHT CHANNELS



The methodology presented herein is developed by adapting the City of Austin's "Guidance on Establishing an "Erosion Hazard Zone" for Structure and Utility Locations near Streams" (City of Austin, September, 2007) and "Erosion Setback and Stabilization Criteria for City of Austin Streams" (Ayres Associates, 2004). Essentially, the lateral-erosion distance is three times or six times the main-channel potential incision depth measured from the FEMA floodway line or channel bank line whichever is greater. When historical aerial photos are available to estimate the historical lateral-erosion distance due to a single flood event, the lateral-erosion distance is the largest lateral-erosion distance due to the single flood event plus three times the main-channel potential incision depth. When historical aerial photos are not available, the lateral-erosion distance is six times the main-channel potential incision depth. The slope of 6:1 (horizontal:vertical) is Ayres (2004) recommendation for alluvial channels. The details for computing lateral-erosion distance are discussed in the rest of this section.

The procedure is presented for several scenarios which are (1) a straight channel reach when sufficient historical aerial photos are available to estimate the largest single-event lateral-erosion distance; (2) a straight channel reach when there are not sufficient historical aerial photos to estimate the largest single-event lateral-erosion distance; (3) when the project site is located between upstream and downstream bends; and (4) when the project site is located on a straight reach, but upstream or downstream of a bend. Areas that are within the FEMA floodway are considered to be automatically within the lateral-erosion zone. Lateral-erosion is assumed to be limited by man-made or natural physical constraints such as bank protection, bedrock, or other lateral-erosion-resistant features. However, detailed engineering and geologic analyses must be performed in order to remove an area from the lateral-erosion zone because of man-made or natural physical constraints. Since the methodology is related to the FEMA 100-year floodplain and scour analysis for a 100-year flood, the results can be considered as the 100-year lateral-erosion hazard zone.

11.9.1 Straight Channel Reach (historical aerial photos available)

Historical aerial photos should be sought. Historical aerial photos are available for most of the Metro Phoenix area and most large rivers from FCDMC, Arizona State University libraries, and other federal, county, and local agencies. The Maricopa County web site (www.maricopa.gov) also has a free online GIS application that displays some historical aerial photos. If no aerial photos are found, historical aerial photos at a different segment of the same river or photos for nearby similar rivers may be used.

When the largest single-event lateral-erosion distance can be estimated based on geo-referenced historical aerial photos for a time period of at least 50 years, the lateral-erosion distance, L, is estimated as the sum of the largest single-event lateral-erosion distance, L_H, and 3 times the main-channel potential incision depth. The largest single-event lateral-erosion distance may be available from existing watercourse master plans or can be estimated by comparing historical aerial photos. The main-channel potential incision depth is estimated as three times the channel depth, D, in feet (Figure 11.39) if the FCDMC total scour estimation method (Section 11.8.2) is not applied. It is the distance vertically measured from the channel thalweg to the highest elevation of the FEMA floodway elevations on both banks or the highest channel bank elevations when the FEMA floodway does not exist. Thus, the lateral-erosion distance, L (feet), is estimated by:

$$L = L_H + 3(3D) = L_H + 9D \tag{11.77}$$

where:

L_H = the largest single-event lateral-erosion distance, in feet, and

D = the channel depth, in feet.

If the total scour depth for the 100-year flood event is estimated based on FCDMC's total scour estimation method (Section 11.8.2), the main-channel potential incision depth is estimated as the sum of the main-channel depth, D, and the total scour depth, Z_t, in feet. Thus, the lateral-erosion distance, L (feet) is estimated by:

$$L = L_H + 3(D + Z_t) \tag{11.78}$$

where:

L_H = the largest single-event lateral erosion distance in feet,

D = the channel depth in feet, and

Z_t = the total scour depth.

11.9.2 Straight Channel Reach (historical aerial photos not available)

In general, this section is applicable to small washes when historical aerial photos cannot be found after extensive effort is taken to seek them. When historical aerial photos are not available, the lateral-erosion distance is six times the main-channel potential incision depth. The main-channel potential incision depth is estimated as three times the channel depth, D, in feet (see Figure 11.39). It is the distance vertically measured from the channel thalweg to the highest elevation of the FEMA floodway elevations on both banks or the highest channel bank elevations when the FEMA floodway does not exist. The channel lateral-erosion distance, L in feet, is estimated by:

$$L = 6(3D) = 18D \tag{11.79}$$

where:

D = the channel depth in feet.

If total scour depth for a 100-year flood event is estimated based on FCDMC's total scour estimation method presented in this chapter, the main-channel potential incision depth is estimated as the sum of the main-channel depth, D, and the total scour depth, Z_t, in feet. Total scour depth is measured from the thalweg of the channel. The lateral-erosion distance, L in feet, is estimated by:

$$L = 6(D + Z_t) \tag{11.80}$$

where:

D = the channel depth in feet, and

Z_t = is the total scour depth in feet.

11.9.3 Channel Located Between Upstream and Downstream Bends

If there is a bend upstream and downstream of a project site (illustrated by the red dot in Figure 11.40, the lateral-erosion distances at meander peaks are first determined by the procedure discussed above. Then, the lateral-erosion zone for this channel reach is determined by connecting the lateral-erosion distance ending points as illustrated by the red dashed lines in Figure 11.40. It should be pointed out that the bend scour should be included based on the total scour estimation method presented in this chapter.

11.9.4 Straight Channel Reach Upstream and Downstream of a Bend

If a project site is on a straight channel reach, but located downstream of a bend, the lateral-erosion distance should be estimated for both the bend (the peak of the bend) and straight reach at a location of about one wave length downstream of the peak of the bend by the same procedure discussed above. As indicated by Leopold et al. (1964), the meander length (wave length) is approximately 10 to 14 channel widths. Thus, wave length can be estimated by multiplying the meandering channel width by 14. The channel width is defined as the distance between FEMA floodway lines or bank stations whichever is greater, measured perpendicular to flow direction. If the project site is on a straight reach, but located upstream of a bend, the lateral-erosion distance should be estimated for both the bend (the peak of the bend) and the straight reach at a location of one wave length upstream of the peak of the bend by the same procedure discussed above. Figure 11.41 illustrates these two cases. It should be pointed out that the bend scour should be included based on the total scour estimation method presented in this chapter.

FIGURE 11.40
LATERAL-EROSION ZONE FOR A CURVED CHANNEL

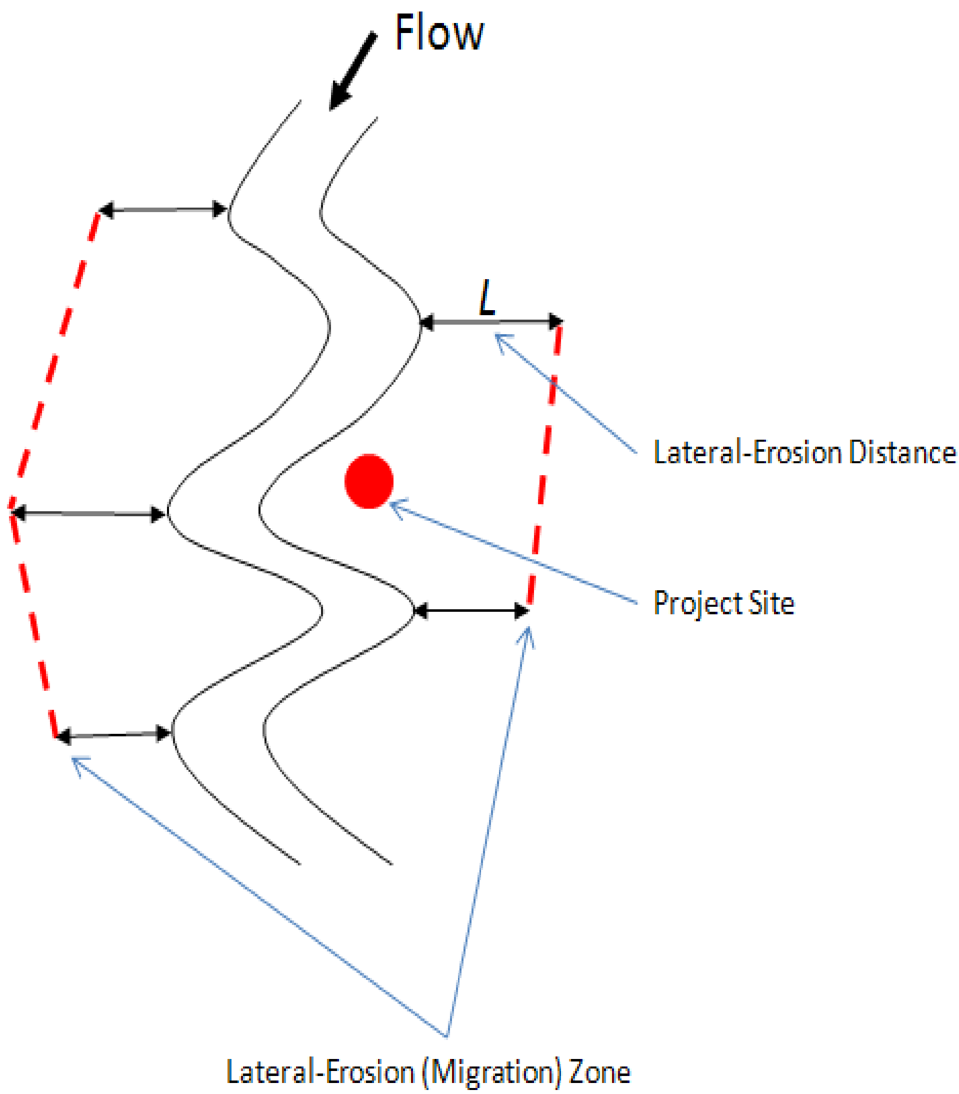
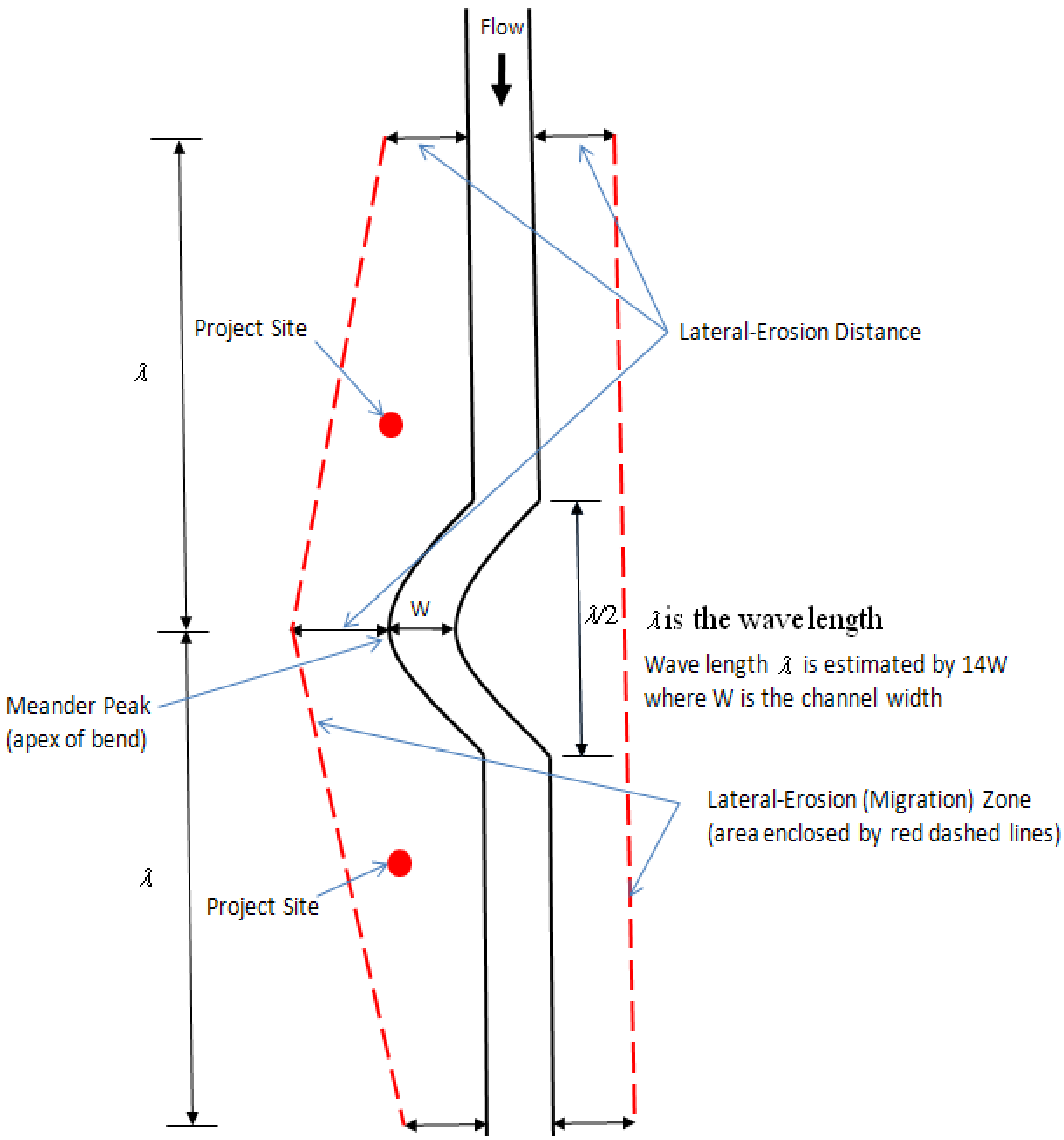


FIGURE 11.41
LATERAL EROSION ZONE FOR STRAIGHT AND CURVED CHANNELS



11.9.5 Application Limitations and Design Implications

The lateral-erosion hazard zone methodology presented herein is straightforward, practical, and reproducible. However, the methodology should be only applicable to straight or meandering natural channels. In general, the methodologyis is not applicable to channels on alluvial fans because alluvial fans channels may move around due to avulsion. It should be pointed out that the estimated hazard zone can only be considered as a rough estimate for potential lateral-erosion hazard zone because of the very complex mechanisms for lateral-erosion. Adjustment may be necessary based on engineering judgment or additional engineering analyses.

The lateral-erosion can be controlled by man-made or natural physical constraints such as bank protection, bedrocks, or other lateral-erosion-resistant features. However, detailed geologic, hydraulic, river mechanic, and geotechnical engineering analyses must be performed in order to remove an area from the lateral-erosion zone. Although the lateral-erosion hazard zone methodology is reproducible, estimation of largest single-event lateral erosion can still be subjective especially when there are not sufficient photos or the time span is not more than fifty years. Aerial photos may be only available on a yearly basis or every five years. It is rare to have aerial photos right before and after a flood event. Engineering judgment plays a key role in this scenario. Typically, for a large river, a few hundred feet is a reasonable estimate for the lateral-erosion distance from one single event. The estimation may be obtained by analyzing aerial photos for similar rivers or different reaches in the same river. An erosion monitoring plan may be added as part of erosion protection measure when it is difficult to estimate the largest single event lateral-erosion distance based on available historical aerial photos.

11.9.6 Sand and Gravel Mining Operation Exception

For a sand and gravel mining site located between upstream and downstream meandering peaks as shown in Figure 11.40, an exception may be made to allow reasonable mining operations without adversely impacting the neighboring properties. If an erosion monitoring plan can be added as part of sand and gravel mining permit requirements to monitor each major flood event, the lateral-erosion hazard line may be developed without connecting the lateral-erosion distance ending points at the peaks of the bends. Figure 11.42 illustrates this situation. However, due to the greater uncertainty for channel avulsion at the meandering locations, additional setback should be added to the lateral-erosion distance equations discussed above. The modified lateral-erosion distance equation for the case when the largest single-event lateral-erosion distance is available can be estimated by:

$$L = L_H + 18D \tag{11.81}$$

where:

L_H = the largest single-event lateral-erosion distance, in feet, and

D = the channel depth, in feet.

When the total scour depth is computed based on FCDMC's total scour estimation method presented in this chapter, the modified lateral-erosion distance equation for the case when the largest single-event lateral-erosion distance is available can be estimated by:

$$L = L_H + 6(D + Z_t) \tag{11.82}$$

where:

L_H = the largest single-event lateral erosion distance in feet,

D = the channel depth in feet, and

Z_t = the total scour depth.

If the historical aerial photos are not available to estimate the largest single-event lateral-erosion distance, the modified lateral-erosion distance can be estimated by:

$$L = 36D \tag{11.83}$$

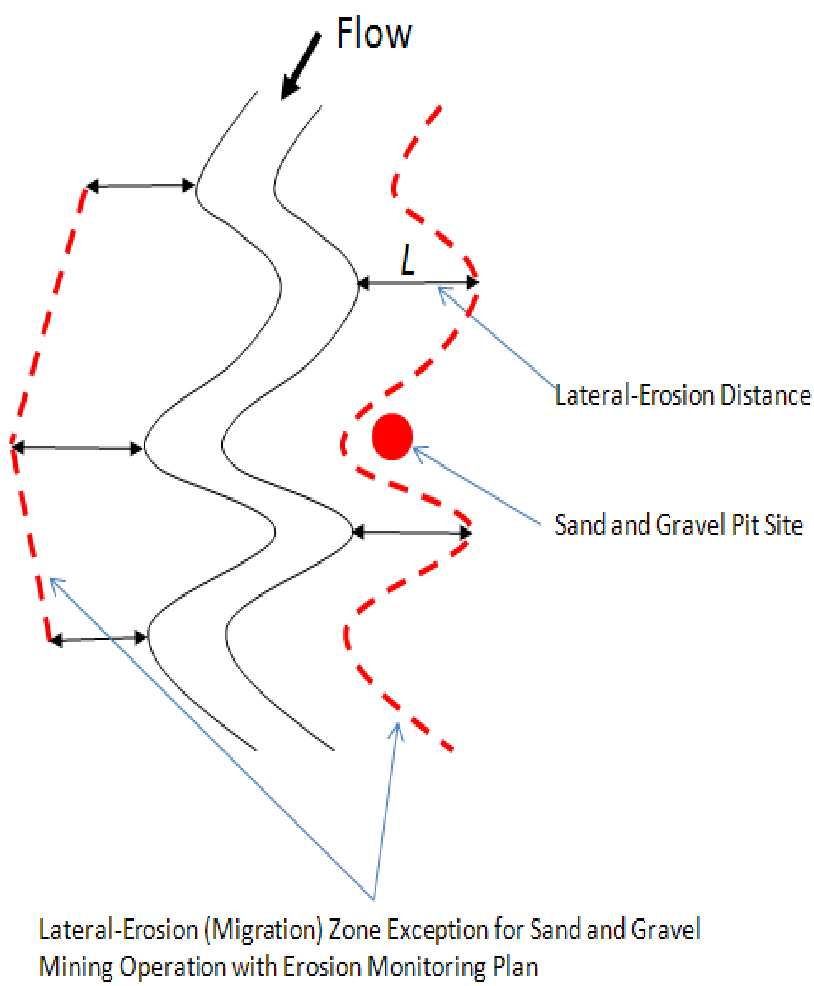
where D is the channel depth. If total scour depth for a 100-year flood event is estimated based on FCDMC's total scour estimation method presented in this chapter, the main-channel potential incision depth is estimated as the sum of the main-channel depth, D, and the total scour depth, Z_t, in feet. Total scour depth is measured from the thalweg of the channel. The modified lateral-erosion distance for the case when the historical aerial photos are not available can be estimated by:

$$L = 12(D + Z_t) \tag{11.84}$$

The lateral-erosion zone can be identified by using these modified lateral-erosion distance equations. When the lateral-erosion distance computed by these modified equations is less than 500 feet, a minimum distance of 500 feet is recommended for large rivers. The lateral-erosion zone can be used in toe-down design for engineering berms. For the engineering berms or levees that separate the pits from the river flow, erosion protection shall be provided on the river-side with toe-down protection depth which is the total scour depth. For areas at or within the lateral-erosion line for a sand and gravel mining site between upstream and downstream meandering peaks, the toe-down depth should be the total channel scour depth below the channel thalweg. For areas outside the lateral-erosion line, the toe-down depth should be the total channel scour depth below the adjacent ground if a monitoring plan is developed to monitor the safety and erosion potential for the erosion protection structures. The main channel velocity or floodway velocity should be used for scour calculations.

For a sand and gravel mining pit located in a straight channel reach, the toe-down requirement for erosion protection can be found in Section 11.8.2 Total Scour. The launchable riprap approach can be used instead of fully extending the protection to the toe-down depth. The design methodology for launchable riprap can be found in Section 6.6.4 Toe Protection.

FIGURE 11.42
LATERAL EROSION ZONE FOR SAND AND GRAVEL MINING EXCEPTION



11.10 SUMMARY AND CONCLUSIONS

This chapter provides the basic concepts of sedimentation engineering and some analytical methods and design procedures for sediment yield and scour estimation. Sedimentation problems as well as solutions to these problems are briefly discussed. Fundamentals for alluvial channels and sediment transport are also discussed. Detailed procedures for estimating sediment yield, scour, and lateral erosion hazard zones are given in this chapter.

One of the most important philosophies for sedimentation engineering analysis is to make sure that the post-project conditions (scour, deposition, and sediment transport) should not cause any adverse effects as compared with the pre-project conditions. In Maricopa County, one of the most important sedimentation engineering issues is scour estimation, which is critical to the stability of bridges, channel bank protection, pipeline crossings, and other structures. Due to potential flash flood conditions and a lack of data in the desert environment, the estimation for scour and other sedimentation engineering parameters is a challenging task. A higher safety factor may be used to deal with the uncertainties in those sedimentation engineering parameters.

11.11 REFERENCES

11.11.1 Cited In Text

Alonso, M.A., 1997, Controls on Erosion, Subbasins of Walnut Gulch, Arizona: in Management of Landscapes Disturbed by Channel Incision, Edited by Want and others, University of Mississippi, Oxford, MS.

American Society of Civil Engineers (ASCE), 1975, Sedimentation Engineering, Manual 54, Sedimentation Committee of the Hydraulics Division, Edited by Vito Vanoni, New York, NY.

--, 2006, Sedimentation Engineering, Manual 54, Sedimentation Committee of the Hydraulics Division, Edited by Vito Vanoni, New York, NY.

--, 2008, Sedimentation Engineering, Processes, Measurements, Modeling, and Practice, Manual 110, Sedimentation Committee of the Hydraulics Division, Edited by Marcelo Garcia, New York, NY.

ASCE Task Committee on Hydraulics, Bank Mechanics, and Modeling of River Width Adjustment, 1998a, River Width Adjustment. I: Processes and Mechanisms, Journal of Hydraulic Engineering, 124(9), 881-902.

-- ASCE Task Committee on Hydraulics, Bank Mechanics, and Modeling of River Width Adjustment, 1998b, River Width Adjustment, II: Modeling, Journal of Hydraulic Engineering 124(9), 903-917.

Arizona Department of Water Resources (ADWR), 1996, State Standard for Watercourse System Sediment Balance, Flood Warning and Dam Safety Section.

Ayres Associates, 2004, Erosion Setback and Stabilization Criteria for City of Austin Streams, Prepared for City of Austin, Watershed Protection and Development Review Department, Watershed Engineering Division.

Bell, R.G. and Sutherland, A.J. 1983, Nonequilibrium Bedload Transport by Steady Flow, Journal of Hydraulic Engineering, ASCE, 109(3), 351-367.

Bennett, S.J., Alonso, C.V., Prasad, S.N. and Romkens, M.J. M. 1997. Dynamics of Headcut in Upland Concentrated Flows, Proceedings of the Conference on Management of Landscapes Disturbed by Channel Incision, Oxford, Mississippi, pp. 510-515.

Bettess, R. and White, W.R., 1987, Extremal Hypotheses Applied to River Regime. Sediment Transport in Gravel-bed Rivers, C.R. Thorne, J.C. Bathurst, and R.D. Hey. eds., John Wiley & Sons, Inc., Chichester, U.K., 767-791.

Blench, T., 1951, Regime Theory for Self-Formed Sediment-Bearing Channels, Proceeding of the American Society of Civil Engineers, Vol. 77.

--, 1957, Regime Behaviour of Canals and Rivers, Butterworth, London, England, pp. 138.

--, 1964, Practical Regime Analysis of River Slopes, Journal of Hydraulics Division, ASCE, 90(HY2): pp.81-98.

--, 1969, Mobile-Bed Fluviology, the University of Alberta Press, Edmonton, Canada.

--, 1986, Mechanics of Plains Rivers: A Regime Theory Treatment of Canals and Rivers for Engineers and Environmentalists, Printing Services University of Alberta, Edmonton, Alberta, Canada.

Borah, D.K. and Bordoloi, P.K., 1989, Stream Bank Erosion and Bed Evolution Model, Sediment Transport Modeling, S. Wang, ed., ASCE, New York, N.Y., 612-619.

Bray, D.I., 1982, Regime Equations for Gravel-bed Rivers, in Gravel-bed Rivers: Fluvial Processes, Engineering and Management, Hey, R.D., Bathurst, J.C., and Thorne, C.R., editors, John Wiley & Sons, Inc., Chichester, U.K., 517-552.

Chang Consultants, 2006, Generalized Computer Program FLUVIAL-12 Mathematical Model for Erodible Channels, Users Manual.

Chang, H.H., 1980, Geometry of Gravel Streams, Journal of Hydraulic Engineering, ASCE, 106(9), 1443-1456.

--, 1988a, Introduction to FLUVIAL-12 Mathematical Model for Erodible Channels, Twelve selected computer stream sedimentation models developed in the United States, S. Fan, ed., Federal Energy Regulatory Commission, Washington, D.C.

--, 1988b, Fluvial Processes in River Engineering, Wiley, New York, NY.

--, 2006, Hydraulic and Sediment Study for Riprarian Restoration in El Monte Valley, prepared for El Capitan Golf Club.

Chen, Y.H. 1980. Investigation of Gravel Mining Effects, Salt River Channelization Project at Sky Harbor International Airport. Prepared for Anderson-Nichols, Palo Alto, California, by Colorado State University, Ft. Collins, Colorado.

Chien, N. 1955, A Concept of Lacey's Regime Theory, Proceedings ASCE, Vol. 81.

Chien, N. and Wan, Z. 1998. Mechanics of Sediment Transport, ASCE.

City of Austin, September 10, 2007. Guidance on Establishing an Erosion Hazard Zone for Structure and Utility Locations near Streams, Watershed Protection and Development Review Department, Texas.

Cruff, R.W., 1999, Channel Subdivision and Bank Selection in Open Channels; Phoenix, Arizona. Flood Control District of Maricopa County Engineering Library, pp. 17.

Darby, S.E. and Thorne, C.R., 1996, Numerical Simulation of Widening and Bed Deformation of Straight Sand-bed Rivers. I: Model Development, Journal of Hydraulic Engineering, ASCE, 122(4), 184-193.

De Ploey, J. 1989. A Model for Headcut Retreat in Rills and Gullies, CATENA Supplement 14:81-86, Cremlingen, West Germany.

Downer, C.W. and Ogden, F.L., 2002, GSSHA User's Manual, US Army Engineering Research and Development Center and University of Connecticut. http://chl.erdc.usace.army.mil/gssha.

Duan, J.G., 2005, Analytical Approach to Calculate Rate of Bank Erosion, Journal of Hydraulic Engineering, ASCE, 131(11), 980-990.

Duan, J.G. and Julien, P.Y., 2005, Numerical simulation of the inception of channel meandering, Earth Surface Processes and Landforms, 30, 1093-1110.

Fisk, G.G., Duet, N.R., McGuire, E.H., Roberts, W.P., Castillo, N.K., and Smith, C.F., 2005, Water Resources Data Arizona Water Year 2005, Water-Data Report AZ-05-1, U.S. Geological Survey, U.S. Department of the Interior.

Flaxman, E. M., 1972, Predicting Sediment Yield in Western United States, Proceeds American Society of Civil Engineers, Journal of the Hydraulics Division, HY 12, p. 2073-2085.

-- 1974, Progress Report on Development of Sediment Yield Predictive Equations, Soil Conservation Service, TSC Advisory ENG-PO-32, Portland, Oregon.

Flood Control District of Maricopa County (FCDMC), 1996, Hydraulics Manual.

--, 2006, Lower Hassayampa Watercourse Master Plan, River Behavior Report, prepared by JE Fuller/Hydrology and Geomorphology, Inc for FCDMC.

--, 2010, Engineering Division, River Mechanics Manual for DDMSW.

Frenette, R. and I. Pestov. 2005. Flow and Erosive Stresses at the Base of a Headcut, Journal of Hydraulic Engineering, ASCE, 131(2), American Society of Civil Engineers.

Garde, R.J. and Raju, K.R. 1985, Mechanics of Sediment Transportation and Alluvial Stream Problems, 2nd edition, Halsted Press, a Division of John Wiley & Sons, New York, New York.

Glymph, L. M., 1951, Relation of Sedimentation to Accelerated Erosion in the Missouri River Basin, Technical Paper 102, Soil Conservation Service, USDA.

Graf, W.H., 1984, Hydraulics of Sediment Transport, Water Resources Publications, Littleton, Colorado.

Guy, H. P., 1969, Laboratory Theory and Methods for Sediment Analyses, Techniques of Water-Resources Investigations, U.S. Geological Survey, U.S. Department of the Interior, Washington, D.C.

--, 1989, Fluvial Sediment Concepts, Techniques of Water-Resources Investigations of the U.S. Geological Survey, U.S. Department of the Interior, Washington, D.C.

Guy, H. P. and Norman, V. W., 1970, Field Methods for Measurement of Fluvial Sediment, Techniques of Water-Resources Investigations, U.S. Geological Survey, U.S. Department of the Interior, Washington, D.C.

Gyr, A., and Hoyer, K., 2006. Sediment Transport a Geophysical Phenomenon, Springer, The Netherlands.

Han, Q. 1980. A Study on the Non-Equilibrium Transport of Suspended Load, Proceedings of the International Symposium on River Sedimentation, Beijing, China, pp. 793-802 (in Chinese).

Hanson, G.J., Robinson, K.M., and Cook, K.R. 1997. Headcut Migration Analysis of a Compacted Soil, Transactions of the American Society of Agricultural Engineers, 40(2): 355-361.

--, 1999. Prediction of Headcut Migration Using a Deterministic Approach, An ASAE Meeting Presentation at the 1999 ASAE/CSAE-SCGR Annual International Meeting, Toronto, Ontario, Canada.

--, 2001. Prediction of Headcut Migration Using a Deterministic Approach, Transactions of the American Society of Agricultural Engineers.

Hasegawa, K., 1989, Universal Bank Erosion Coefficient for Meandering Rivers, Journal of Hydraulic Engineering, ASCE, 115(6), 744-765.

Henderson, F. M., 1966, Open Channel Flow, MacMillan, New York, NY.

Hey, R.D. and Thorne, C.R., 1986. Stable Channels with Mobile Gravel Beds, Journal of Hydraulic Engineering, ASCE, 112(6), 671-689.

Hooke, R.L., 1994. On the efficacy of humans as geomorphic agents. GSA Today, 4(9), 217, 224-225.

Hooke, J. and Mant, J. 2002, Morpho-dynamics of Ephemeral Streams in Dryland Rivers: Hydrology and Geomorphology of Semi-Arid Channels, Edited by Bull, L.J and Kirkby, M.J., John Wiley and Sons Ltd., England.

Ikeda, S., Parker, G., and Sawai, K., 1981, Bend Theory of River Meanders, Part 1: Linear Development, Journal of Fluid Mechanics, 112, 363-377, Cambridge University Press, Cambridge, United Kingdom.

Interagency Committee on Water Resources, Subcommittee on Sedimentation. 1957. Measurement and Analysis of Sediment Loads in Streams: Report No. 12, Some Fundamentals of Particle Size Analysis, St. Anthony Falls Hydraulic Laboratory, Minneapolis, MN.

Jia, Y. and Wang, S.S.Y., 1999, Numerical Model for Channel Flow and Morphological Change Studies, Journal of Hydraulic Engineering, ASCE, 125(9), 924-933.

Jia, Y.F. and Wang, S., 2001, CCHE2D: Two-dimensional Hydrodynamic and Sediment Transport Model For Unsteady Open Channel Flows Over Loose Bed, National Center for Computational Hydroscience and Engineering, the University of Mississippi. http://www.ncche.olemiss.edu/index.php?page=freesoftware.

Jia, Y. and Wu, W., 2007, Numerical and Empirical Modeling Tools Development for Studying Bank Erosion and Evolution of Headcut and Tailcut due to Sand and Gravel Mining Pits in Alluvial Rivers, Technical Report No. NC CHE-TR-2007-2, National Center for Computational Hydroscience and Engineering, The University of Mississippi.

Julien, P.Y., 2002, River Mechanics, Cambridge University Press, Cambridge, United Kingdom.

Julien, P.Y. and Wargadalam, J., 1995, Alluvial Channel Geometry: Theory and Applications, Journal of Hydraulic Engineering, ASCE, 121 (4), 312-325.

Kennedy, R.G., 1895, The Prevention of Silting in Irrigation Canals, Minutes of the Proceedings Institute of Civil Engineers, London, England, 119, 281-290.

Kennedy, J.F., 1961, Stationary Waves and Antidunes in Alluvial Channels, Caltec Report, No. KH-R-2.

Kondolf, G.M., 1994, Geomorphic and Environmental Effects of Instream Gravel Mining, Landscape and Urban Planning, Elsevier Science B.V., Amsterdam, 28, 225-243.

--, 1997, Hungry Water: Effects of Dams and Gravel Mining on River Channels, Environmental Management, 21(4), 533-551.

Lacey, G., 1929, Stable Channels in Alluvium, Minutes of Proceedings Institute of Civil Engineers, London, England, 229, 259-292.

--, 1937, Stable Channels in Erodible Material: A Discussion, Transactions of the American Society of Civil Engineers, Vol. 102.

Lane, E. W., 1947, Report on the Subcommittee on Sediment Terminology, Transactions of the American Geophysical Union, Washington, D.C., Vol. 28 (6), 936-938,

--, 1955, Design of Stable Channels, Transactions of the American Society of Civil Engineers, 120, 1234-1260.

Lane, L.J., Nichols, M. H., and Paige, G.B., 1995, Modeling Erosion on Hillslopes: Concepts, Theory, and Data, In: Proceedings of the International Congress on Modeling and Simulation (MODSIM'95) (ed. by P. Binning, H. Bridgman, and B. Williams), November 27-30, 1995, University of Newcastle, Newcastle, NSW, Australia, Uniprint, Perth, Australia, pp. 1-7. http://eisnr.tucson.ars.ag.gov/hillslopeerosionmodel/.

Lane, L.J., Kidwell, R., and Weltz, A., 2000, Watershed Sediment Yield and Rangeland Health, International Journal of Sediment Research, 15(1), 51-59.

Langbein, W. B., Hains, C. H. and Culler, R. C., 1951, Hydrology of Stock-water Reservoirs in Arizona, U.S. Geological Survey Circular 110, 18.

Langbein, W. B. and Schumm, S.A., 1958, Yield of Sediment in Relation to Mean Annual Precipitation, Transactions of the American Geophysical Union, 39, 1076-1084, Washington, D.C.

Lee, H.Y., Fu, D.T. and Song, M.H., 1993, Migration of Rectangular Mining Pit Composed of Uniform Sediment, Journal of Hydraulic Engineering, ASCE, 119(1), 64-80.

Leopold, L.B. and Maddock, T. M., 1953, The Hydraulic Geometry of Stream Channels and Some Physiographic Implications, USGS Professional Paper 252, Washington, D.C.

Leopold, L.B. and Miller, J. 1956. Ephemeral Streams-Hydraulic Factors and Their Relation to the Drainage Net, USGS Professional Paper 282-A, Washington, D.C.

Leopold, L.B. and Wolman, M.G., 1957, River Channel Patterns: Braided, Meandering and Straight, USGS Professional Paper 282-B, Washington, D.C.

Leopold, L.B. and Wolman, M.G., 1960, River Meanders, Geologic Society of America Bulletin, Vol. 71.

Leopold, L.B., Emmett, W. W. and Myrick, R. M., 1966, Channel and Hillslope Processes in a Semiarid Area of New Mexico, USGS Professional Paper 352-G, Washington, D.C., 193-253,

Leopold, L.B., Wolman, M.G., and Miller, J.P., 1964, Fluvial Processes in Geomorphology, W.H. Freeman and Company, San Francisco, CA.

Li, L., and Wang, S.S.Y., 1993, Numerical Modeling of Alluvial Stream Bank Erosion, Advances in Hydro-Science and Engineering, S.S.Y. Wang, ed., University of Mississippi, Oxford, Mississippi, Vol. 1, 2085-2090.

Li, R. M., Cotton, G. K., Zeller, M. E., Simons, D. B., and DesChamps, P. Q., 1989, Effects of Instream Mining on Channel Stability, prepared for Arizona Department of Transportation, FHWA-AZ89-250, Phoenix, AZ.

Lindley, E.S., 1919, Regime Channels, Proceedings, Punjab Engineering Congress, Vol. VII.

Lopez S., J.L., 2004, Channel Response to Gravel Mining Activities in Mountain Rivers, Journal of Mountain Science, 1(3), pp. 264-269.

MBH Software, Inc., 2002, Sedimentation in Stream Networks User's Manual (HEC-6T), Clinton, Mississippi.

Mahmood, K., and Shen, H. W., 1971, Regime Concept of Sediment-Transporting Canals and Rivers, River Mechanics, Vol. 2, H. W. Shen, ed., Fort Collins, CO.

Meade, R.H. and Parker, R.S., 1984, Sediment in Rivers of the United States, U.S. Geological Survey Water Supply Paper 2250, 49-60.

Mays, Larry W. 1999, Hydraulic and Design Handbook, McGraw-Hill.

Meyer-Peter, E. and Muller, R, 1948, Formulas for Bed-load Transport, International Association for Hydraulic Structures, Second Meeting, Stockholm, Sweden.

Millar, R.G. and Quick, M.C., 1997, Discussion of Development and Testing of a Riverbank-stability Analysis, by S.E. Darby and C.R. Thorne, Journal of Hydrologic Engineering, ASCE, 123(11), 1051.

Mussetter, R. A., Lagasse, P.F., and Harvey, M.D., 1994. Erosion and Sediment Design Guide, prepared for Albuquerque Metropolitan Arroyo and Flood Control Authority.

Nakagawa, H. Tsujimoto, T. and Murakami, S. 1989. Convolution-integral Modeling of Non-Equilibrium Sediment Transport, Proceedings of the International Symposium on Sediment Transport Modeling, edited by Wang, S.Y., ASCE.

Nanson, G.C., Tooth, S., and Knighton, D. 2002, A Global Perspective on Dryland Rivers: Perceptions, Misconceptions and Distinctions, in Dryland Rivers: Hydrology and Geomorphology of Semi-Arid Channels, Edited by Bull, L.J and Kirkby, M.J., John Wiley and Sons Ltd., England.

Natural Resources Conservation Services (NRCS). 1997. Earth Spillway Erosion Model, Chapter 51 from National Engineering Handbook (part 628 Dams).

Neill, C. R., 1973, Guide to Bridge Hydraulics, Roads and Transportation Association of Canada.

O'Brien, J. S., 1986, Physical Processes, Rheology and Modeling of Mud Flows, Ph.D. Dissertation, Colorado State University.

Odgaard A., 1989, River Meander Model, I: Development. Journal of Hydraulic Engineering, ASCE, 115(11): 1433-1450.

Olsen N.R.B. 2003. Three-dimensional CFD modelling of self-forming meandering channel. Journal of Hydraulic Engineering, ASCE, 129(5): 366-372.

Osman, A.M., 1985, Channel Width Response to Changes in Flow Hydraulics and Sediment Load, PhD thesis, Colorado State University, Fort Collins, Colorado

Osman, A. M. and Thorne, C. R., 1988a, Riverbank stability analysis, I: Theory, Journal of Hydraulic Engineering, ASCE, 114(2).

--, 1988b, Riverbank stability analysis, II: Application, Journal of Hydraulic Engineering, ASCE, 114(2).

Pacific Southwest Inter-Agency Committee, 1974, Erosion and Sediment Yield Methods, Pacific Southwest Inter-Agency Committee, Report of the Water Management Subcommittee, p. 45.

Parker, G., Sawai, K., and Ikeda, S., 1982, Bend Theory of River Meanders. Part 2. Nonlinear Deformation of Finite-amplitude Bends, Journal of Fluid Mechanics, 115, 303-314, Great Britain.

Pemberton, E. L. and Lara, J. M., 1984, Computing Degradation and Local Scour, Technical Guideline, U.S. Bureau of Reclamation, Engineering and Research Center, Denver, CO. January 1984.

Peterson, H. V., 1962, Hydrology of Small Watersheds in Western States, U.S. Geological Survey Water-Supply Paper 1475-I, pp. 217-356.

Porterfield, G., 1972, Computation of Fluvial-Sediment Discharge, Techniques of Water-Resources Investigations, U.S. Geological Survey, U.S. Department of the Interior, Washington, D.C.

Renard, K. G. and Stone, J. J., 1981, Sediment Yield From Small Semiarid Rangeland Watersheds, Proceedings of the Workshop on Estimating Erosion and Sediment Yield on Rangelands, Tucson, Arizona, U.S. Department of Agriculture, pp. 129-144.

Richards, K. 1982, Rivers: Form and Process in Alluvial Channels, Methuen, London.

Robinson, K.M., and G.J. Hanson, 1994. A Deterministic Headcut Advance Model, Transactions of the American Society of Agricultural Engineers, 37(5): 1437-1443.

--, 1995. Large-Scale Headcut Erosion Testing, Transactions of the American Society of Agricultural Engineers, 38(2), 429-434.

Robinson, K.M., Bennett, S.J., Casali, J., and GJ. Hanson. 2000. Processes of Headcut Growth and Migration In Rills and Gullies, International Journal of Sediment Research, 15(1), 69-82.

Rouse, H. 1938. Fluid Mechanics for Hydraulic Engineers, Chap. XI, Dover, New York.

Schoklitsch, 1932, Prevention of Scour and Energy Dissipation, translated from German by E.F. Wilsey, U.S. Bureau of Reclamation.

Schumm, S. A., 1971, Fluvial Geomorphology: Historical Perspective, River Mechanics, Vol. 1, H. W. Shen, ed., Fort Collins, CO., pp. 4-30.

--, 1977, The Fluvial System, Wiley, New York.

Scott, K.M., 1973, Scour and Fill in Tujunga Wash - A Fanhead Valley in Urban Southern California - 1969, USGS Professional Paper 732-B, Washington, D.C.

Semmens, D.J., S.N. Miller, M. Hernandez, P. Miller, D. Goodrich, and W. Kepner, 2001. Automated Geospatial Watershed Assessment (AGWA) - A GIS Based Hydrologic Modeling

Tool: Documentation and User Manual. U.S. Department of Agriculture, Agriculture Research Service. http://www.tucson.ars.ag.gov/agwa.

Shen, H. W., ed., 1971, River Mechanics, Volumes I & II, published by H. W. Shen, Ft. Collins, CO.

--, 1972, Sedimentation Symposium to Honor Professor H. A. Einstein, published by H.W. Shen, Ft. Collins, CO.

--, 1973, Environmental Impact on Rivers, published by H. W. Shen, Ft. Collins, CO.

Shen, H.W. and Julien, P. (1993). Chapter 12 Erosion and Sediment Transport, In: Handbook of Hydrology, edited by Maidment, D.R., McGraw-Hill, Inc., New York, NY.

Sheppard, J. R., 1960, Investigation of Meyer-Peter, Muller Bed Load Formulas, U.S. Bureau of Reclamation, Denver, CO.

Shulits, S., 1935, The Schoklitsch Bed Load Formula, Engineering, London, England.

Simon, A., Curini, A., Darby, S.E., and Langendoen, E.J., 2000, Bank and Near-bank Processes in an Incised Channel, Geomorphology, Volume 35, 193-217, Elsevier, Amsterdam.

Simons, D.B. and Albertson, M.L., 1963, Uniform Water Conveyance Channels in Alluvial Material, Transactions of the American Society of Civil Engineers, Vol. 128, Part I.

Simons, D.B. and Senturk, F., 1992, Sediment Transport Technology, Water Resources Publications, Fort Collins, Colorado.

Simons, D.B., Richardson, E.V., and Haushild, W.L., 1963, Studies of flow in alluvial channels. Some effects of fine sediment on flow phenomena. USGS Water Supply Paper No. 1498-G.

Simons, Li & Associates, Inc., 1982, Engineering Analysis of Fluvial Systems, Fort Collins, Colorado.

--, 1985, Design Manual for Engineering Analysis of Fluvial Systems, prepared for Arizona Department of Water Resources, Phoenix, Arizona.

--, 1986, Hydraulic Model Study of Local Scour Downstream of Rigid Grade-control Structures, prepared for Pima County Department of Transportation and Flood Control District, Tucson, Arizona.

Slatyer, R.O. and Mabbutt, J.A. 1964, Hydrology of Arid and Semiarid Regions, In: Handbook of applied hydrology, edited by Chow, V.T., McGraw-Hill, New York.

Smith, R.E., D.C. Goodrich, D.A. Woolhiser, and C.L. Unkrich, 1995. KINEROS - A kinematic runoff and erosion model; Chapter 20 in V.P. Singh (editor), Computer Models of Watershed Hydrology, Water Resources Publications, Highlands Ranch, Colorado, 1130 pp. http://www.tucson.ars.ag.gov/kineros/.

Soil Conservation Service (SCS), 1977, Design of Open Channels, Technical Release No. 25, U.S. Department of Agriculture, Washington, D.C.

Soil and Water Conservation Society, 1995, RUSLE User Guide, Revised Universal Soil Loss Equation Version 1.04.

Stein, O.R., and P.Y. Julien. 1993. Criterion Delineating the Mode of Headcut Migration, Journal of Hydraulic Engineering, ASCE, 119(3), 37-50.

Stein, O.R. and D.A. LaTray. 2002. Experiments and Modeling of Head Cut Migration in Stratified Soils, American Geophysical Union, Water Resources Research, 28(12).

Temple, D.M., 1992. Estimating Flood Damage to Vegetated Deep Soil Spillways, Applied Engineering in Agriculture, 8(2): 237-242.

Temple, D.M. and J.S. Moore, 1994. Headcut Advance Prediction for Earth Spillways, American Society of Agricultural Engineers, Paper No. 94-2540, St. Joseph, Michigan.

--, 1997. Headcut Advance Prediction for Earth Spillways, Transactions of the American Society of Agricultural Engineers, 40(3): 557-562.

Thomas, W.A., Copeland, R.R., and McComas, D.N., 2002, SAM Hydraulics Design Package for Channels, Coastal and Hydraulics Laboratory, U.S. Army Engineer Research and Development Center, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199.

Tiffany, T.B. 1939, Studies of Meandering of Model Streams, Transactions of the American Geophysical Union, Vol. 20.

Toy, T. J. and Osterkamp, W. R., 1995, The Applicability of RUSLE to Geomorphic Studies, Journal of Soil and Water Conservation, V. 50, No. 5, pp. 498-503.

Urban Drainage and Flood Control District, 1987, Technical Review Guidelines for Gravel Mining Activities Within or Adjacent to 100-year Floodplains, Denver, Colorado.

U.S. Army Corps of Engineers (USACE), 1970, Engineering and Design Laboratory Soils Testing, Engineer Manual EM 1110-2-1906, pp. V-26.

--, 1974, New River and Phoenix City Streams, Arizona-Design Memo. No. 2, Hydrology, Part 1, Los Angeles, District, 51 p and plates.

--, 1980, Sand and Gravel Mining Guidelines, Los Angeles District.

--, 1981, Phoenix Flood Damage Survey, February 1980.

--, 1987, Skunk Creek, New and Agua Fria Rivers, Los Angeles, District

--, 1989, Sedimentation Investigations of Rivers and Reservoirs, Engineer Manual 1110-2-4000.

--, 1991, HEC-6, Scour and Deposition in Rivers and Reservoirs, User's Manual. [HEC WEB Site]

--, 1994, Channel Stability Assessment for Flood Control Projects, Engineering Manual 1110-2-1418, published by ASCE Press, New York, NY, pp.108.

--, 1995, Sedimentation Investigations of Rivers and Reservoirs, Engineer Manual 1110-2-4000, Revised.

U.S. Department of Agriculture (USDA), 1972, National Engineering Handbook, Washington D.C. Section 3, Chapter 6. Sedimentation.

--, 1997, Predicting Soil Erosion by Water: A Guide to Conservation Planning with the Revised Universal Soil Loss Equation (RUSLE), Agricultural Handbook Number 703.

U. S. Department of the Interior, Bureau of Reclamation (USBR), 1977, Design of Small Dams, U. S. Government Printing Office, Washington, D.C.

U.S. Department of Transportation (USDOT), 2001a, Bridge Scour and Stream Instability Countermeasures, Hydraulic Engineering Circular No. 23, FHWA NHI-01-003 (USDOT Hydraulics WEB Site).

--, 2001b, Evaluating Scour at Bridges: Fourth Edition, Hydraulic Engineering Circular No.18, Publication No. FHWA NHI 01-001.

--, 2001c, Stream Stability at Highway Structures: Third Edition, Hydraulic Engineering Circular No. 20, Publication No. FHWA NHI 01-002.

--, 2001d, River Engineering for Highway Encroachments: Highways in the River Environment, Hydraulic Design Series Number 6, Publication No. FHWA NHI 01-004.

--, 2006, Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14, Publication No. FHWA NHI 06-086.

U.S. Environmental Protection Agency (EPA), 1976, Erosion and Sediment Control, Surface Mining in the Eastern U.S., EPA-625/3-76-006.

Veronese, A., 1937, Erosioni Di Fondo A Valle Di Uno Searico.

Vieira, D.A. and Wu, W., 2002, One-Dimensional Channel Network Model CCHE1D, User's Manual, National Center for Computational Hydroscience and Engineering, the University of Mississippi. http://www.ncche.olemiss.edu/index.php?page=cche1d_documents.

White, W.R., Paris, E., and Bettess, R., 1981, Tables for the Design of Stable Alluvial Channels, Rep. No. IT 208, HR Wallingford, Wallingford, United Kingdom, 30.

Wischmeier, W.H. 1972, Estimating the Cover and Management Factor for Undisturbed Areas, Presented at the Sediment Yield Workshop, USDS Sedimentation Laboratory, Oxford, Mississippi.

Wischmeier, W.H. and D.D. Smith, 1978, Predicting Rainfall Erosion Losses. A Guide to Conservation Planning. Agriculture Handbook No. 537, USDA-SEA, US. Government Printing Office, Washington, D.C. pp. 58.

Wolman, M.G., 1954, A Method of Sampling Coarse River Bed Material, Transactions of the American Geophysical Union, Vol. 35.

Wu, W., 2007. Computational River Dynamics, Taylor & Francis Group, New York, New York.

Wu, W., and Vieira, D.A., 2002, One-Dimensional Channel Network Model CCHE1D, Technical Manual, National Center for Computational Hydroscience and Engineering, the University of Mississippi. http://www.ncche.olemiss.edu/index.php?page=cche1d_documents.

Yang, C.T., 1973, Incipient Motion and Sediment Transport, Journal of the Hydraulics Division, ASCE, Vol. 99, No. HY10.

--, 1992, Force, Energy, Entropy and Energy Dissipation Rate, Entropy and Energy Dissipation in Water Resources, V.P. Singh and M. J. Fiorention, eds., Kluwer Academic Publishers, The Netherlands, 63-89.

--, 2003, Sediment Transport: Theory and Practice, McGraw-Hill Companies, Inc.

Yang, C.T. and Song, C.C.S., 1986, Theory of Minimum Energy and Energy Dissipation Rate. Encyclopedia of Fluid Mechanics, Gulf Publishing Co., Houston, Texas, 353-399.

Yang, C.T., Molinas, A., and Song, C.C.S., 1988, GSTARS - Generalized Stream tube Model for Alluvial River Simulation, Twelve selected computer stream sedimentation models developed in the United States, S. Fan, ed., Federal Energy Regulatory Commission, Washington, D.C.

Yang, C.T., Song, C.C.S., and Woldenberh, M.J., 1981, Hydraulic Geometry and Minimum Rate of Energy Dissipation, American Geophysical Union, Water Resources Research, 17(4), 1014-1018.

Zanke, U., 1976, Uber den Einfluss von Kornmaterial, Stromung un Wassestanden auf die Kengrossen von Transportkorpern in offenen Gerinnen. Mitt. Franzius-Inst. TU Hannover 44.

Zeller, M. E., 1981, Scour depth formula for estimation of toe-protection against general scour, Pima County Department of Transportation and Flood Control District, Tucson, Arizona.

Zeller, M. E. and W. T. Fullerton, 1983. A Theoretically Derived Sediment Transport Equation for Sand-Bed Channels in Arid Regions, Proceedings of the D. B. Simons Symposium on Erosion and Sedimentation, R. M. Li and P. F. Lagasse, eds., Colorado State University and ASCE.

Zimmerman, F. and Maniak, U. 1967, Scour Studies, Zimmerman, F. and Maniak, U. 1967, Scour Studies, Journal of Hydraulic Engineering Division, ASCE.

11.11.2 References Relevant to Chapter

Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA), 1994, Sediment and Erosion Design Guide, prepared by Resource Consultants and Engineers, Inc., Albuquerque, NM.

Blench, T., 1969, Coordination in Mobile Bed Hydraulics, Journal of the Hydraulic Division, ASCE, Vol. 95, No. HY6, pp 1871-1898.

Chang, H. H., 1986, FLUVIAL12, Mathematical Model for Erodible Channels, User's Manual.

Federal Energy Regulatory Commission, 1988, Twelve Selected Computer Stream Sedimentation Models Developed in the United States.

Federal Highway Administration, 1978, Countermeasures for Hydraulic Problems at Bridges, Volume II, FHWA RD-78-163, U.S. Geological Survey, Water Resources Division.

Froehlich, D. C., 1988, Analysis of Onsite Measurements of Scour at Piers, Proceedings of the ASCE National Hydraulic Engineering Conference, Colorado Springs, Colorado.

--, 1989, Local Scour at Bridge Abutments, Proceedings of the 1989 National Conference on Hydraulic Engineering, New Orleans, LA, pp. 13-18, ASCE.

Guy, H. P., 1970, Fluvial Sediment Concepts, Techniques of Water-Resources Investigations, U.S. Geological Survey, U.S. Department of the Interior, Washington, D.C.

Hedman, E. R. and Osterkamp, W. R., 1982, Streamflow Characteristics Related to Channel Geometry of Streams in the Western United States, USGS Water Supply Paper 2193.

Hjalmarson, H. W., 1998, Piedmont Flood Hazard Assessment for Flood Plain Management for Maricopa County, Arizona.

Lagasse, P. F. and Schall, J. D., 1988, Delineation of a Flooding and Erosion buffer for a Southwestern Arroyo, Proceedings of the Conference on Arid West Floodplain Management Issues, Association of State Flood Plain Managers Arid West Committee, Las Vegas, NV.

Lane, E.W., 1952, Progress Report on Results of Studies on Design of Stable Channels, HYD-352, U.S. Bureau of Reclamation.

Lane, E. W., and Koelzer, V.A., 1953, Density of Sediments Deposited in Reservoirs: Report No 9 of a Study of Methods Used in Measurement and Analysis of Sediment Loads in Streams, U.S. Army Corps of Engineers, St. Paul District, St. Paul, MN.

Laursen, E. M., 1960, Scour at Bridge Crossings, Journal of Hydraulic Engineering, ASCE, Vol. 89, No. HY3.

--, 1963, An Analysis of Relief Bridges, Journal of Hydraulic Engineering, ASCE, Vol. 92, HY3.

Leopold, L. B. and Wolman, M. G., 1957, River Channel Patterns-Braided, Meandering, and Straight: USGS Professional Paper 282-B. Washington, D.C.

MacBroom, J. G., 1981, Applied Fluvial Geomorphology, University of Connecticut - Storrs, Institute of Water Resources Report No. 31.

Mavis, F. T. and Laus, L. M., 1948, A Reappraisal of the Beginning of Bed-movement-competent velocity, International Association for Hydraulic Research, Second Meeting, Stockholm, Sweden.

National Research Council, 1996, Alluvial Fan Flooding, National Academy Press, Washington, D.C., p. 172.

Pemberton, E. L. and Lara, J. M., 1971, A Procedure to Determine Sediment Deposition in a Settling Basin, U.S. Bureau of Reclamation, Sedimentation Section, Denver, CO.

Rosgen, D., 1996, Applied River Morphology, Wildland Hydrology, Pagosa Springs, CO.

Schumm, S. A., 1961, Effect of Sediment Characteristics on Erosion and Deposition in Ephemeral Stream Channels, USGS Professional Paper 352-C. Washington, D.C.

Shields, A., 1936, Anwendung der Aenlichkeitsmechanik und der Turbulenzforschung anf die Geschiebebewegung, Mitteil, PVWES, Berlin, No. 36.

Stevens, M. A., Simons, D. B., and Richardson, E. V., 1975, Nonequilibrium River Form, Journal of the Hydraulics Division, ASCE, Vol. 101, No. HY5.

Thorne, C. R., 1998, Stream Reconnaissance Handbook - Geomorphological Investigation and Analysis of River Channels, John Wiley & Sons, New York.

U.S. Army Corps of Engineers (USACE), 2001a, HEC-RAS River Analysis System, Hydraulic Reference Manual. [HEC WEB Site].

--, 2001b, HEC-RAS River Analysis System, User's Manual. [HEC WEB Site].

U.S. Department of Transportation (USDOT), 1989, Design of Riprap Revetment, Hydraulic Engineering Circular No. 11, FHWA-IP-89-016.