Drainage Design Manual

for Maricopa County, Arizona

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Adopting Agencies

The following agencies and municipalities adopt the methodologies incorporated into the *Drainage Design Manual for Maricopa County, Hydraulics*.

Adopted By:	**Date**
Flood Control District of Maricopa County	
City of Phoenix	
Maricopa County Department of Transportation	

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 released in January 2004.

Comments

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

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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 September 2003 (draft)

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.

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.

Draft 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.

September 10, 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.

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1 INTRODUCTION

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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 the 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.

1.3.7 Hydraulic Structures

The hydraulic structures that are described in Chapter 7 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 8 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 ground water 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 9, 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 10 provides an overview of sediment transport theory. There is a general discussion of scour and sedimentation. 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 principals:

- 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 by 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 polocies 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

United States Army Corps of Engineers Hydrologic Engineering Center, 1990, HEC-1 Flood Hydrograph Package.

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.

2 HYDROLOGY

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2 HYDROLOGY
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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* (District, 2003), 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 Hydtology 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 (District) 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. Table 2.1 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 juristiction.

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 District.

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 (District), 2003, *Uniform Drainage Policies and Standards for Maricopa County*, Arizona.

——, 2003, *Drainage Design Manual for Maricopa County - Hydrology*, 3rd Edition.

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

3 STREET DRAINAGE

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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, 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

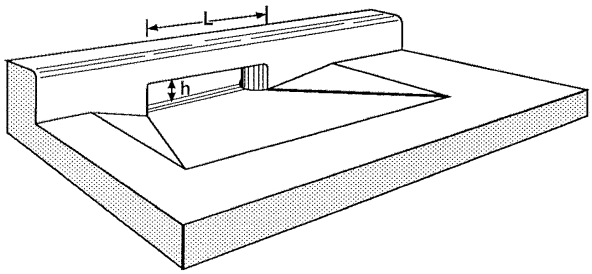
undepressed and are more efficient than curb-opening catch basins when located on a continuous grade. When grated catch basins are used, the engineer should design them to optimize hydraulic efficiency, bicycle and pedestrian safety, and structural adequacy. Grated catch basins shall not extend into 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.

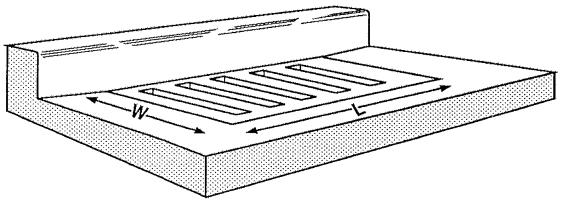
In a sump, combination inlets are very desirable. The curb-opening provides a relief if the grate should become clogged.

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.

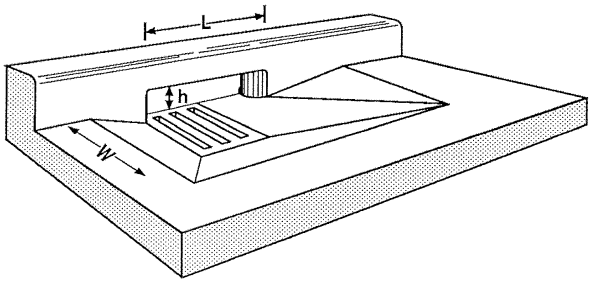
FIGURE 3.1
CATCH BASIN INLETS



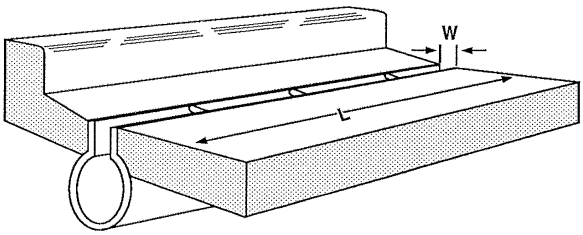
(a) Curb Opening Catch Basin Inlet



(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.

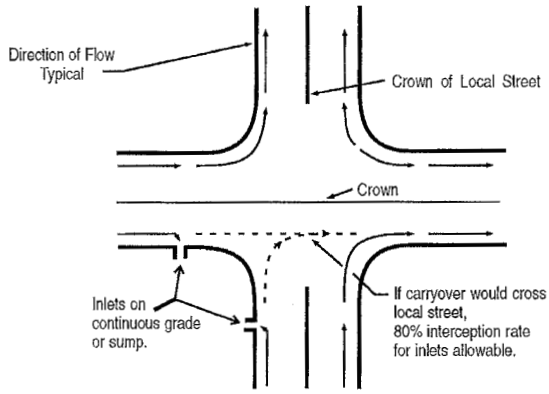
Storage Facilities

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.

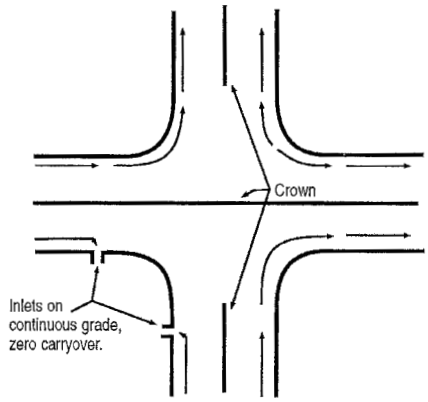
Roadside ditches are commonly used in rural areas to convey runoff from the highway pavement, and from areas which drain toward the highway. Where practicable, the flow from major areas draining toward curbed highway pavements should be intercepted by ditches.

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.

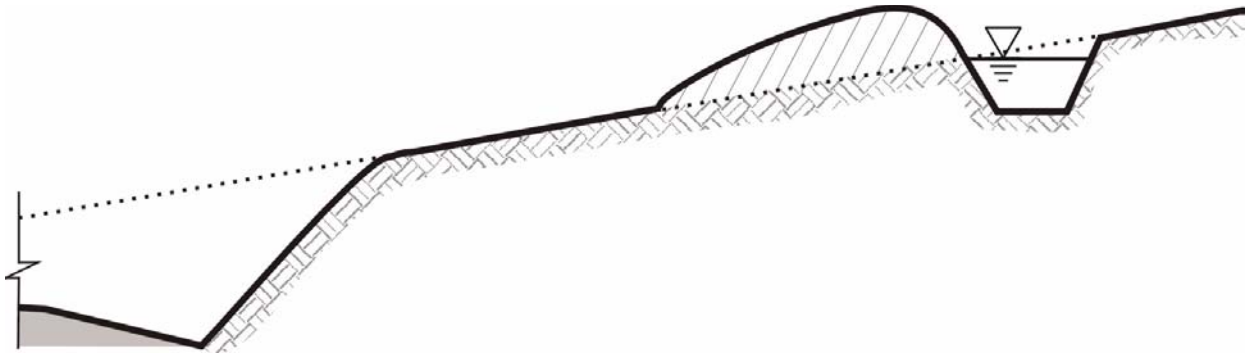
Channel side slopes should be as mild as practical and should be no steeper than 4:1 where terrain and right-of-way permit. The advantages of mild slopes are that the potential for erosion and slides is lessened, the cost of maintenance is reduced, and the safety of errant vehicles is enhanced. Safety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

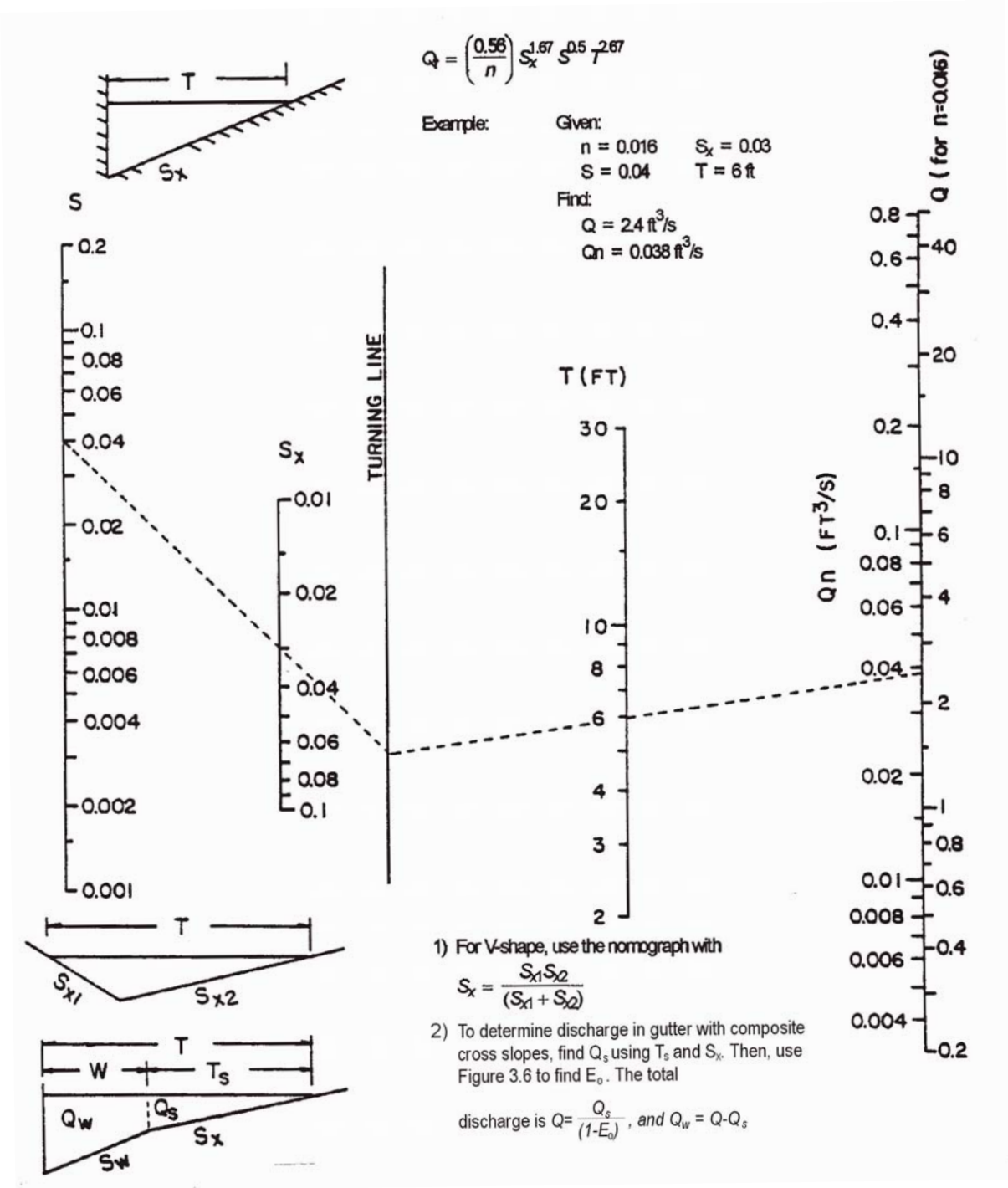
When the allowable pavement spread has been determined, the theoretical gutter carrying capacity shall be computed using the modified Manning's formula as expressed in Equation (3.2) or shown on Figure 3.4.

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

where:

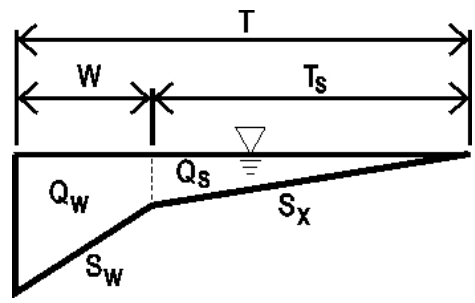
Q_t	=	Theoretical gutter carrying capacity, cfs
T	=	Spread of flow on pavement, ft
S_x	=	Pavement cross slope, ft/ft
S	=	Longitudinal slopes, ft/ft

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



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

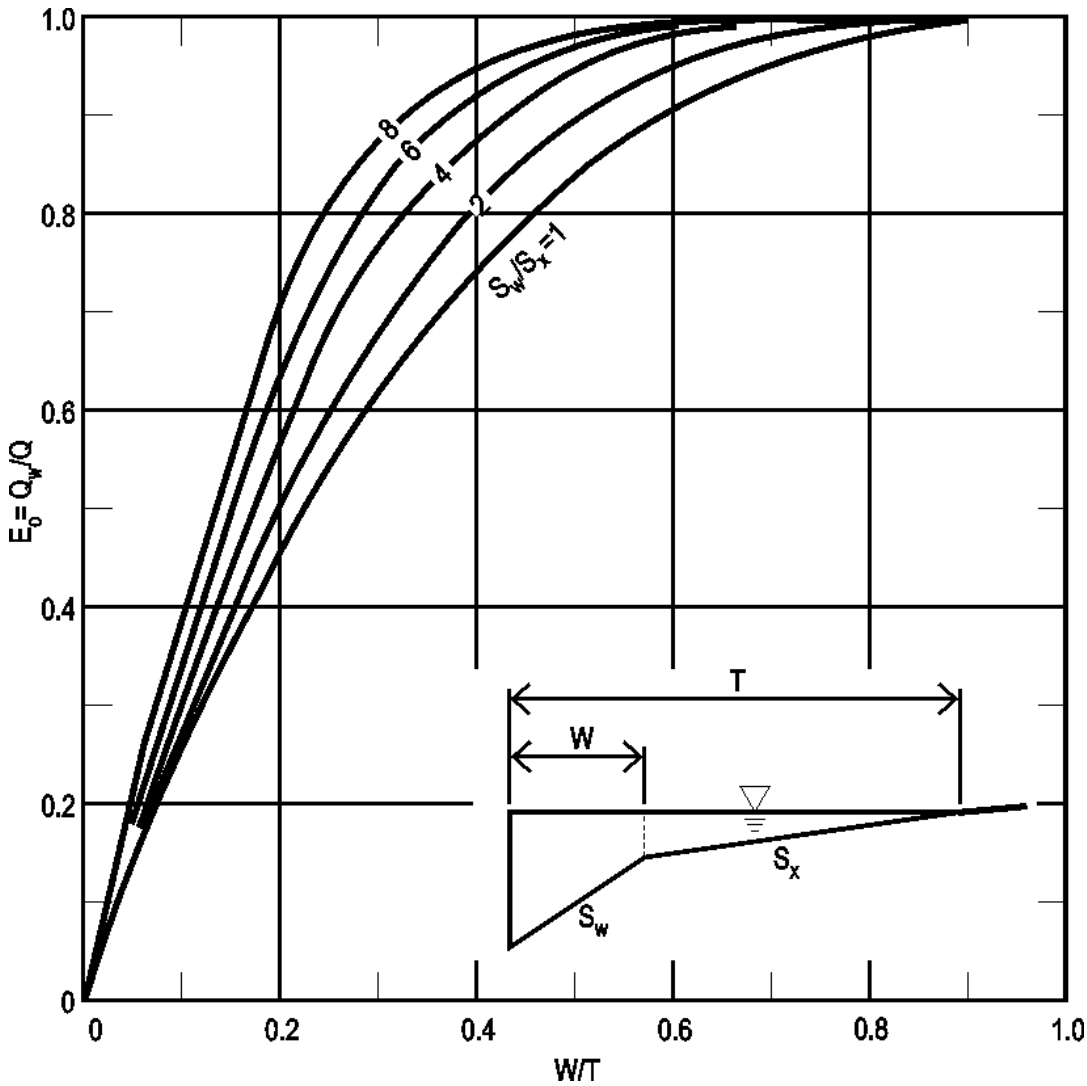
$$E_o = 1 \Big/ \left\{ 1 + \frac{S_w / S_x}{\left[1 + \dfrac{S_w / S_x}{\dfrac{T}{W} - 1} \right]^{2.67} - 1} \right\} \tag{3.4}$$

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 ($S_x + \frac{\text{gutter depression}}{W}$), ft/ft

(Eqn. 3.4 Ref: USDOT, FHWA, 1996, HEC-22, Eqn 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

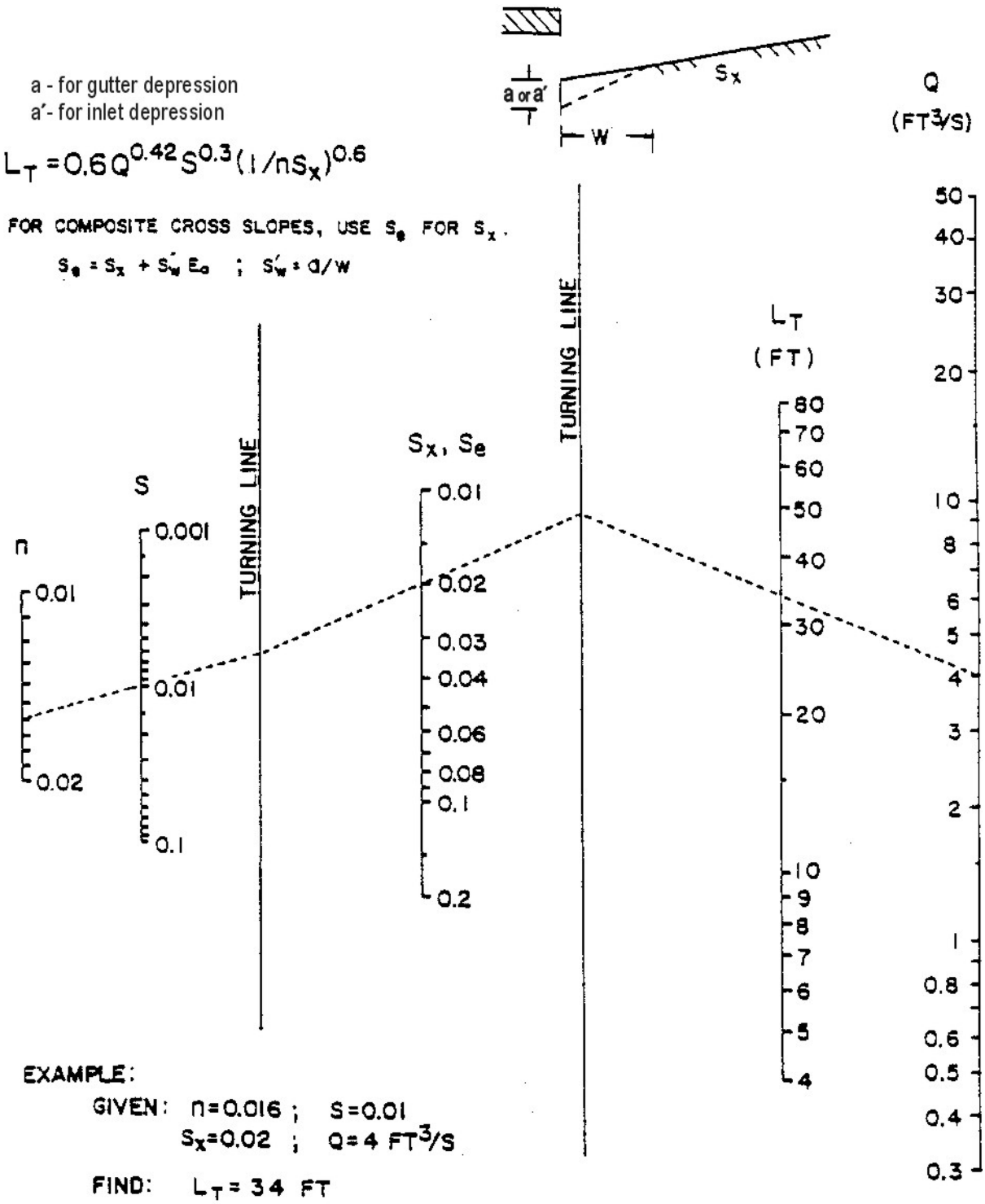
On-Grade - The length (L_t) of curb opening catch basin required for total interception of gutter flow on a pavement section with a straight cross slope is expressed as:

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

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:

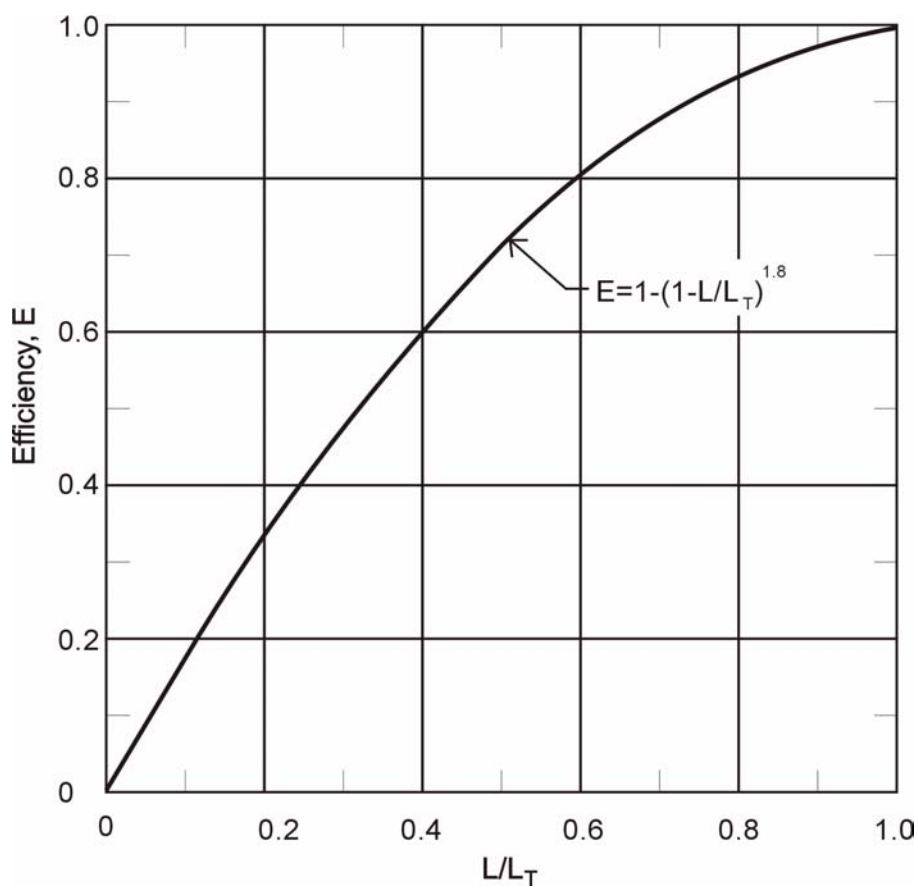
$$E = 1 - \left(1 - \frac{L}{L_t}\right)^{1.8} \tag{3.8}$$

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).

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

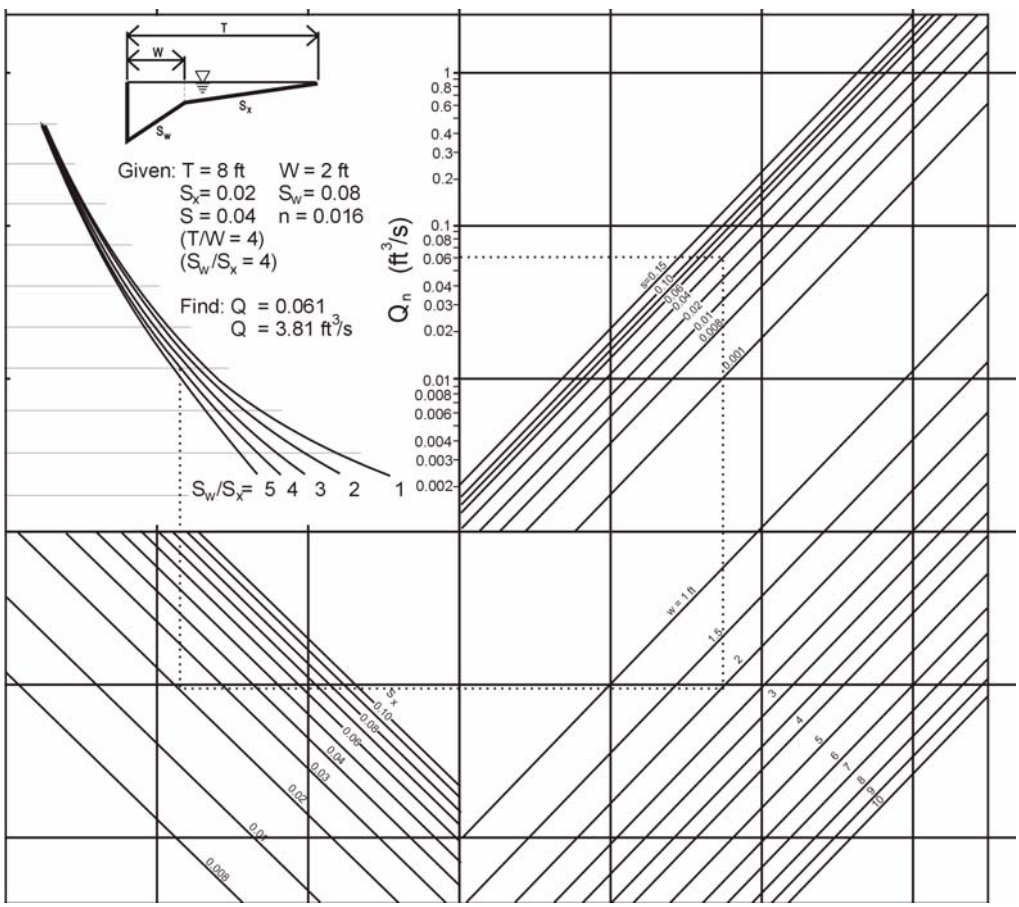
where: S'_w = Cross slope of the gutter (at the inlet) measured from the cross slope of the pavement, ft/ft (S'_w=a/12W; see Figure 3.7)

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:	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:

$$Q_i = C_w L d^{1.5} \tag{3.13}$$

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)

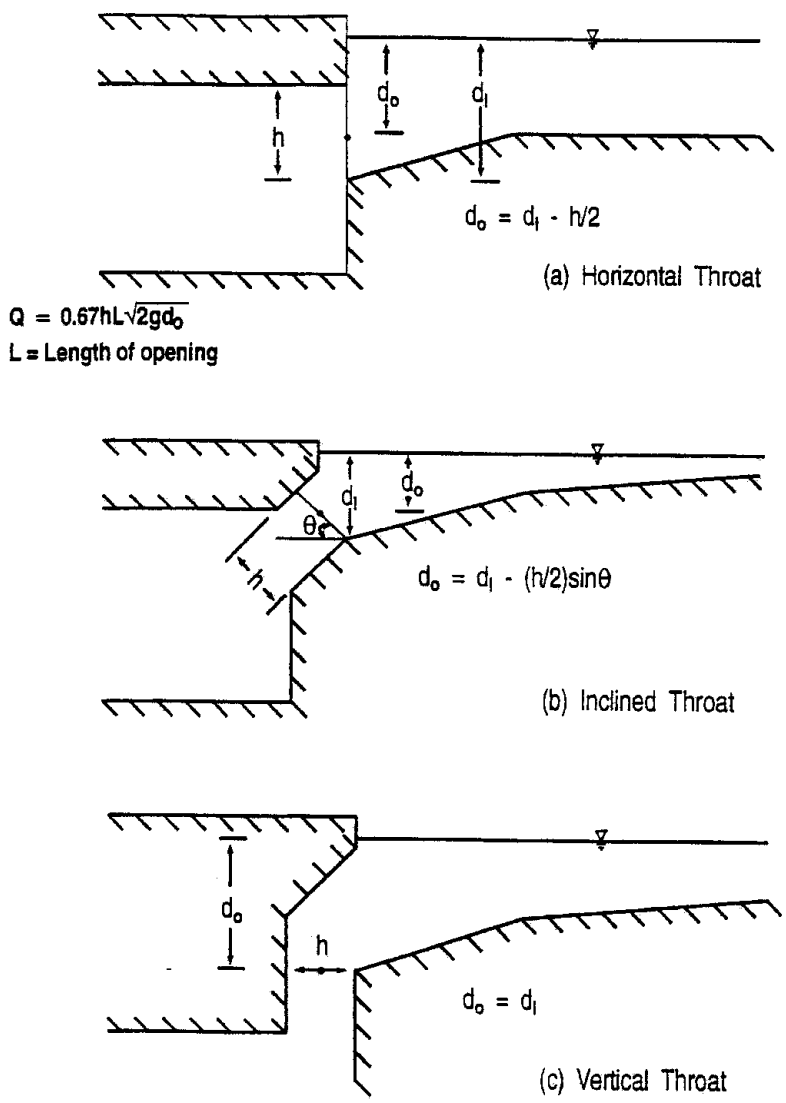


Figure 3.11 provides solutions for Equations 3.11 and 3.14 for depressed curb-opening catch basins, and Figure 3.12 provides solutions for Equations 3.13 and 3.14 for curb-opening catch basins without depression. Figure 3.13 is provided for use for curb openings with inclined or vertical orifice throats.

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

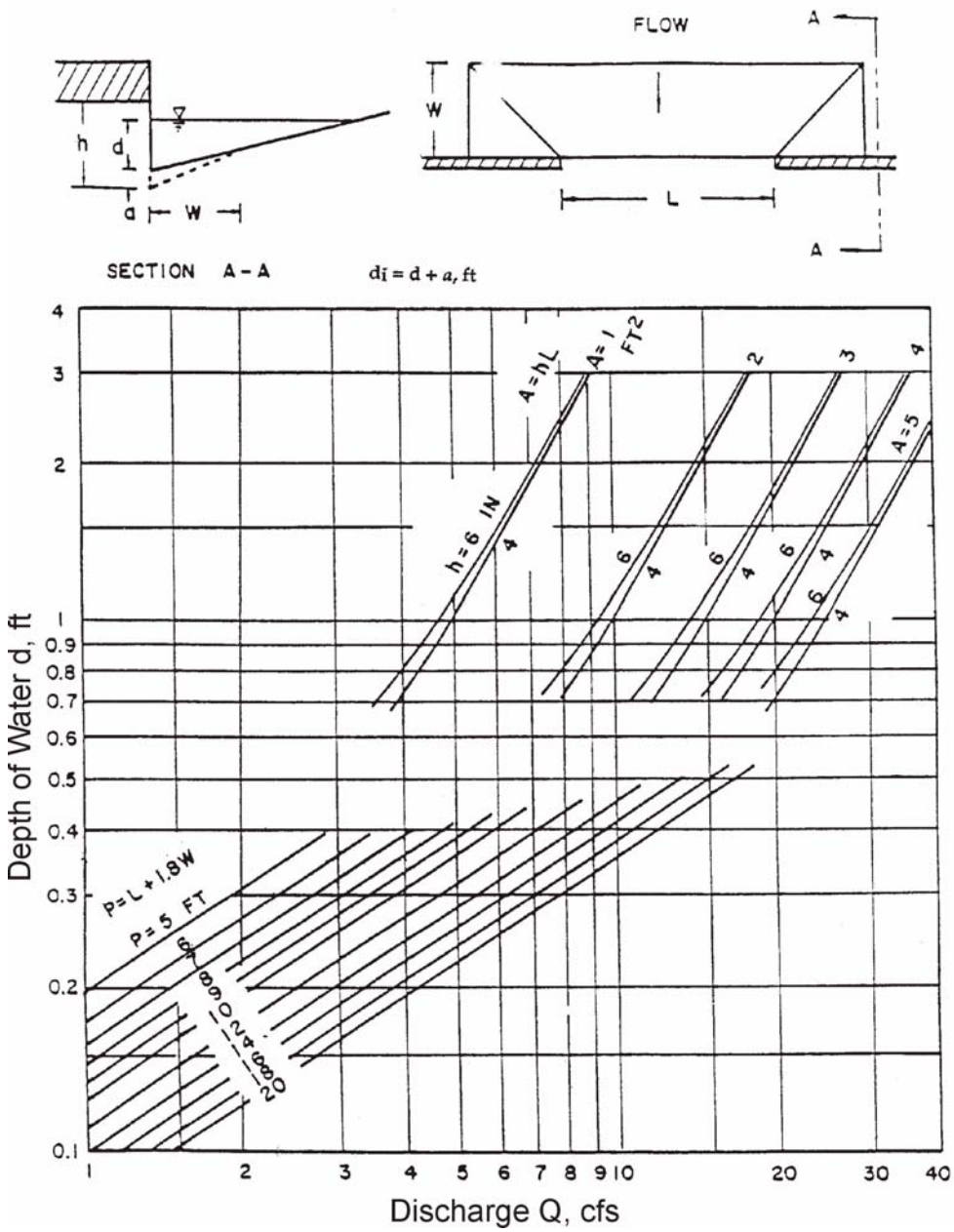


FIGURE 3.12
CURB OPENING INLET CAPACITY IN SUMP LOCATIONS
(USDOT, FHWA, 1984, HEC-12, Chart 13)

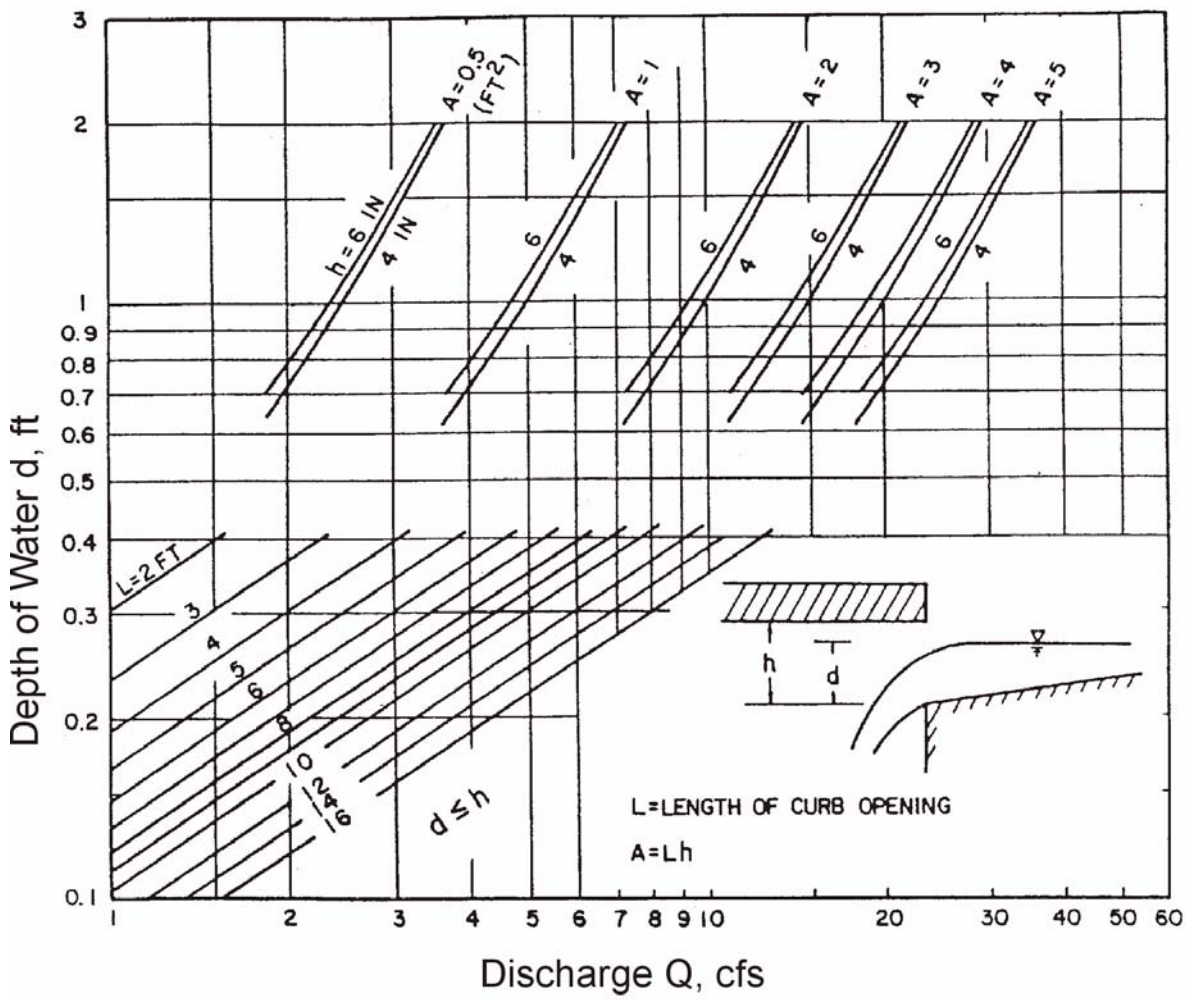
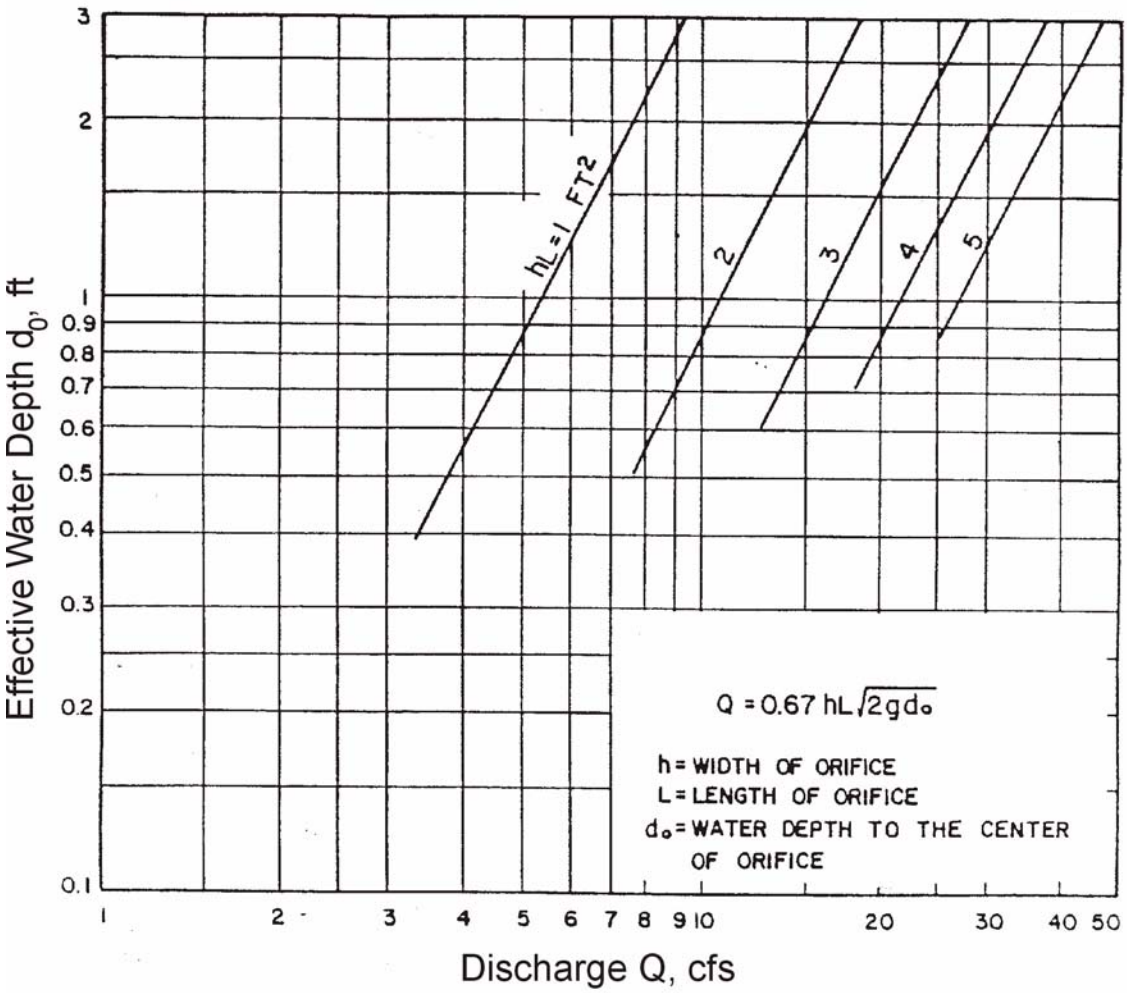


FIGURE 3.13
CURB OPENING INLET CAPACITY FOR INCLINED AND VERTICAL ORIFICE THROATS
(USDOT, FHWA, 1984, HEC-12, Chart 14)



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

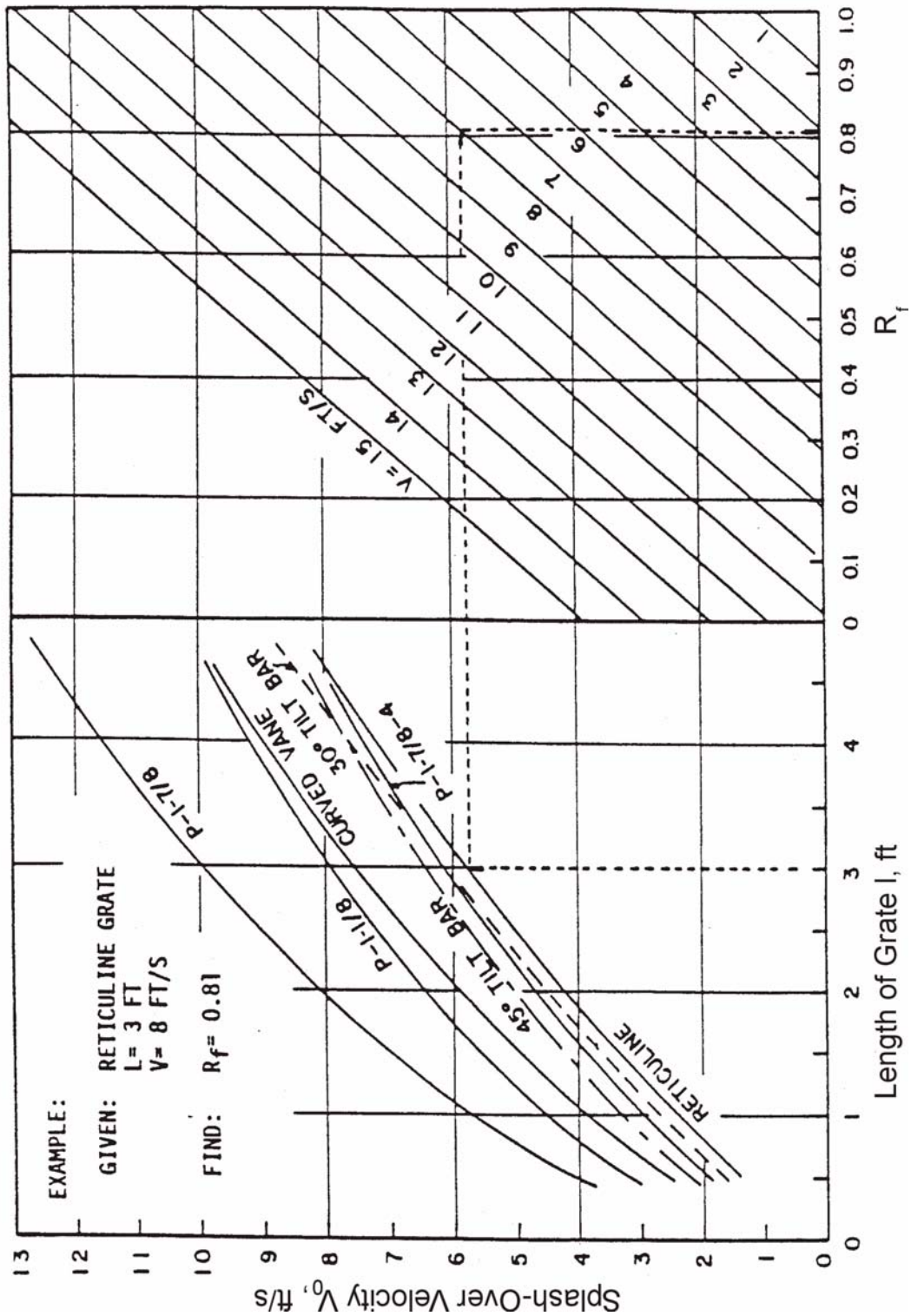
The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed:

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

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:

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

where: S_x = Pavement cross slope, ft/ft

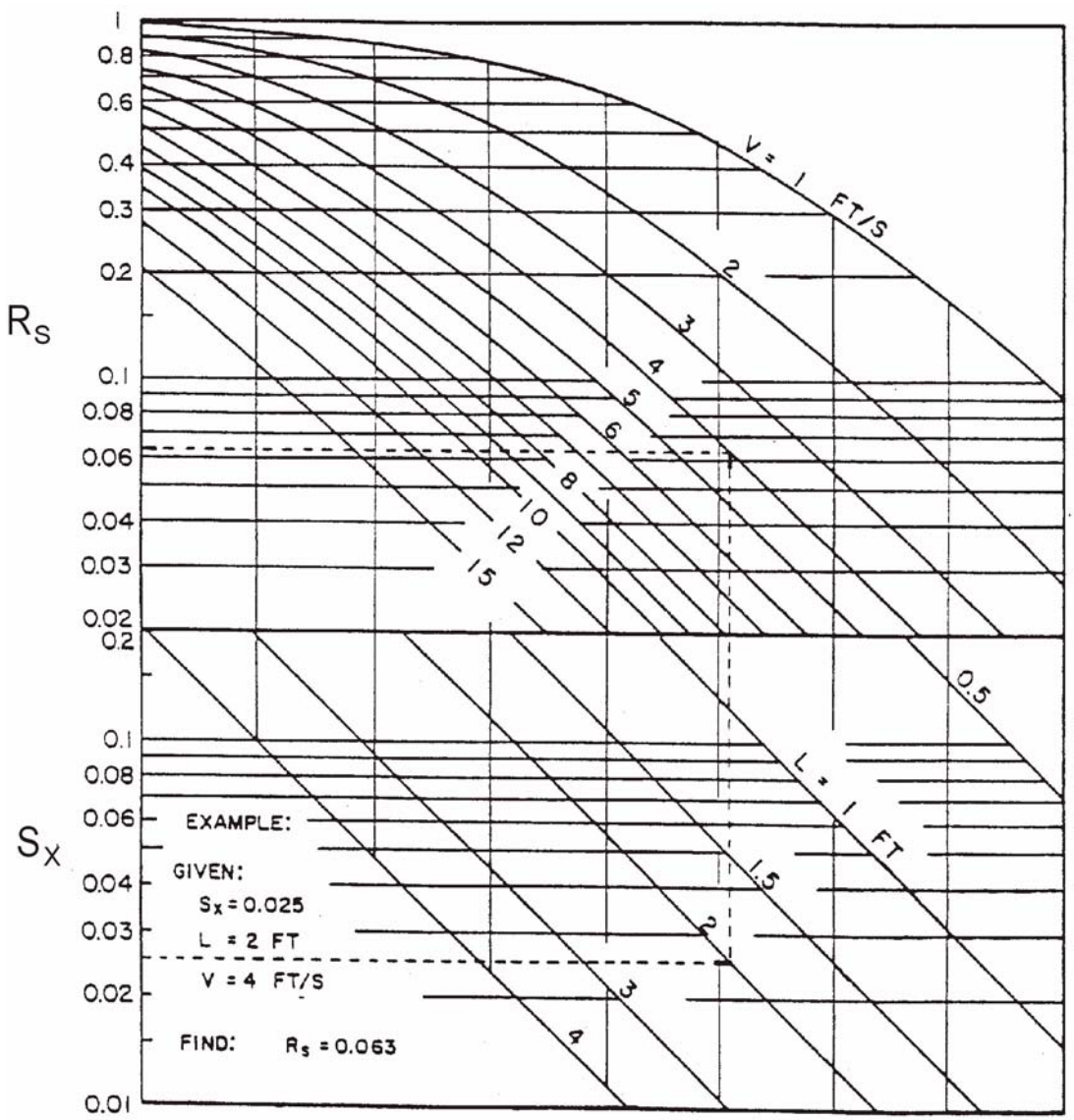
L = Length of grate, ft

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:

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

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:

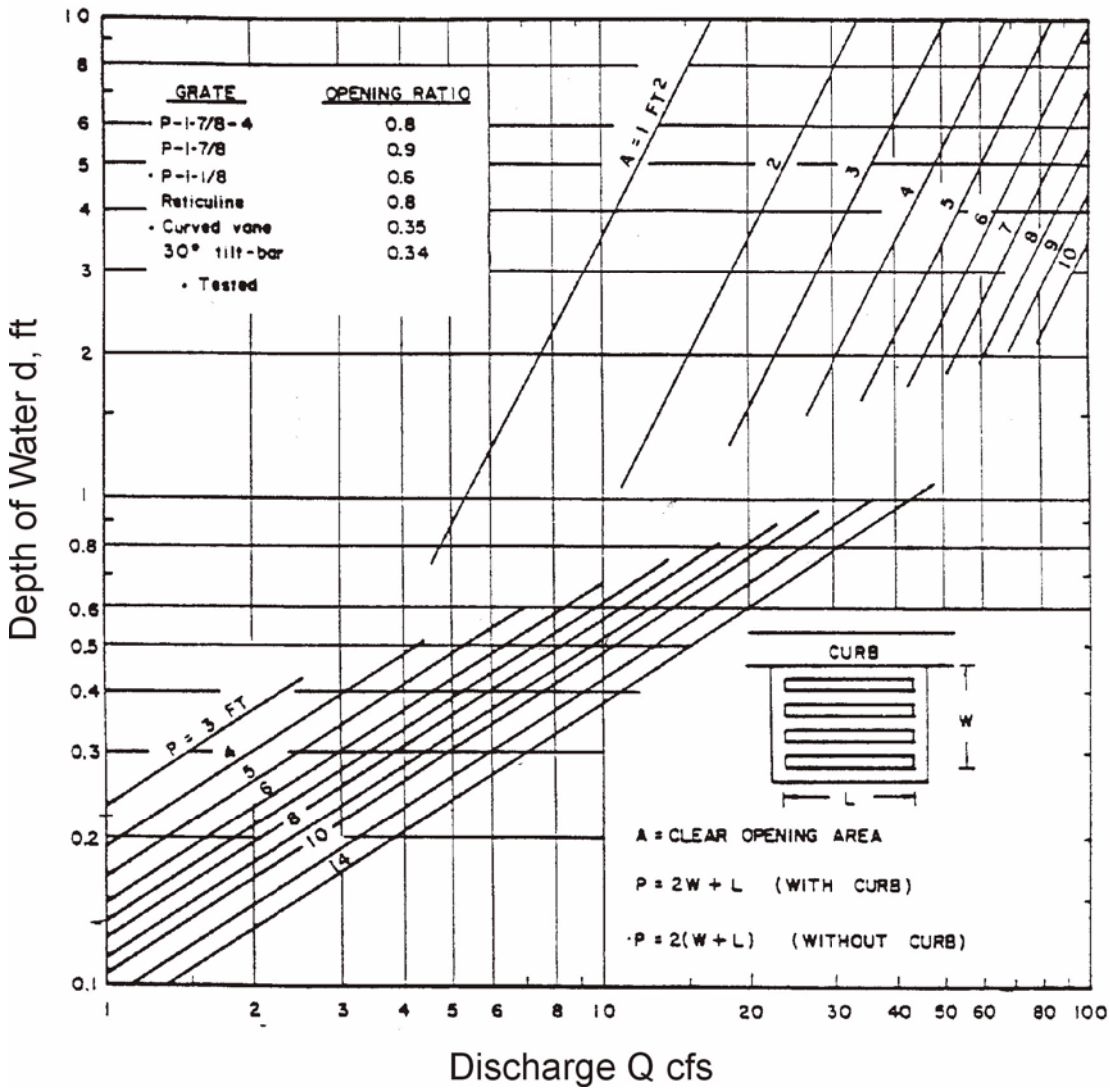
$$Q_i = C_o\,A_g(2gd)^{0.5} \tag{3.22}$$

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):

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

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:

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:

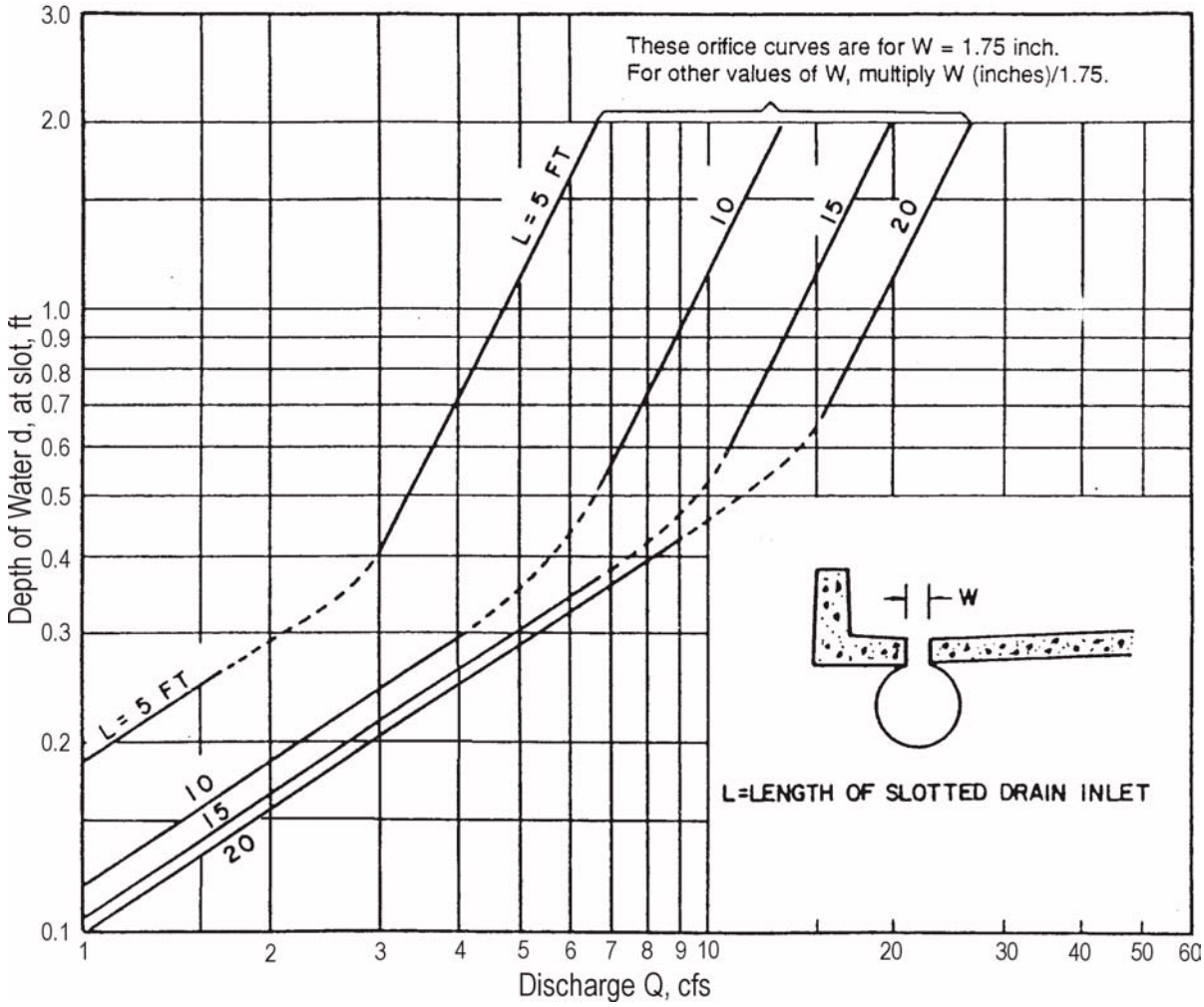
$$Q_i = 0.94Ld^{0.5} \tag{3.25}$$

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.

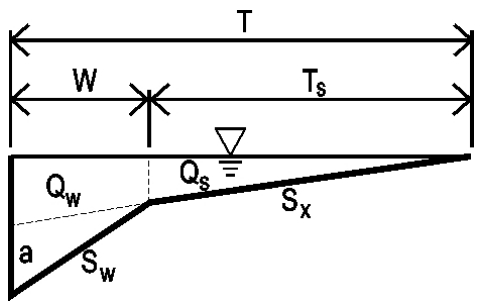
4. Size a catch basin to intercept the calculated flow. Determine the efficiency of the catch basin and determine the flow rate, if any, that will by pass the catch basin.

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 Details, a is typically 0.37 inches.

Step 1:

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

$$T = W + T_s$$

$$T_s = T - W = 12 \text{ ft} - 1.42 = 10.58 \text{ ft}$$

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.

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

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

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

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

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

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 3.6 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.

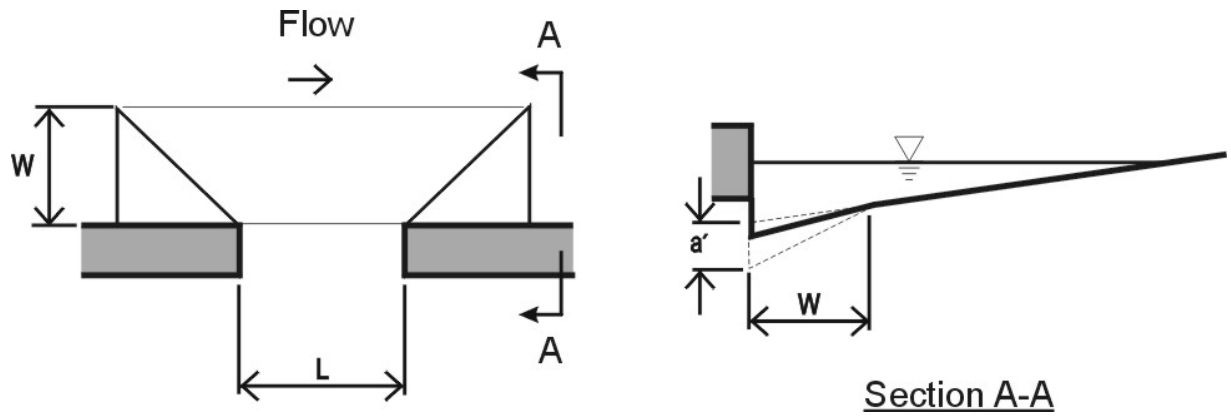
$$E_0 = 0.33$$

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

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

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 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.

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

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

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 (3.7) solve for length.

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

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

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) $= L_t$ x 1.25

= 16.6 x 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.

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

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

$$Q_i = Q \times E$$

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

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

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

$$= 0.7\ cfs$$

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:

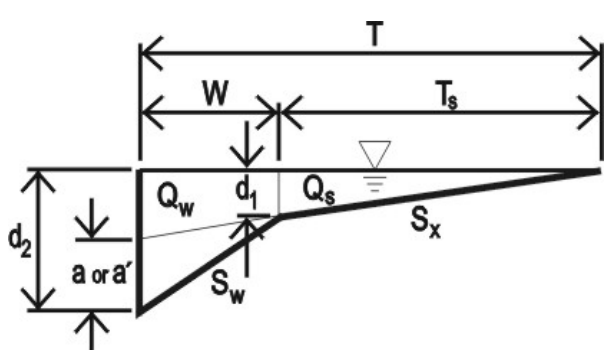
Determine flow rate Q_w in gutter width:

$Q_w = Q - Q_s$

$= 4 - 2.64 = 1.36$ *cfs*

Step 2:

Determine velocity of flow in gutter width; *W*.



Determine d_1 and d_2 in figure above

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

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

Determine flow area of Q_w.

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

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

$$= 0.376\ sq\ ft$$

Use $Q_w = VA_w$ to determine velocity

$$\frac{Q_w}{A_w} = V = \frac{1.36cfs}{0.376sqft} = 3.6fps$$

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 (3.16) or Figure 3.14 determine the ratio of frontal flow intercepted to total frontal flow.

$$R_f = 1 - 0.09\ (V - V_o) = 1 - 0.09\ (3.6\ fps - 6.1\ fps) \qquad (3.16)$$

$$= 1.22 \text{ say } 1.0 \text{ or } 100\%$$

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 x 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 (3.17) 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.

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

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 (3.18) determine the efficiency of the grate.

$E = R_f E_o + R_s (1 - E_o)$(3.18)

= 0.5 x 0.35 + 0.09 x (1 - 0.35)

= 0.234

Step 7:

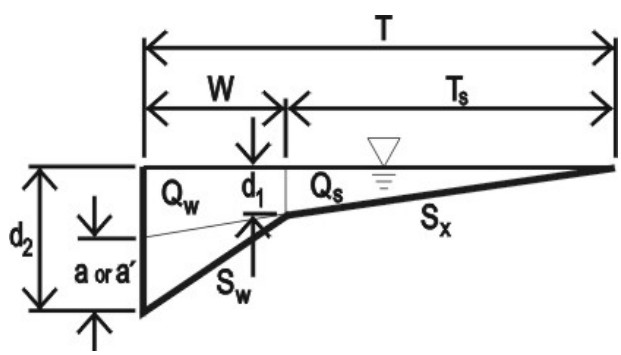
Determine flow rate Q_i intercepted

$Q_i = (Q)(E) = 4 \text{ x } 0.234 = 0.94\ cfs$

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 x 1.25 = 20.75

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

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

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

Q_{ic} = Q x E

Q_{ic} = 4 cfs x 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 a $T_s = 4.2$ 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: $T = T_s + W = 4.2\text{ ft} + 1.42\text{ ft} = 5.62\text{ ft}$

$d_1 = 0.084\,ft$

$d_2 = 0.28\,ft$

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.

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

Step 10:

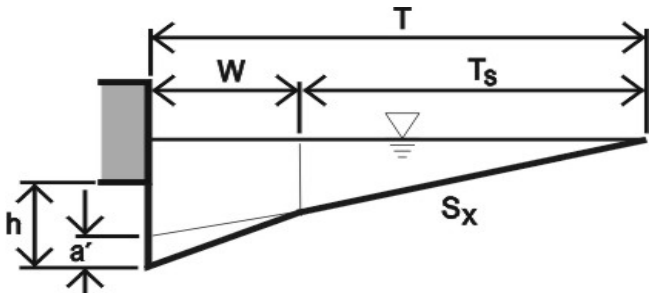
Determine the total flow (Q_i) intercepted by the combination catch basin.

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

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

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)

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

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

Step 2:

Check that

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

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

$$0.24 \text{ ft} < 0.58 \text{ ft}$$

Step 3:

Using Equation (3.11) determine length

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

or

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

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.

$P = 15$

$P = L + 1.8W$

$P - 1.8W = L$

$$15 - (1.8)(1.42) = 12.44 \text{ feet}$$

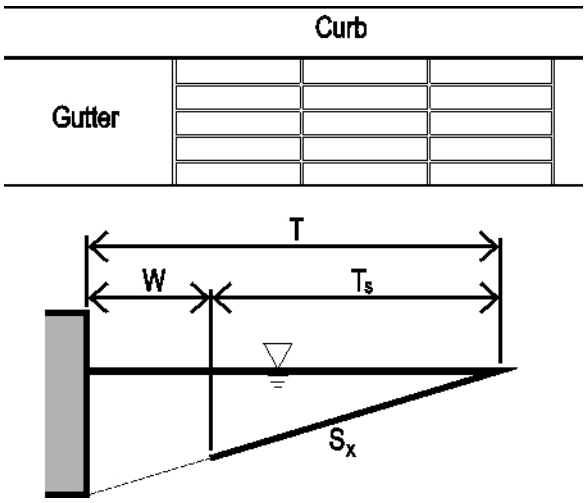
Step 4:

Apply clogging factor to inlet length

$$L \times \text{clogging factor} = 12.44 \times 1.25 = 15.55 \text{ feet}$$

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)

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

Step 2:

Use Equation (3.20) 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.

$$Q_i = C_w P d^{1.5} \qquad (3.20)$$

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

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, 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]

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4 STORM DRAINS

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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 Mannings 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 4.1will 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 4.1details 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)

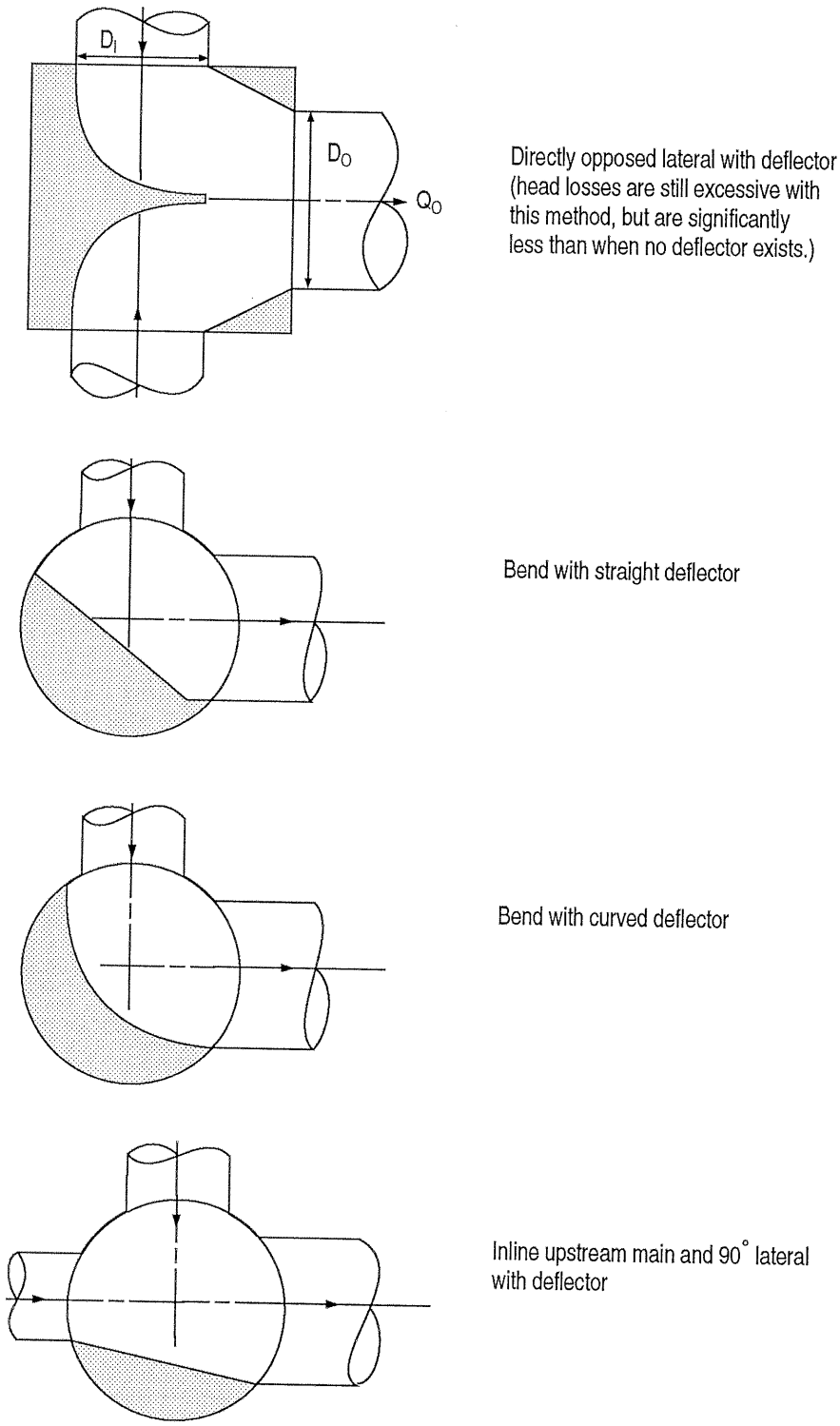
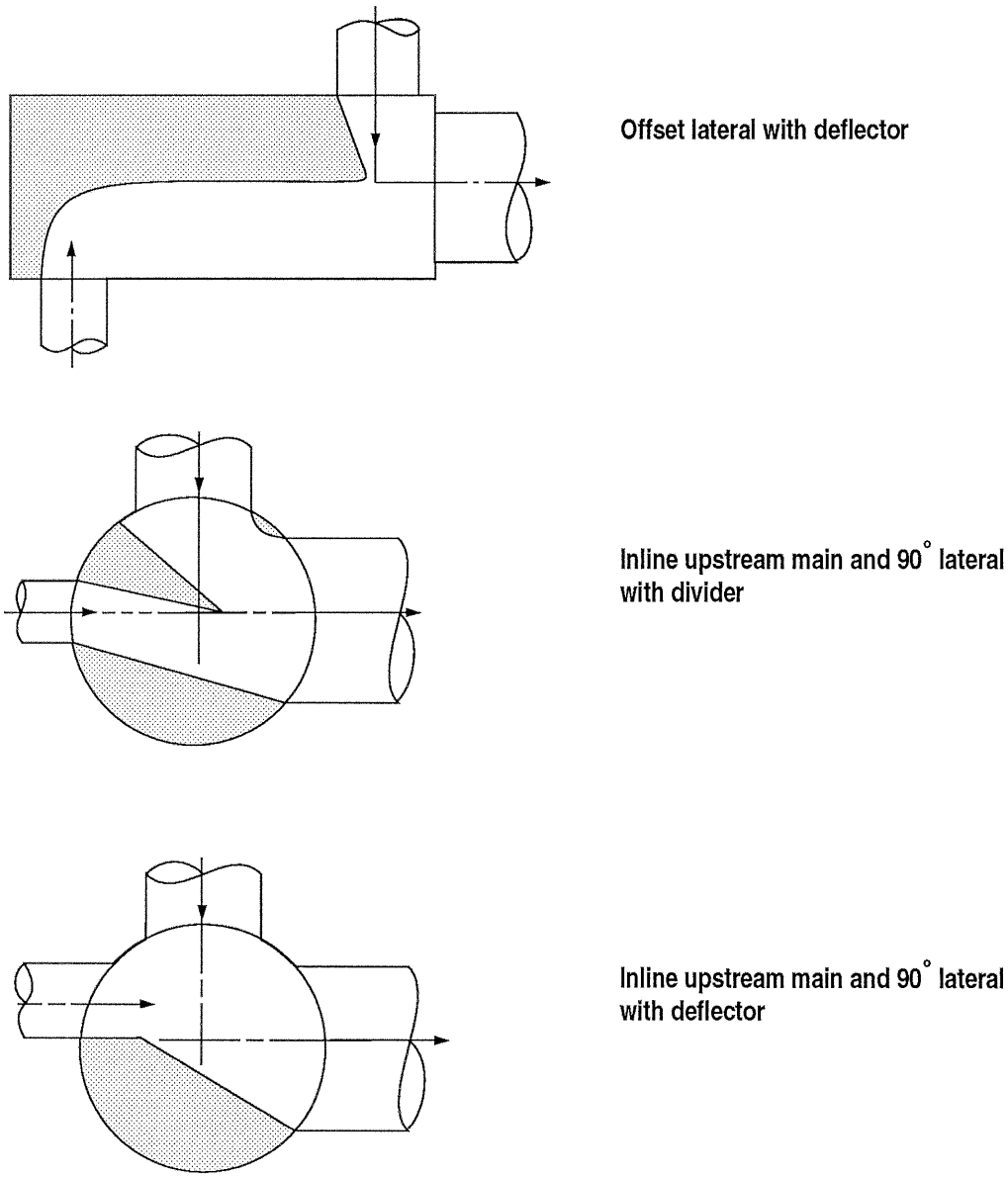


FIGURE 4.2
INEFFICIENT MANHOLE SHAPING



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 7 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 Chapters 5 and 7 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 pro-

vided. 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 7 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.

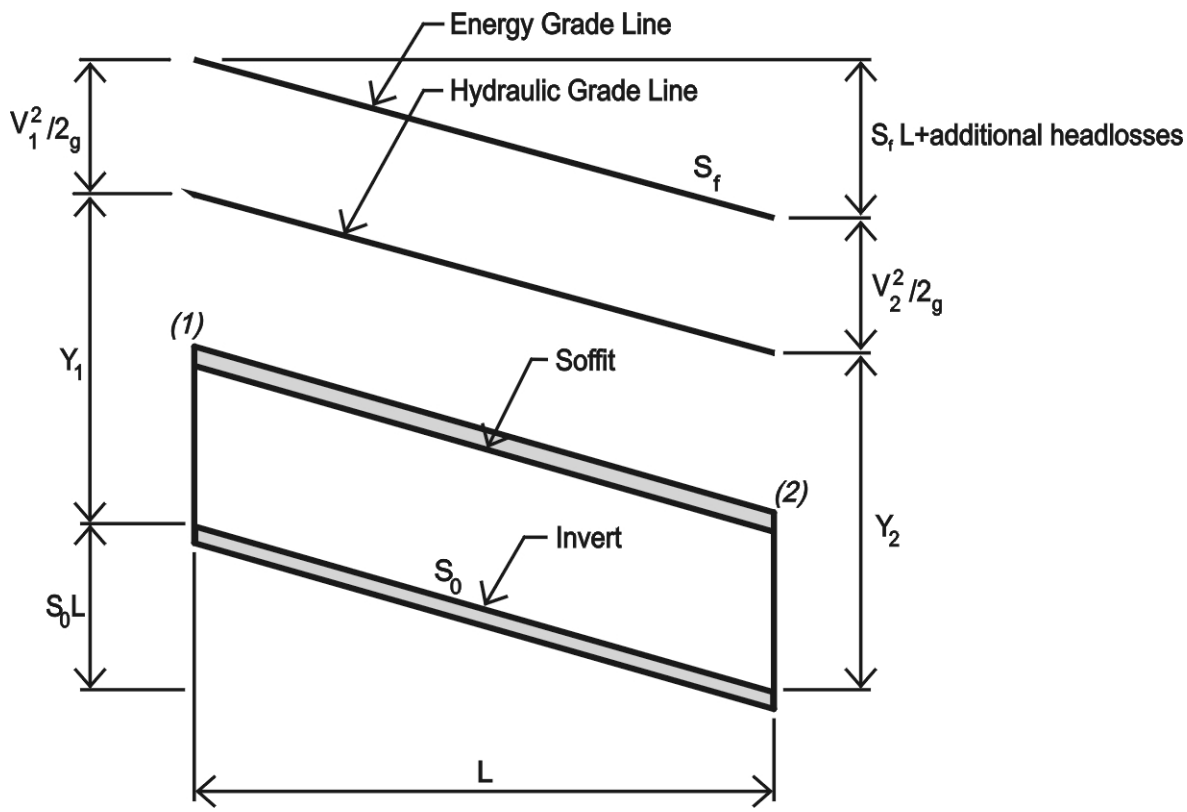
Energy Equation

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

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

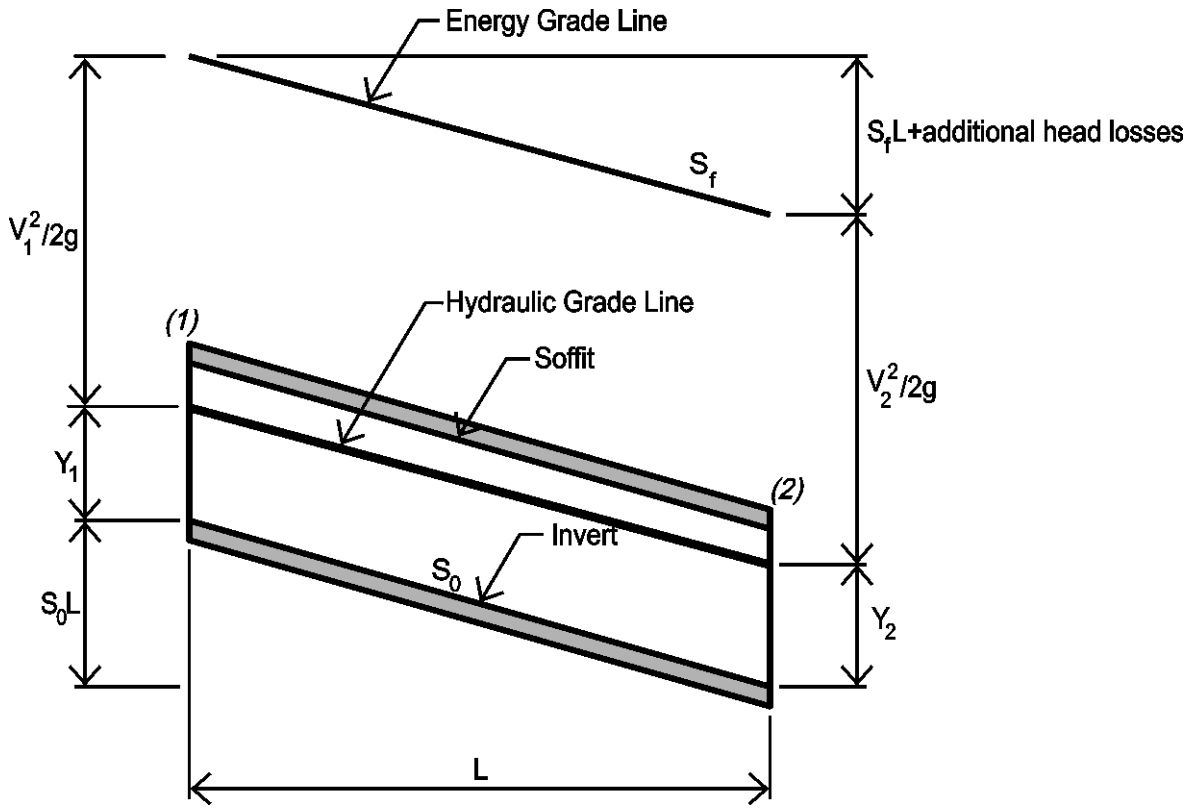
The various terms used in Equation (4.1) are identified in Figure 4.3 and Figure 4.4. Minor losses have been included in the energy equation because of their importance in calculating hydraulic grade line profiles.

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_l, 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 ft	Comments

n = Manning's roughness coefficient

Q = Rate of flow

V = Velocity

$V^2/2g$ = Velocity head

K = $2gn^2 / 2.21$

S_f = Friction slope

h_f = Headloss due to friction

h_b = Headloss due to a bend

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 construction

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

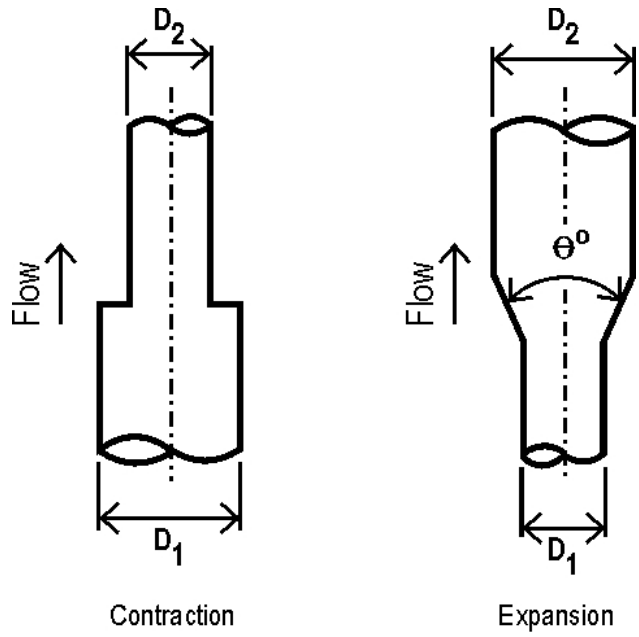


TABLE 4.2
STORM SEWER ENERGY LOSS COEFFICIENTS UNDER OPEN CHANNEL CONDITIONS
(ASCE, 1992)

(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 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)

	Angle of Cone, degrees													
D_2/D_1	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 4.7A, Figure 4.7B, and Figure 4.7C 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 4.7A through Figure 4.7C) 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 4.7A
FORMED OR PREFAB STORM DRAIN JUNCTION

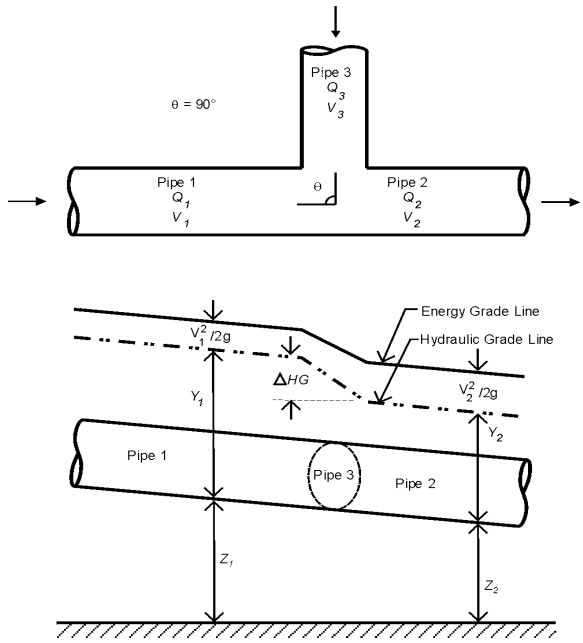


FIGURE 4.7B
STORM DRAIN JUNCTION AT MANHOLE WITH ALIGNED CROWNS UNDER PRESSURE FLOW

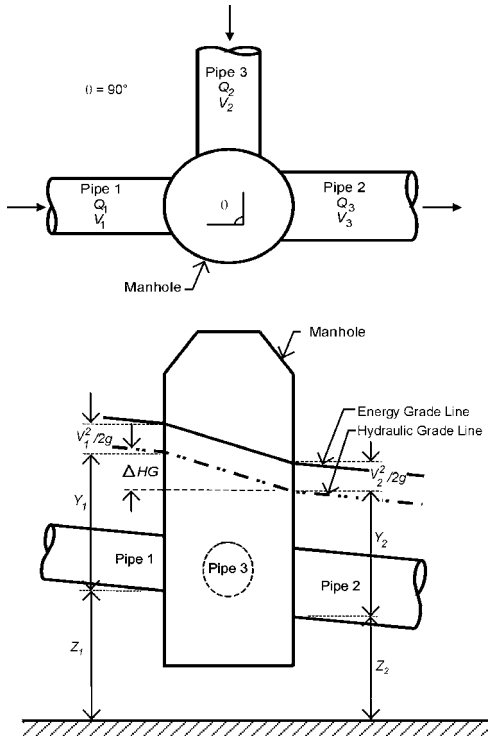
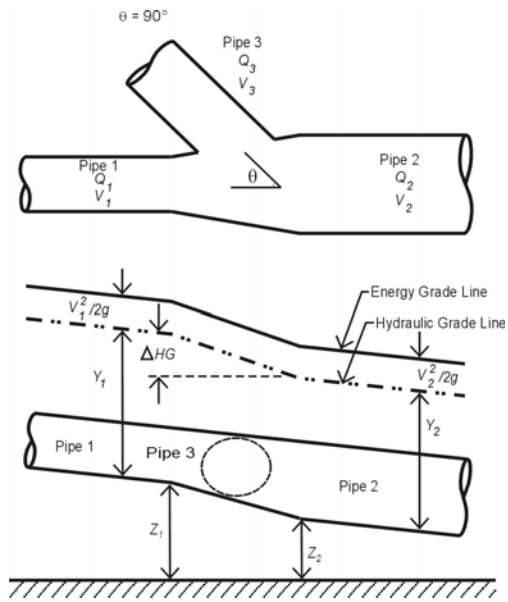


FIGURE 4.7C
FORMED STORM DRAIN JUNCTION WITH ALIGNED CROWNS UNDER PRESSURE FLOW



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.

$$\Delta HG \frac{A_1 + A_2}{2} \;=\; \frac{Q_2 V_2 - Q_1 V_1 - Q_3 V_3 Cos\,\theta}{g} \qquad (4.10a)$$

or

$$\Delta HG = \frac{\left(\frac{Q_2V_2 - Q_1V_1 - Q_3V_3 Cos\,\theta}{g}\right)}{\frac{A_1 + A_2}{2}}$$

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 4.7), 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.

$$\frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3 Cos\,\theta)}{(A_1 + A_2)g} + V_1^2/2g - V_2^2/2g = h_j \qquad (4.10b)$$

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

$$\frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3 Cos\,\theta)}{(A_1 + A_2)g} + V_1^2/2g - V_2^2/2g + \left(\frac{S_{f1} + S_{f2}}{2}\right)L = h_j \qquad (4.10c)$$

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.

$$h_j = \frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3 Cos\,\theta)}{(A_1 + A_2)g} + V_1^2/2g - V_2^2/2g + k_{je}\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \quad (4.10d)$$

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 4.7A through Figure 4.7C 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 APPENDIX 4-A PRESSURE MOMENTUM ANALYSIS.

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: $h_{mh} = 0.05\frac{V^2}{2g}$ (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 4.8.

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

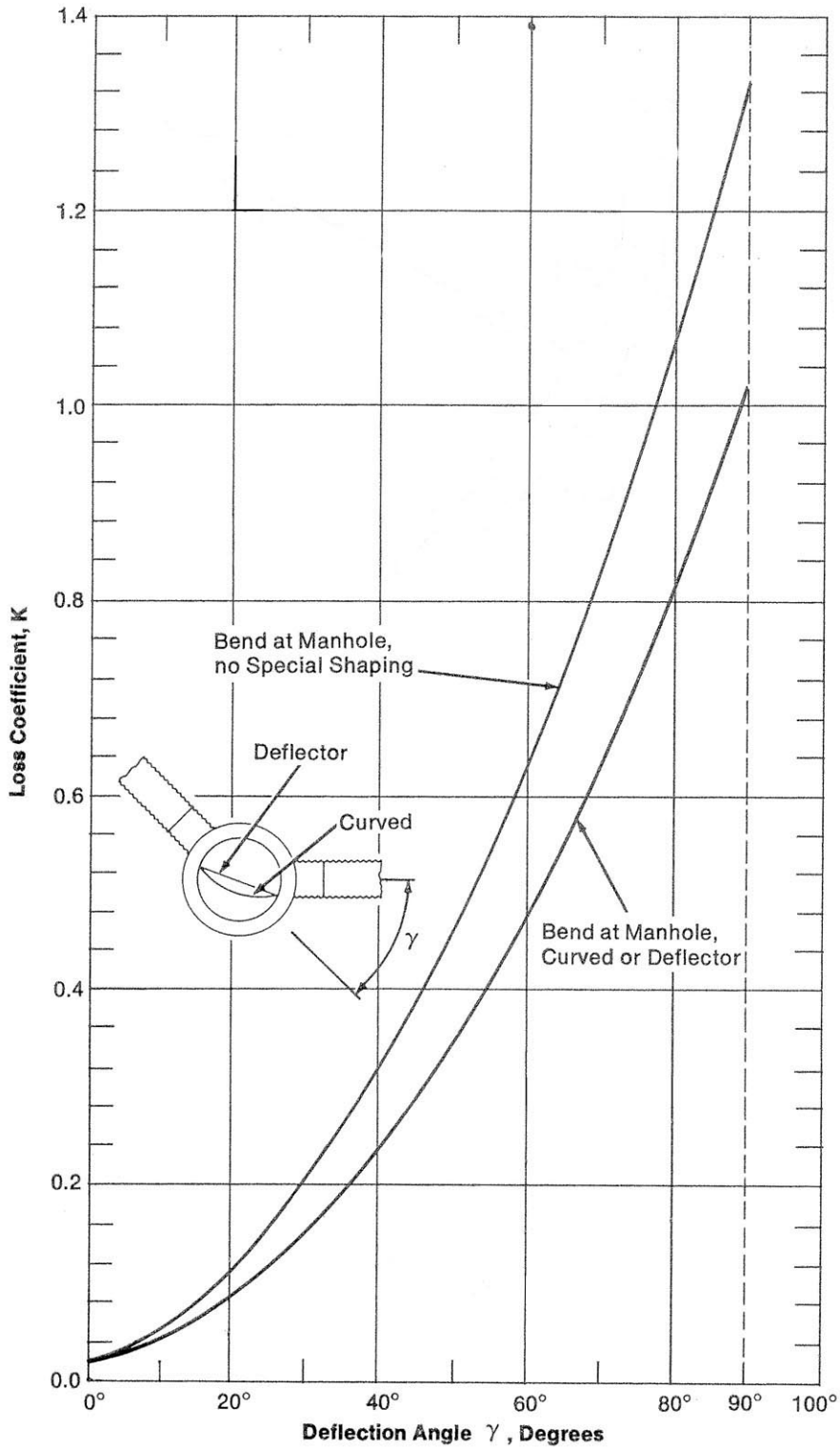
where: h_{mh} = Headloss due to a manhole, ft

k_b = Bend loss coefficient

V = Velocity, ft/sec

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

FIGURE 4.8
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 7.2.5 Trashracks and Access Barriers of Chapter 7.

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 = HeadlossHeadloss 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.

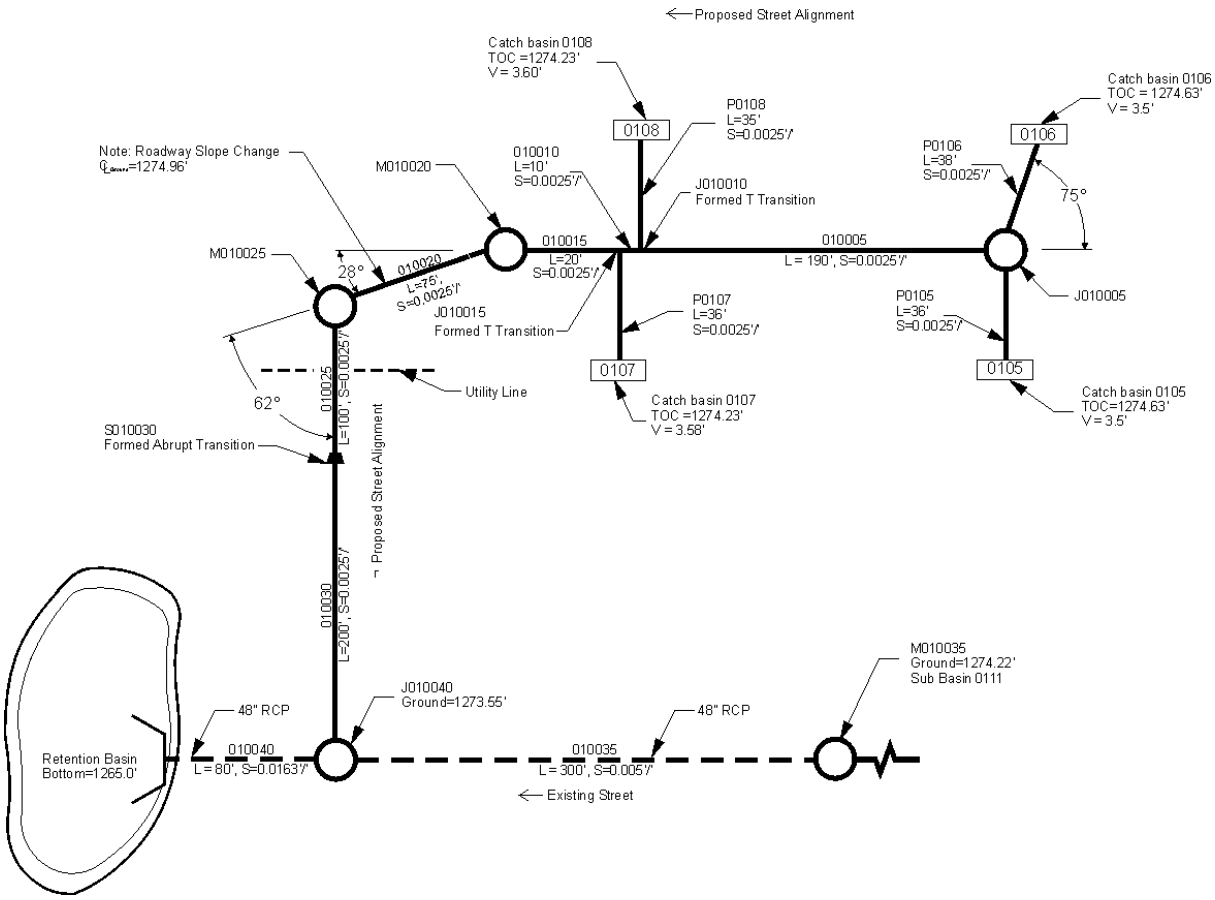
4.6 DESIGN EXAMPLE

OBJECTIVE:

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

PROBLEM STATEMENT:

FIGURE 4.9
INITIAL STORM DRAIN LAYOUT FOR EXAMPLE PROBLEM



Determine the initial and final system design of the initial storm drain layout presented in Figure 4.9 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 catchbasin 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 Details* 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. 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 Details.

Step 2. 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 4.9 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 hydrologic 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 4.10 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:

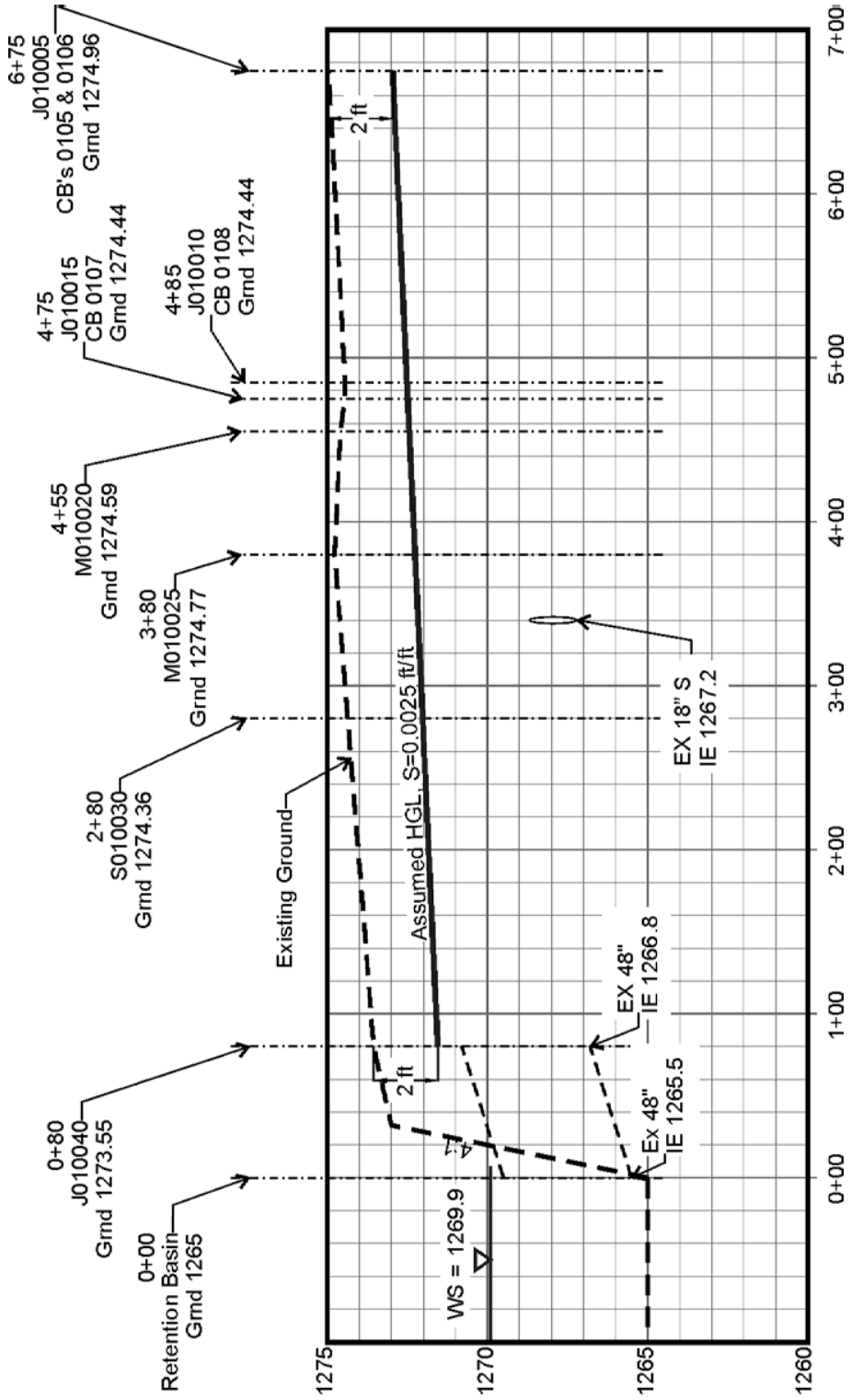
$$D = 1.33\left(\frac{nQ}{\sqrt{S}}\right)^{3/8}$$

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

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 4.10
HGL PROFILE FOR INITIAL STORM DRAIN LAYOUT



2.3 The peak discharge at J010005 was calculated utilizing the following steps:

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

A_T = 1.33 acres

2.3.2 Calculate the weighted runoff coefficient, C_w:

$$C_w = \frac{A_1C_1 + A_2C_2 + \ldots + A_{n+1}C_{n+1}}{A_1 + A_2 + \ldots A_{n+1}}$$

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

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 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:

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

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

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/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 Man-

nings 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 = $\frac{190 \text{ ft}}{(1.92 \text{ ft/sec})(60 \text{sec}/\text{min})}$ = **<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 = 2.00 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:

$$Q_d = C_w i A_T$$

$$Q_d = (0.91)(2.60)(2.00)$$ = **<u>4.7 cfs</u>**.

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:

$$Q_d = C_w i A_T$$

$$Q_d = (0.91)(2.60)(2.47)$$ = **<u>5.8 cfs</u>**.

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:

$$Q_d = C_w i A_T$$

$$Q_d = (0.853)(2.35)(52.47) = \underline{\mathbf{105.2\ cfs}}.$$

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.2 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 8. 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 Main Line Hydraulic Grade Line 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.

$$h_o = \frac{V_o^2}{2g} \qquad (4.16)$$

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

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

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

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

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

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:

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

$R = \frac{D}{P}$ (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:

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

Calculate the velocity:

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

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

Solving Equation (4.4):

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

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

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

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

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 = **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 elev.) + 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).

$$h_j = \frac{2(Q_2 V_2 - Q_1 V_1 - Q_3 V_3 Cos\,\theta°)}{(A_1 + A_2)g} + \frac{V_1^2}{2g} - \frac{V_2^2}{2g} \qquad (4.10b)$$

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):

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

$$h_f = (0.0050)(300)\text{ ft}$$ = **<u>1.50 ft</u>**

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

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

$$h_{mh} = 0.05\frac{8.12^2}{(2)(32.2)}$$ = **<u>0.05 ft</u>**

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.

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

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

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

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

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:

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

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).

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

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

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):

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

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

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 4.8 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 4.8, 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):

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

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

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

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

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

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):

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

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

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 4.8 to estimate bend loss coefficient and Equation (4.12) to estimate head-loss 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 4.8, determine the bend loss coefficient (k_b):

(k_b) = 0.19

4.7.2.3 Calculate the manhole headloss using Equation (4.12):

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

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

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

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

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

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.

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

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

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

$$h_f = (0.0030) \times (20 \text{ ft}) =$$ **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).

$$h_j = \frac{2(Q_2 V_2 - Q_1 V_1 - Q_3 V_3 Cos\,\theta°)}{(A_1 + A_2)g} + \frac{V_1^2}{2g} - \frac{V_2^2}{2g} \qquad (4.10b)$$

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):

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

$h_f = (0.0020)(10\text{ ft}) =$ **<u>0.02 ft</u>**

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) = **<u>1271.81 ft</u>**

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

4.9.2.2 Calculate junction loss utilizing Equation (4.10b).

$$h_j = \frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3 Cos\,\theta°)}{(A_1 + A_2)g} + \frac{V_1^2}{2g} - \frac{V_2^2}{2g} \qquad (4.10b)$$

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 P0108 = 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):

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

$$h_f = (0.0010)(190 \text{ ft})$$ = **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.

$$S_f = 0.0049 \frac{1.30^2}{(2)((32.2)(0.31^{4/3}))}$$ = **0.0006 ft/ft**

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

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

$$h_f = (0.0006)(36 \text{ ft})$$ = **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):

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

$$h_f = (0.0007)(35\ \text{ft}) = \underline{\mathbf{0.02\ ft}}$$

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):

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

$$h_f = (0.0012)(38\ \text{ft}) = \underline{\mathbf{0.05\ ft}}$$

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):

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

$$h_f = (0.0002) \times (36\ \text{ft}) = \underline{\mathbf{0.01\ ft}}$$

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:

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

$$h_i = (1 + 0.5)\frac{0.65^2}{(2)(32.2)} = \underline{\mathbf{0.01\ ft}}$$

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 4.11 displays the final HGL profile calculated for the proposed storm drain.

FIGURE 4.11
FINAL HGL PROFILE FOR PROPOSED STORM DRAIN

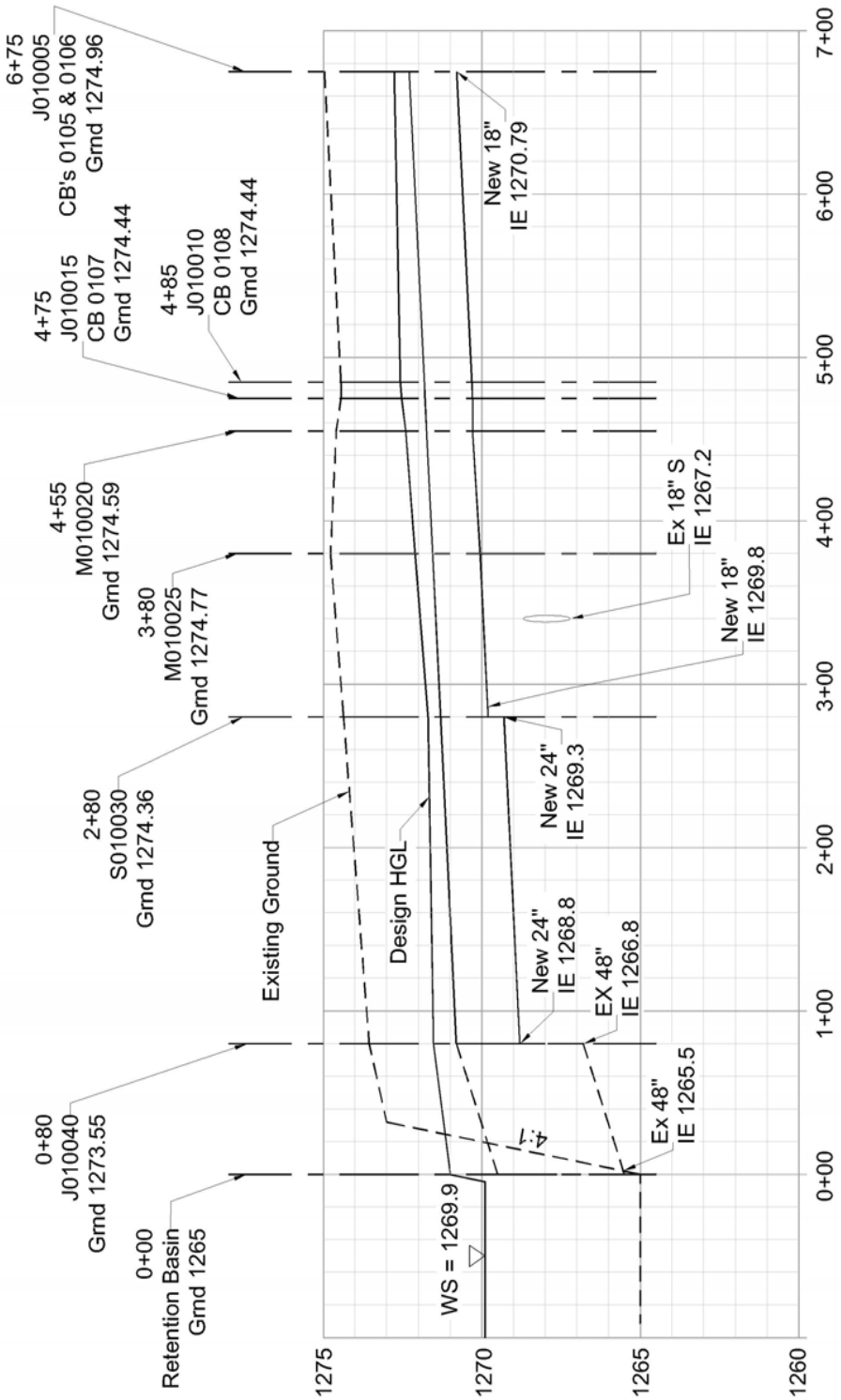


TABLE E.4
HYDRAULIC GRADE LINE CALCULATION SHEET

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

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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:

$$M = wQV / g \text{(lbs/s)}$$

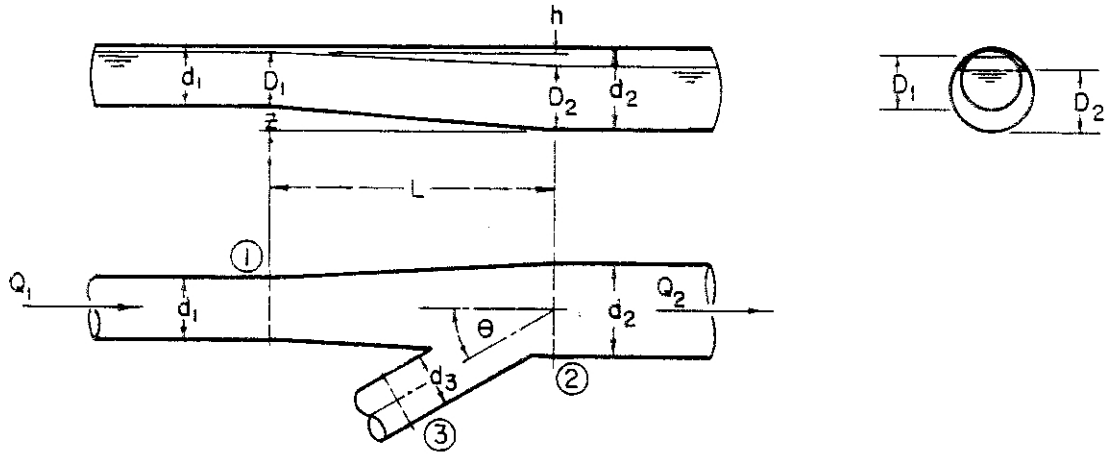
However, since the unit weight of water (w) appears in all terms of the general equilibrium equation it may be omitted and the dimension for P+M becomes feet to the third power.

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)

D/d	K	C	F	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

$$M = k(Q/d)^2 \qquad P = C \cdot d^3$$

$$h_v = F(Q/d^2)^2$$

5 CULVERTS & BRIDGES

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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
A	=	Area of the barrel, sq ft
B	=	Width of culvert opening, ft
C_r	=	Road embankment overtopping discharge coefficient
d	=	Inlet bevel, in
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
Q	=	Rate of flow, cfs
Q_o	=	Rate of flow overtopping roadway, cfs
S	=	Slope, ft/ft
TW	=	Tailwater depth measured from culvert outlet invert, ft
t	=	Time, minutes
t_o	=	Base time used in experiments to derive scour coefficients τ_c, β, and θ
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

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 stormdrain 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 lin-

ing 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 6.1, 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.

$$H_t = \left(\frac{V_a^2}{2g}\right)\sin a \tag{5.1}$$

FIGURE 5.1
BARREL SKEW ANGLE
(USDOT, FHWA, HDS-5, 1985)

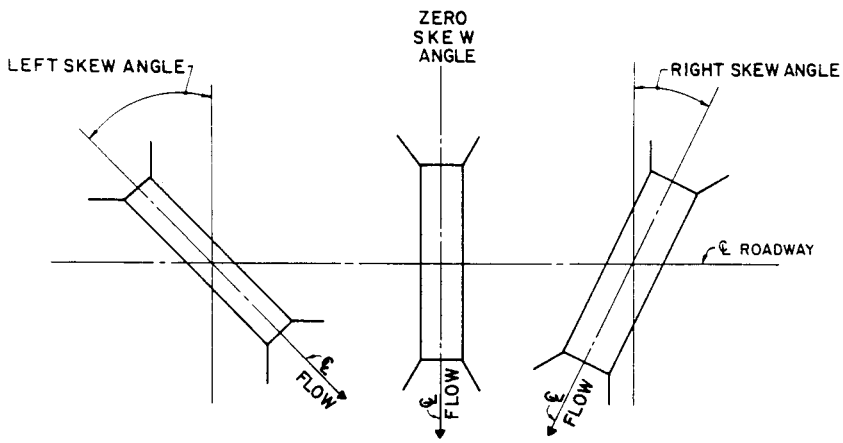


FIGURE 5.2
INLET SKEW ANGLE
(USDOT, FHWA, HDS-5, 1985)

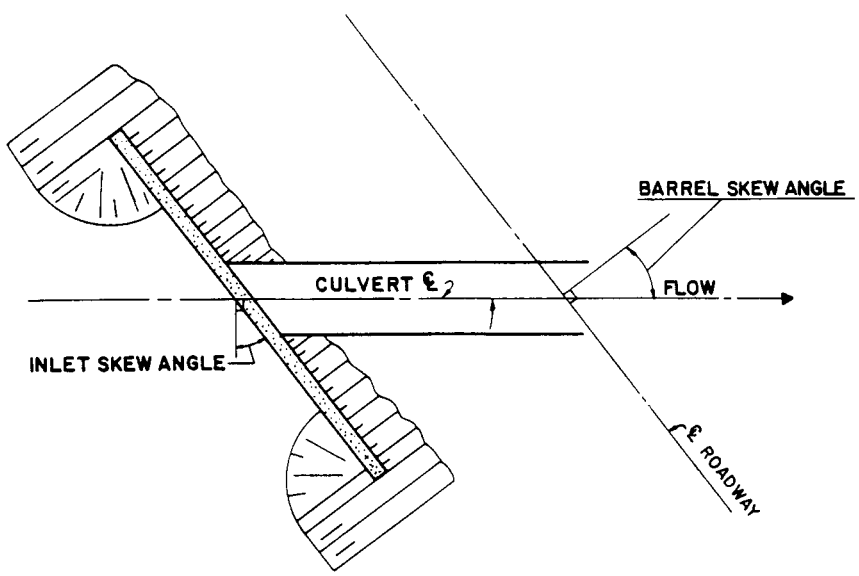
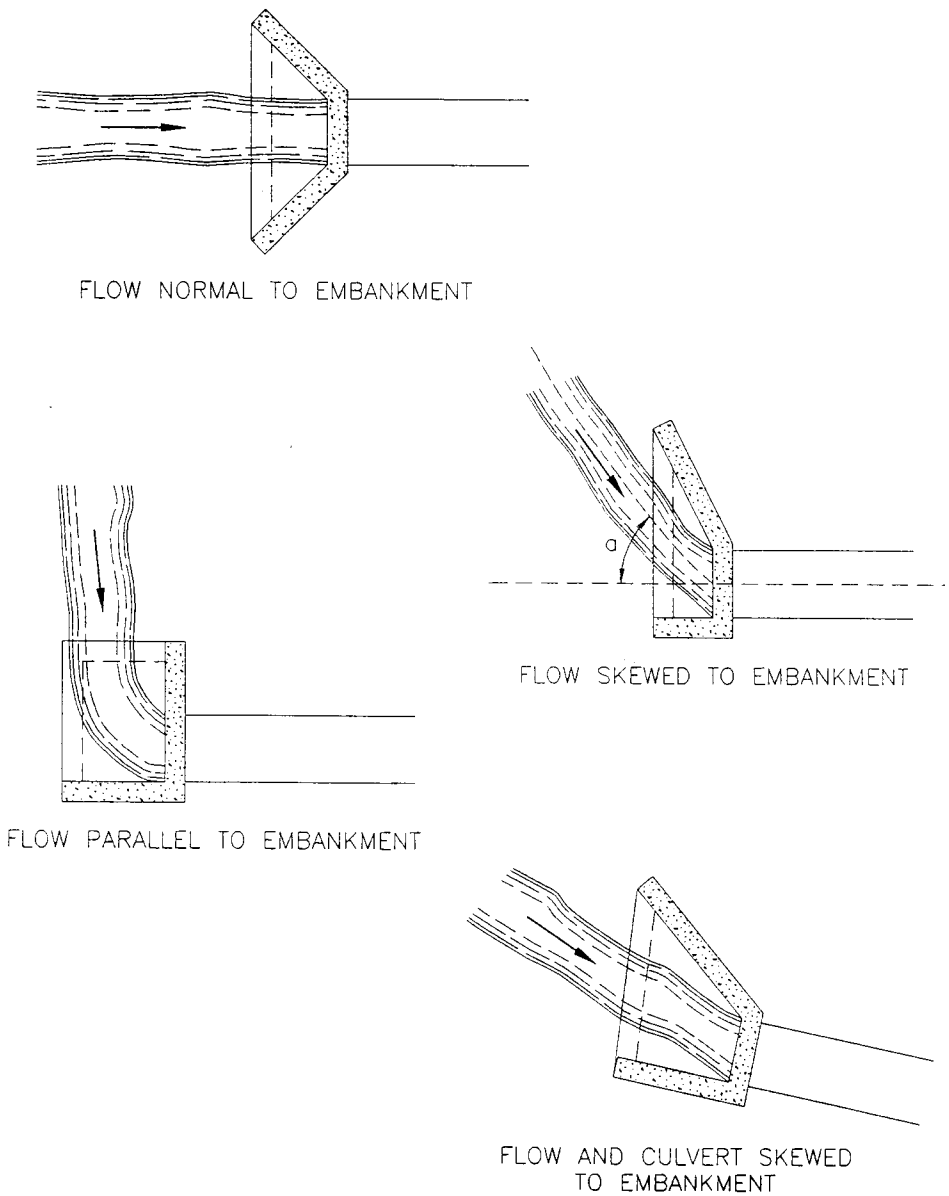


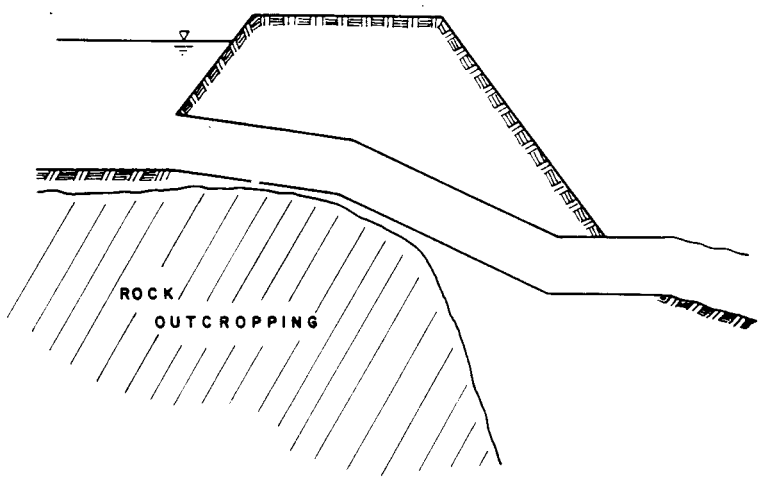
FIGURE 5.3
TYPICAL HEADWALL/WINGWALL CONFIGURATIONS FOR SKEWED CHANNELS
(USDOT, FHWA, HDS-5, 1985)



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:

$$H_b = K_b \frac{V^2}{2g} \tag{5.2}$$

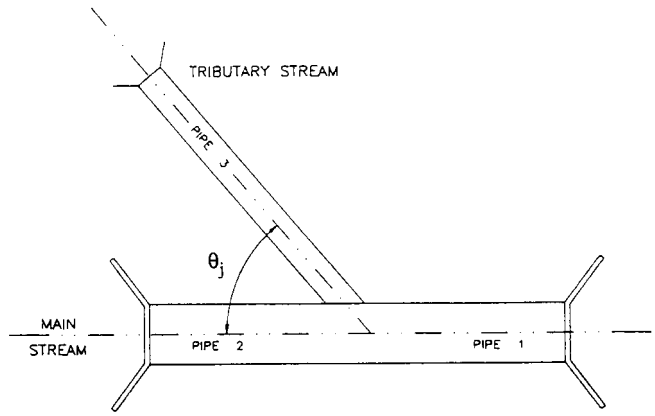
H_b is added to the other outlet losses. See Chapter 4, Storm Drains, to determine loss coefficients (K_b) for bend losses in conduits flowing full.

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 stormdrains 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:

$$y' = \frac{Q_2 V_2 - Q_1 V_1 - Q_3 V_3 \cos\theta_j}{0.5(A_1 + A_2)g} \tag{5.4}$$

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 7, Section 7.6.4.

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)

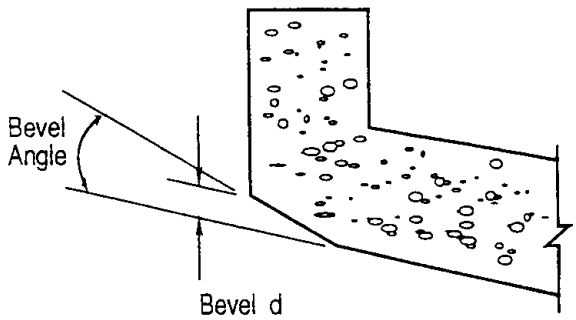


FIGURE 5.7
SIDE-TAPERED INLET
(USDOT, FHWA, HDS-5, 1985)

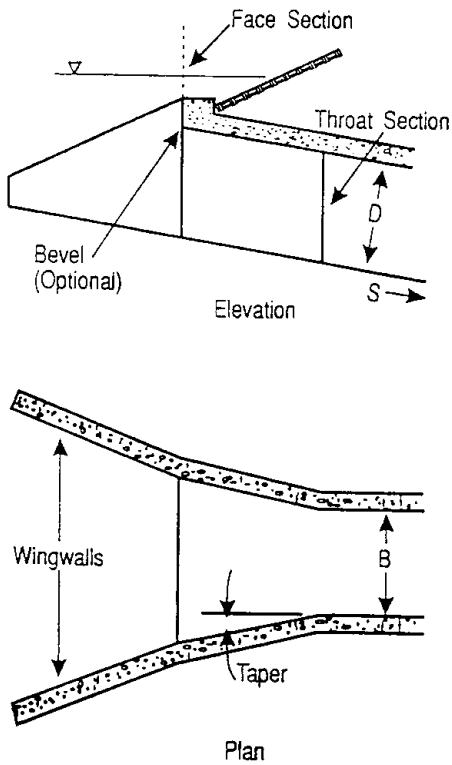


FIGURE 5.8
SLOPE-TAPERED INLET
(USDOT, FHWA, HDS-5, 1985)

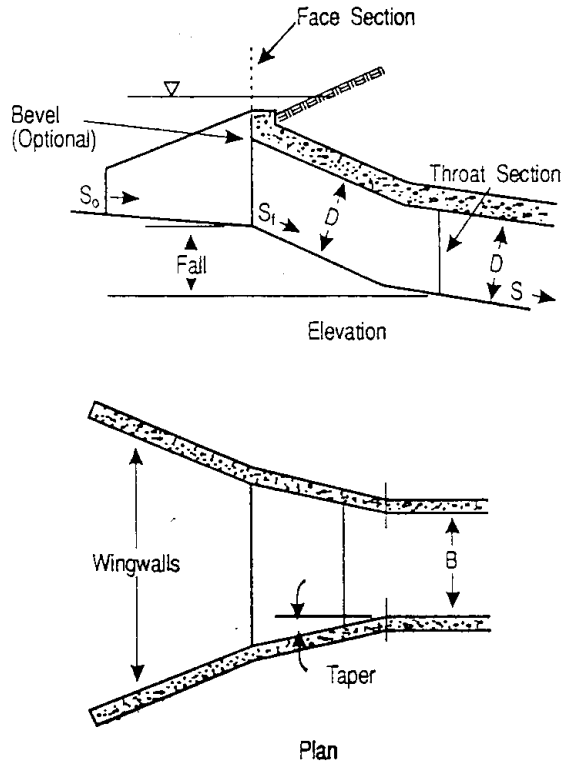
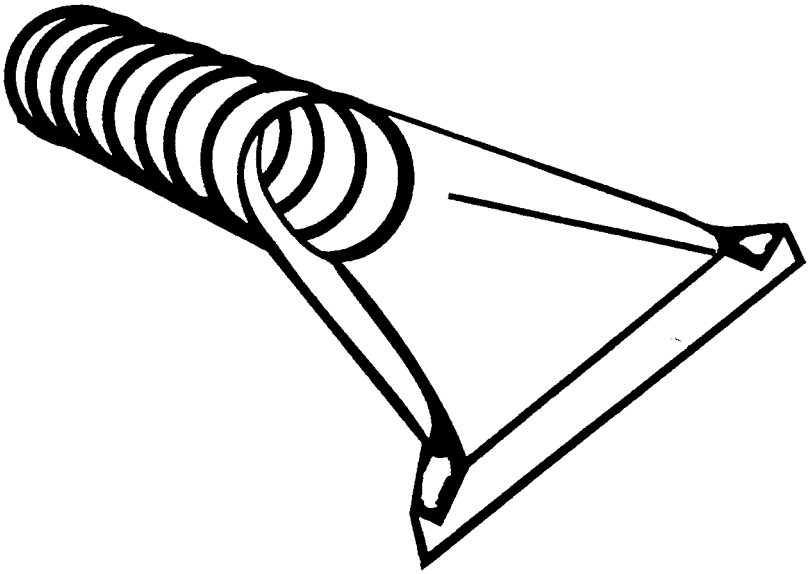


FIGURE 5.9
PREFABRICATED CULVERT END SECTION



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 7, Hydraulic Structures, Section 7.4.

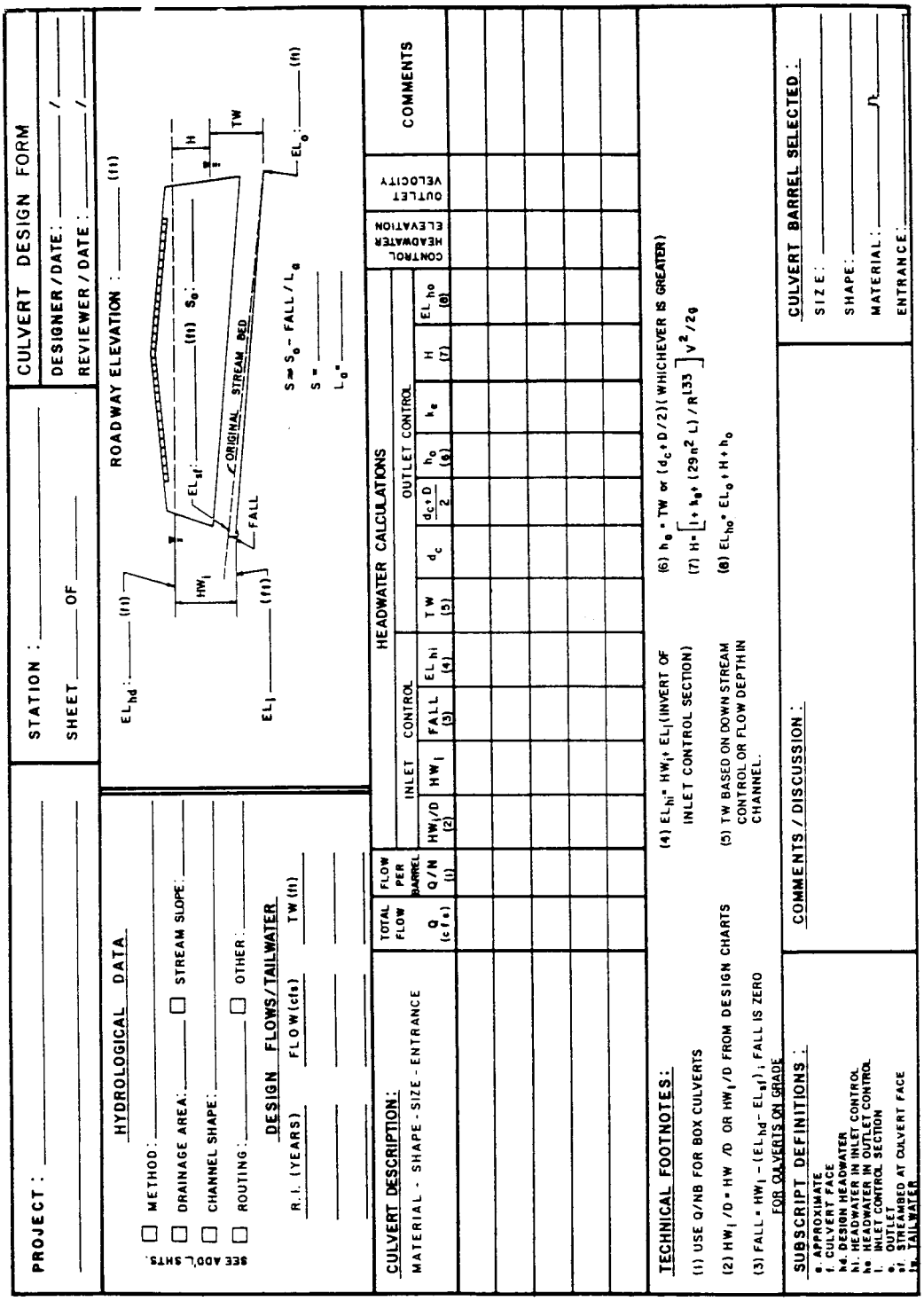
5.3.3 Design Procedures

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.

   $$EL_{hi} = EL_i + HW$$

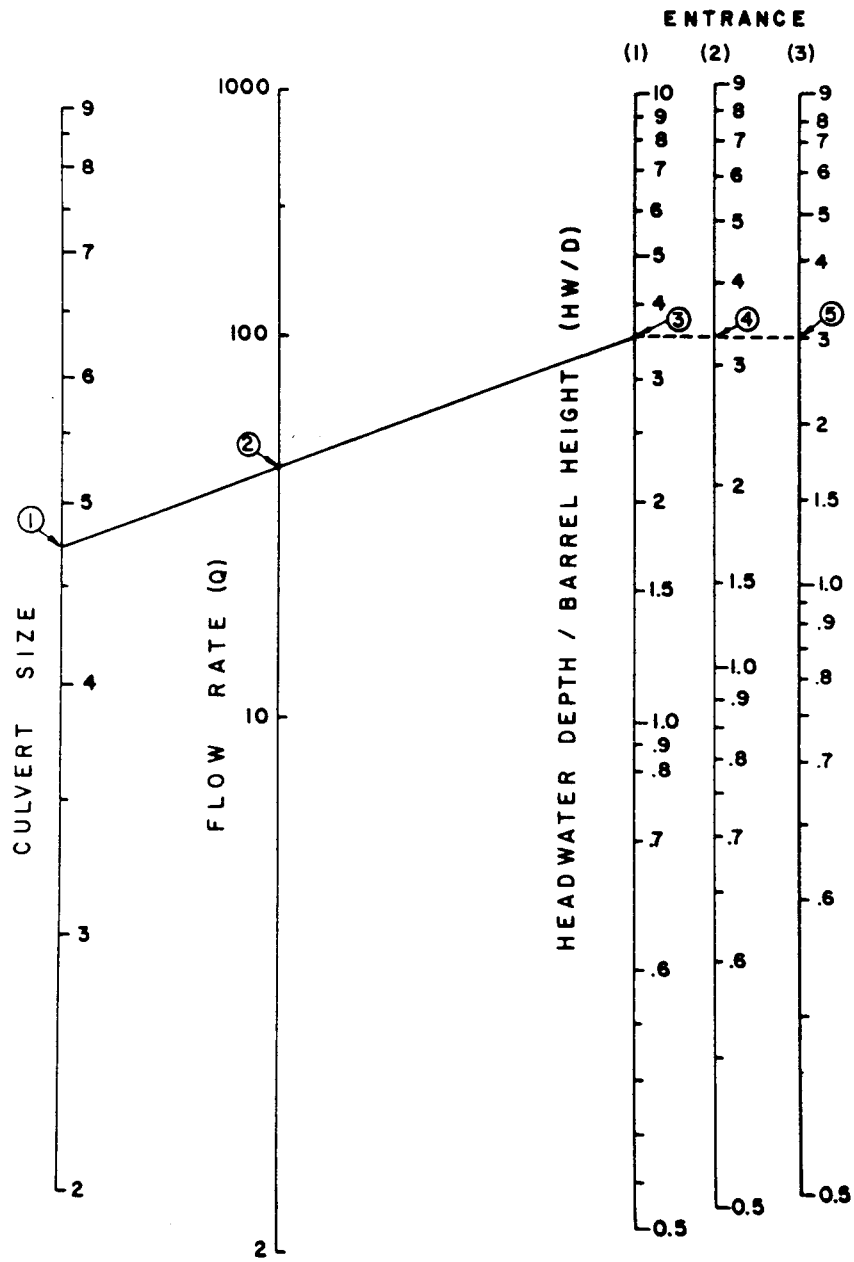
   where EL_i is the invert elevation at the inlet.

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.)

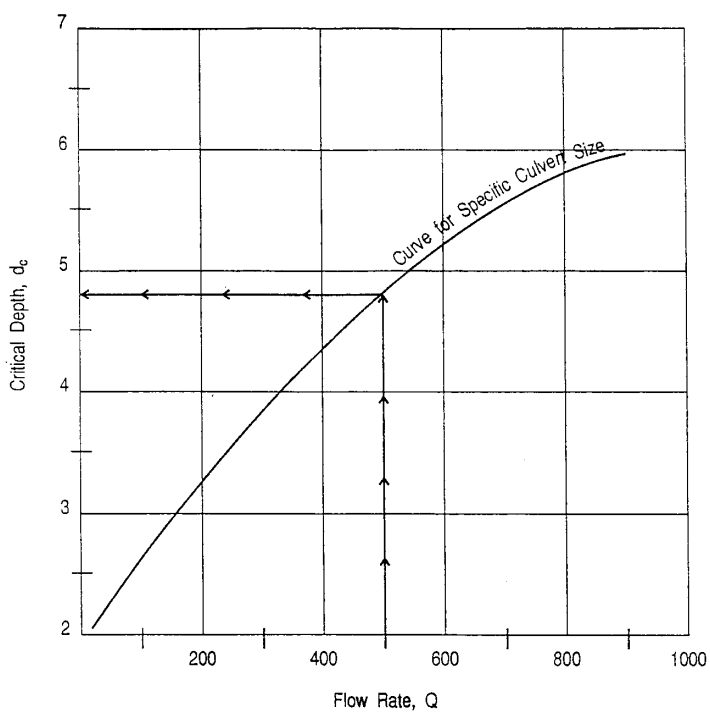
3. Calculate $(d_c + D)/2$

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.

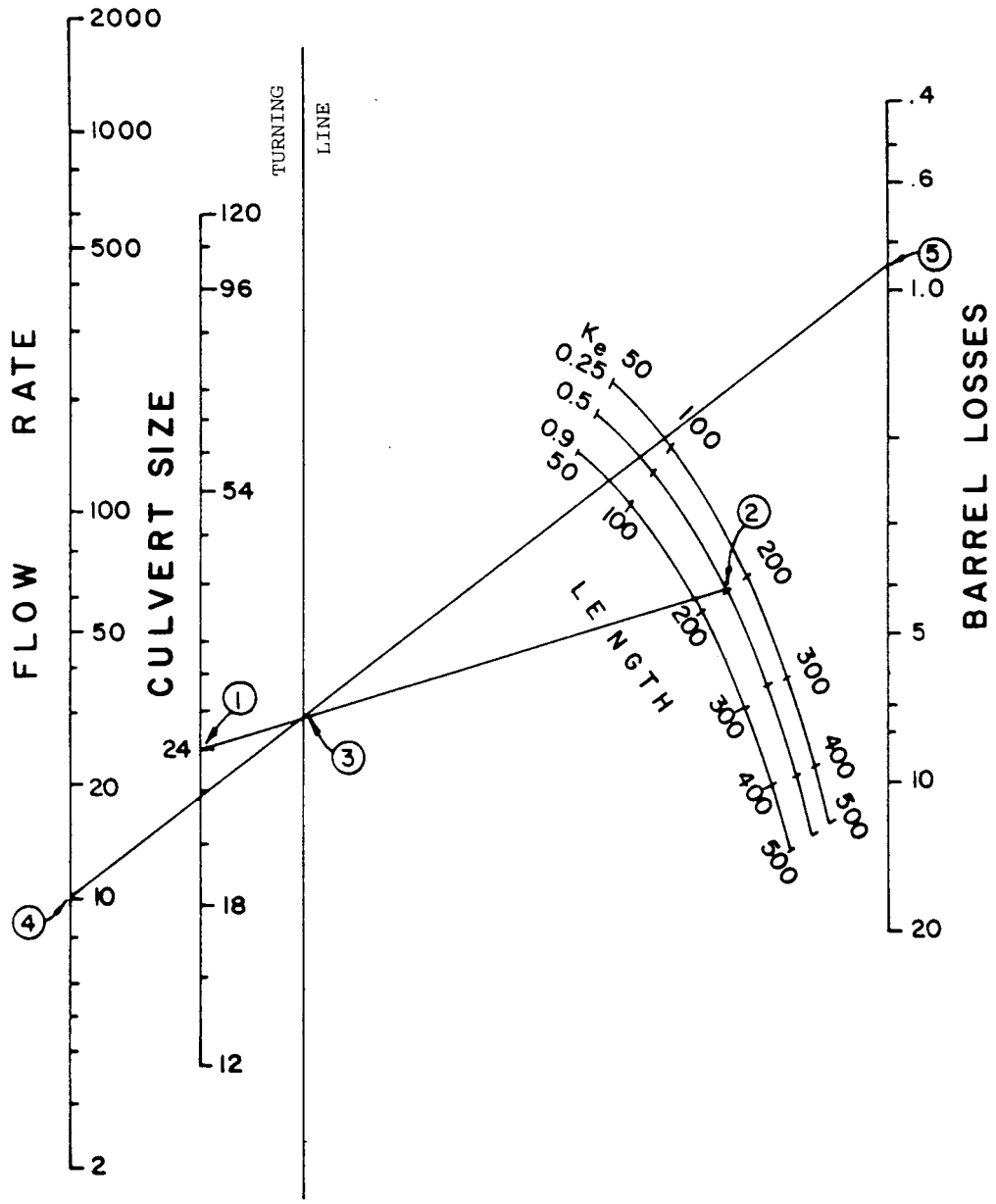
Evaluation of Results

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)



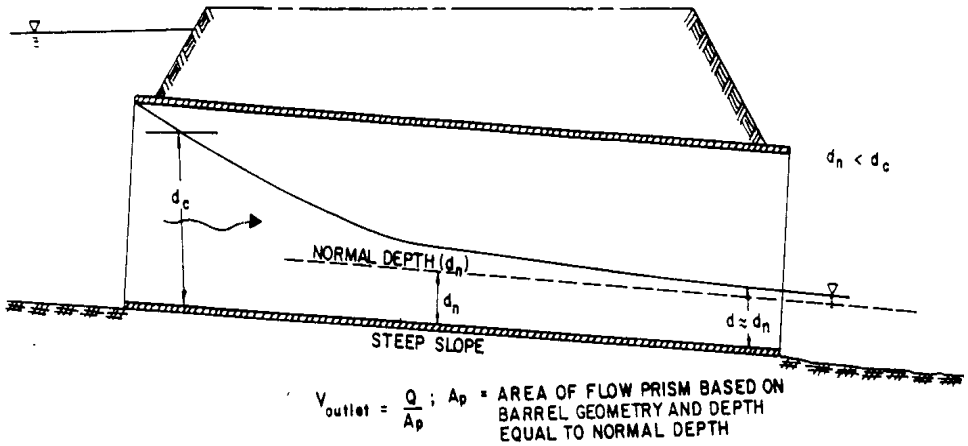
2. If the controlling headwater is in outlet control, determine the area of flow and velocity at the outlet based on the barrel geometry (see Figure 5.15) and the following:

   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 overtopping 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 stormdrain 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

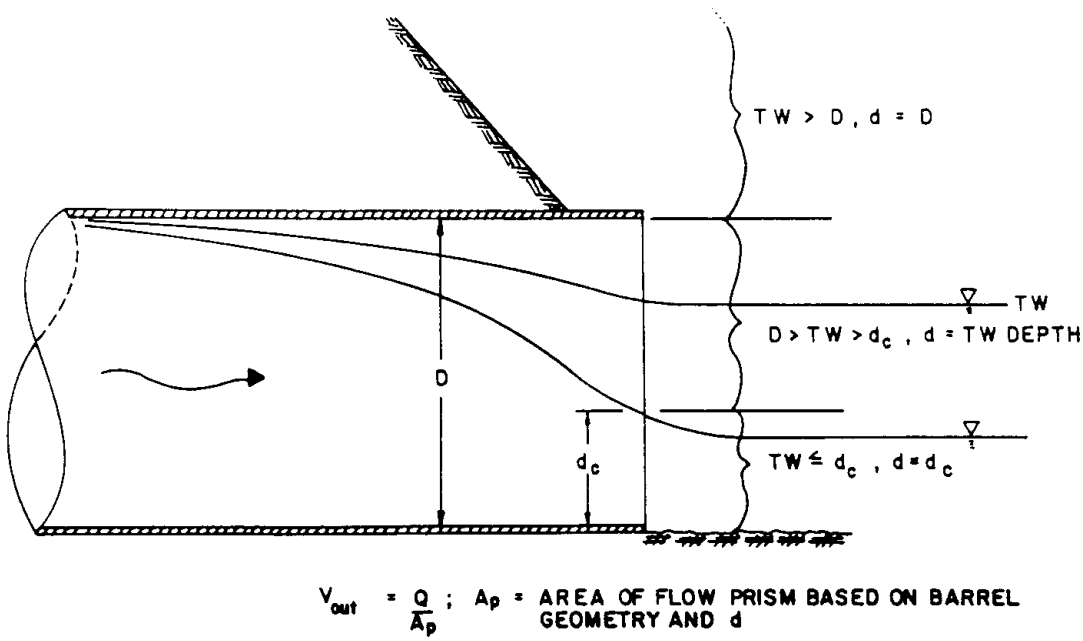
Performance curves are representations of flow rate versus headwater depth or stage for a culvert. Because a culvert has several possible control sections (inlet, outlet, throat), a given installation will have a performance curve for each control section and one for roadway overtopping. The overall 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.

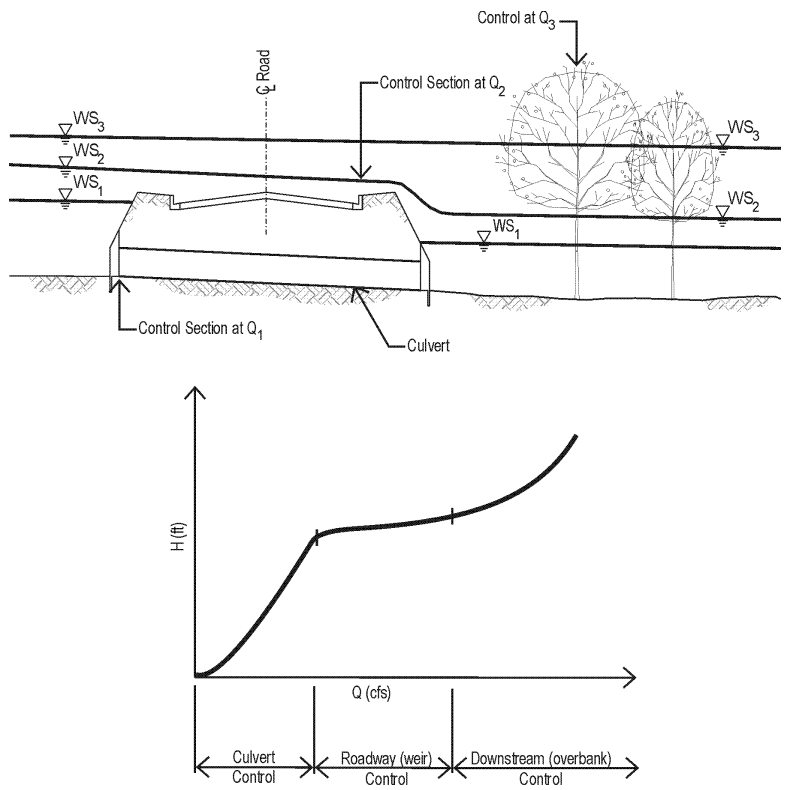
If backwater calculations are performed beginning at the downstream end of the culvert, friction losses are accounted for in the calculations. Adding the inlet loss to the energy grade line in the barrel at the inlet results in the headwater elevation for each flow rate. An example of development of a performance curve is contained in 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 (5.7b) 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, 1983). The U.S. Army Corps of Engineers HEC-2 (USACOE, 1990) and HEC-RAS computer programs (USACOE, 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 Figure 5.17 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:

$$Q_o = \frac{2K_t C_r L_x (HW_{rb}^{5/2} - HW_{ra}^{5/2})}{5(HW_{rb} - HW_{ra})} \tag{5.7b}$$

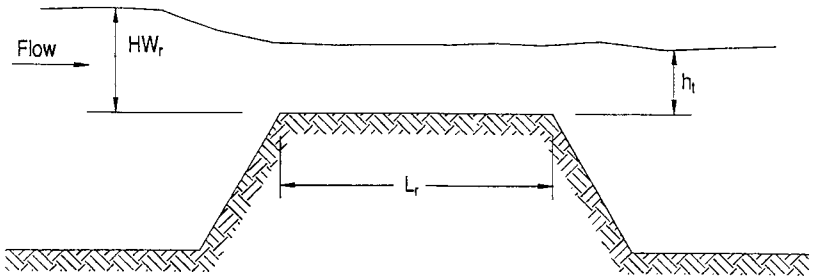
where: HW_{ra} = flow depth above the roadway at the high end of the weir segment, ft.

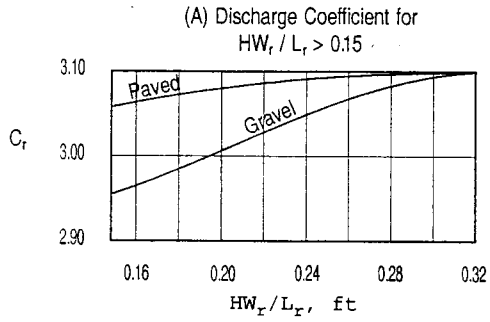
HW_{rb} = flow depth above the roadway at the low end of the weir segment, ft.

Adapted from Hulsing (1968).

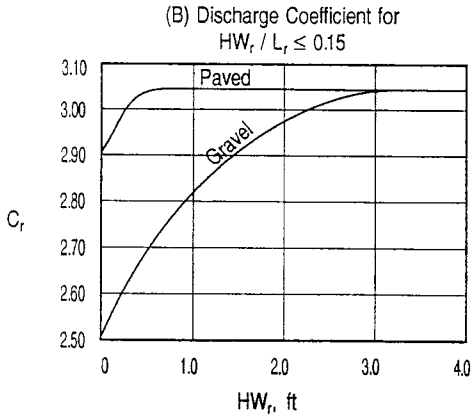
The total flow across the roadway then equals the sum of the roadway overflow plus the culvert flow. A performance curve must be plotted including both culvert flow and road overflow. The headwater depth for a specific discharge, such as the 100-year discharge can then be read from the curve. Design 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)





$C_d = K_t C_r$



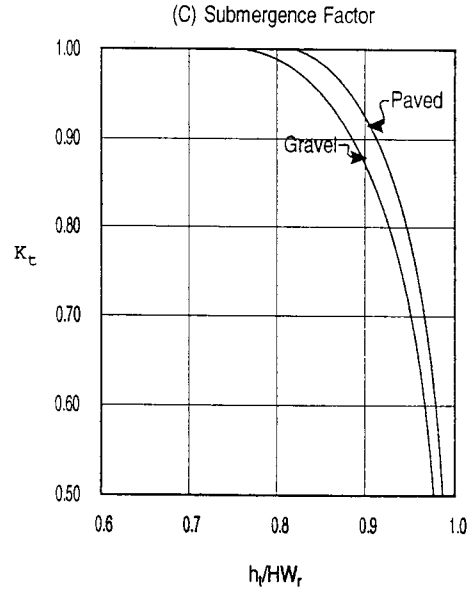
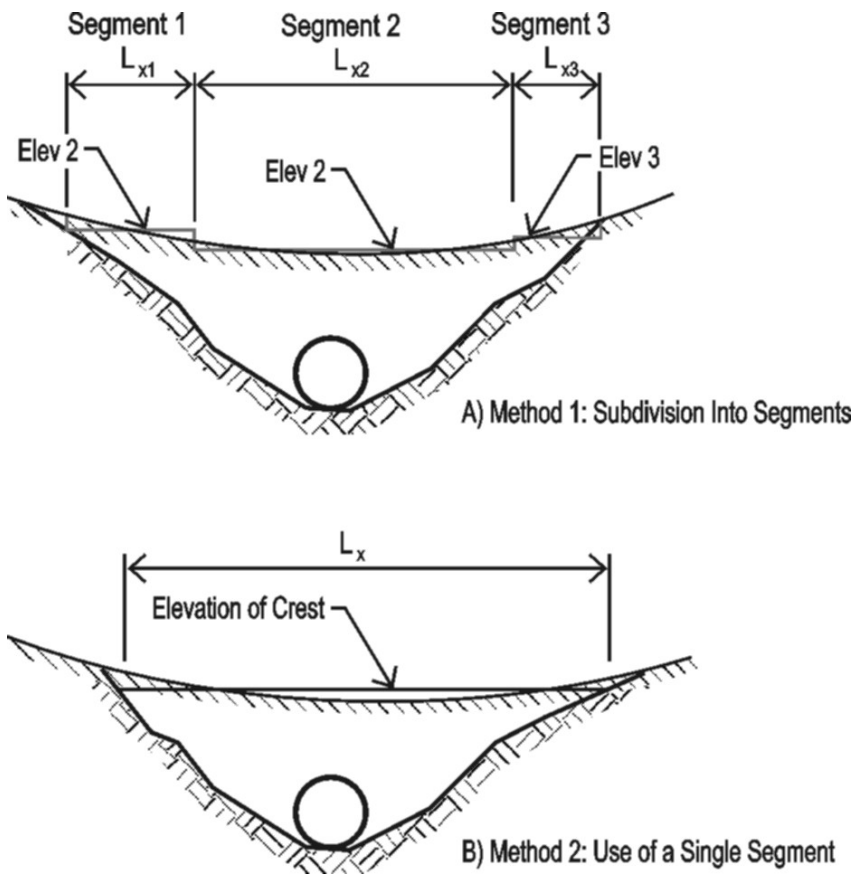


FIGURE 5.18
WEIR CREST LENGTH DETERMINATIONS FOR ROADWAY OVERTOPPING
(USDOT, FHWA, HDS-5, 1985)



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° 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° 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° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwalls at 10° to 25° 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)

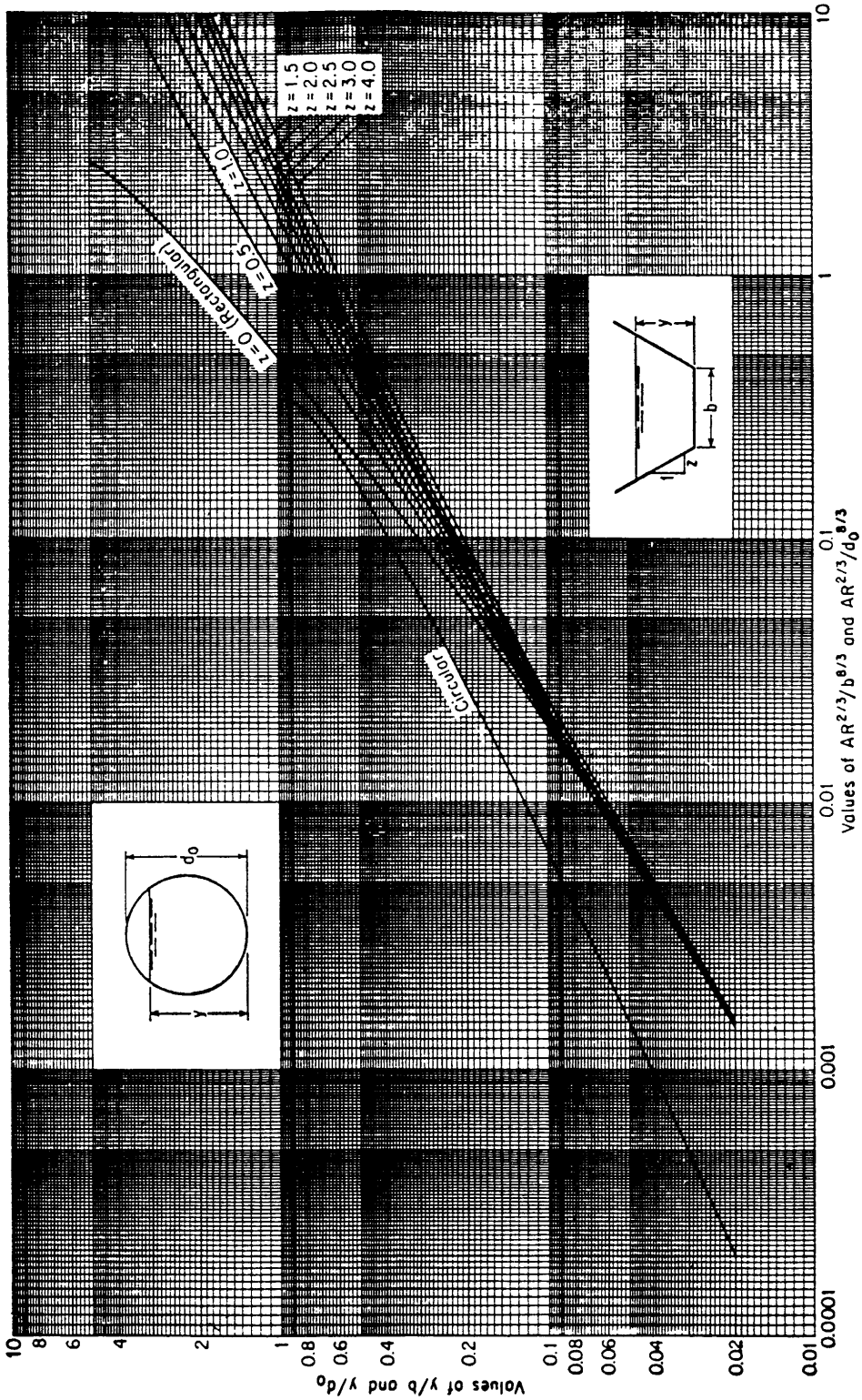


FIGURE 5.20
INLET CONTROL HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS
(USDOT, FHWA, HDS-5, 1985)

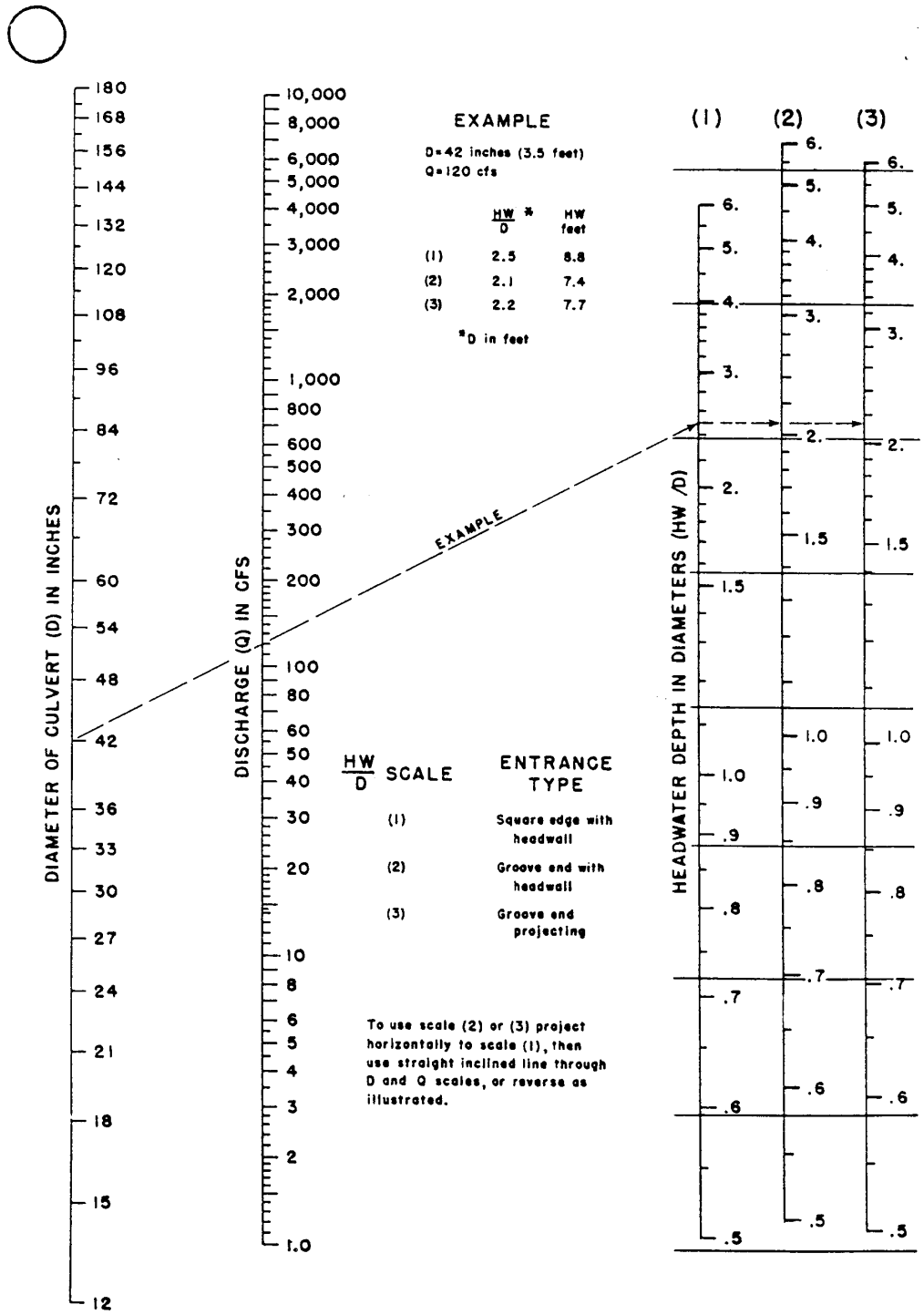


FIGURE 5.21
INLET CONTROL HEADWATER DEPTH FOR C.M. PIPE
(USDOT, FHWA, HDS-5, 1985)

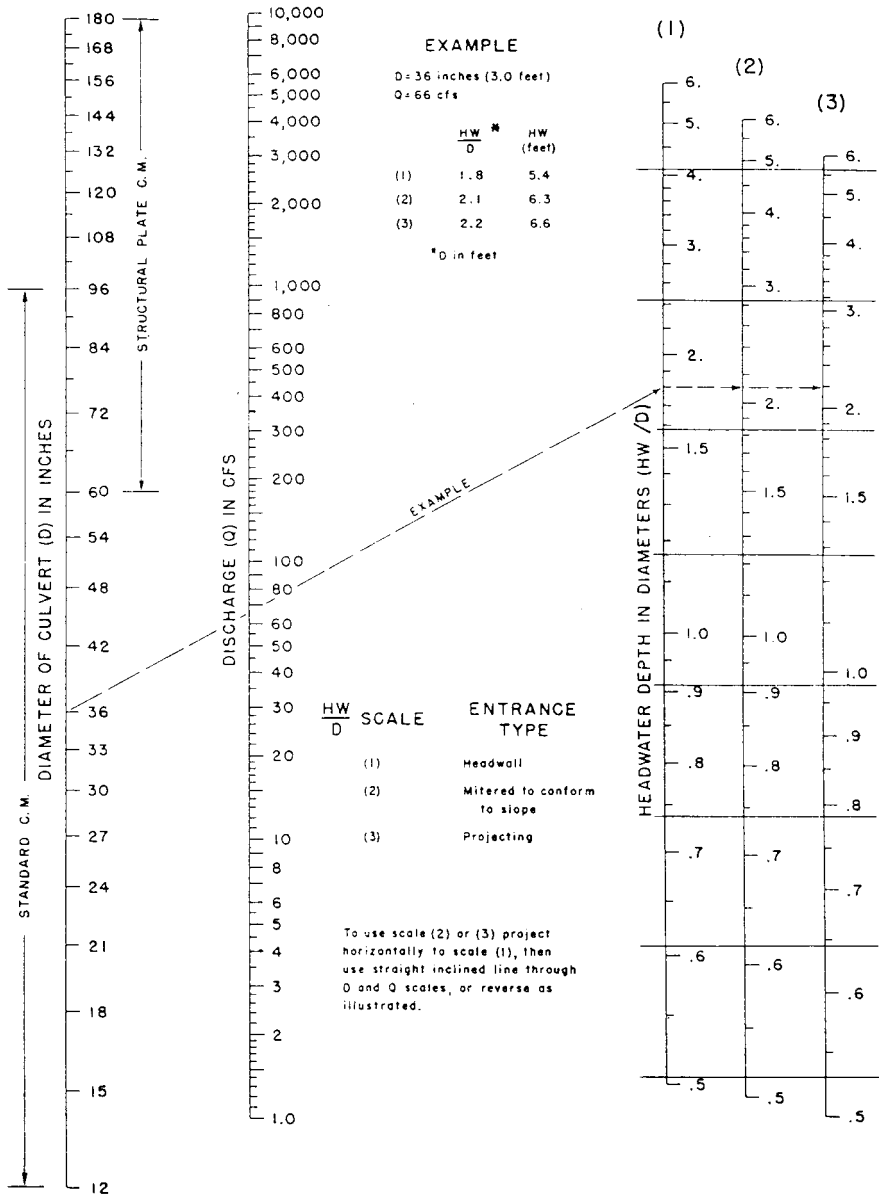


FIGURE 5.22
INLET CONTROL HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING
(USDOT, FHWA, HDS-5, 1985)

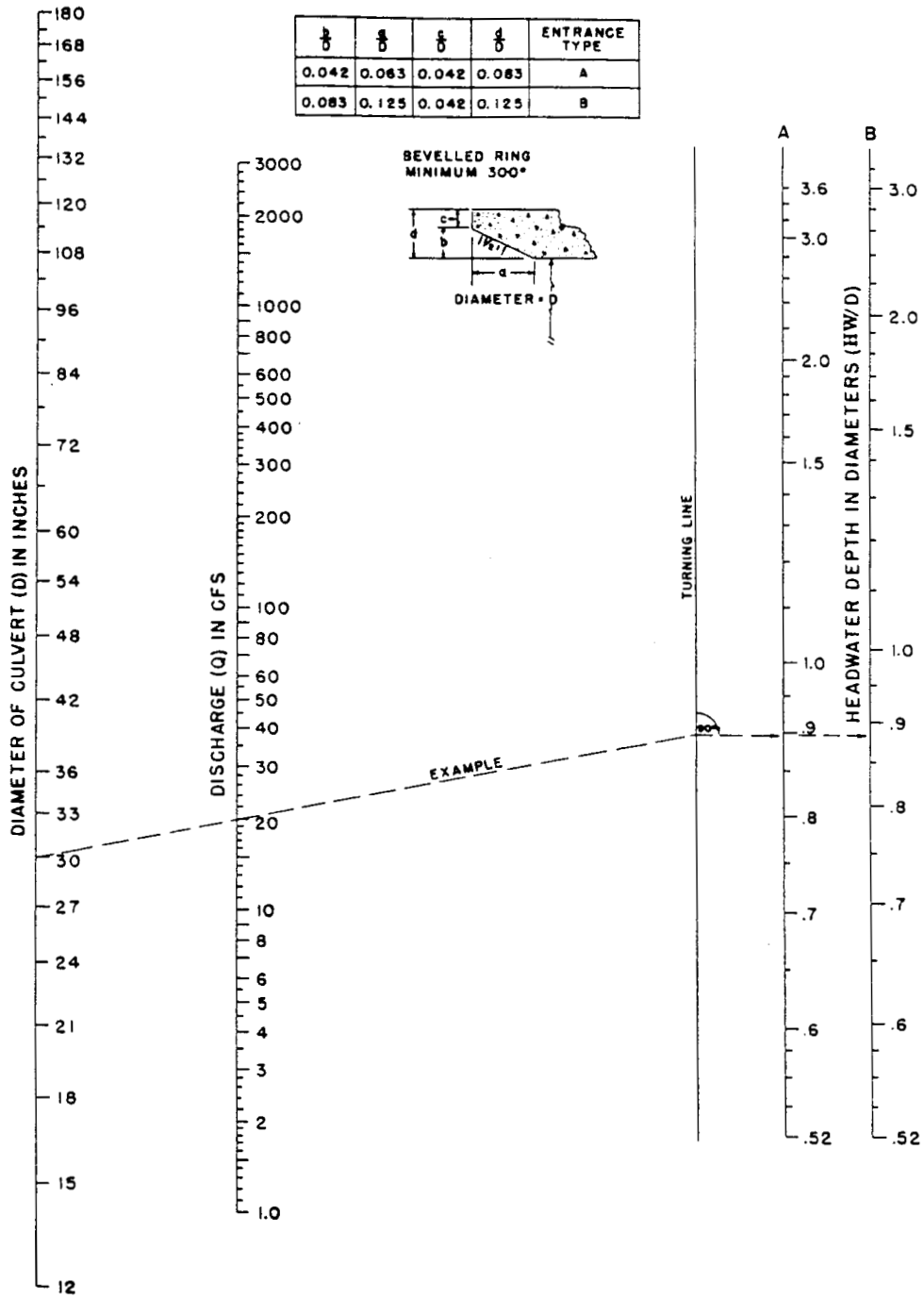


FIGURE 5.23
CRITICAL DEPTH FOR CIRCULAR PIPE
(USDOT, FHWA, HDS-5, 1985)

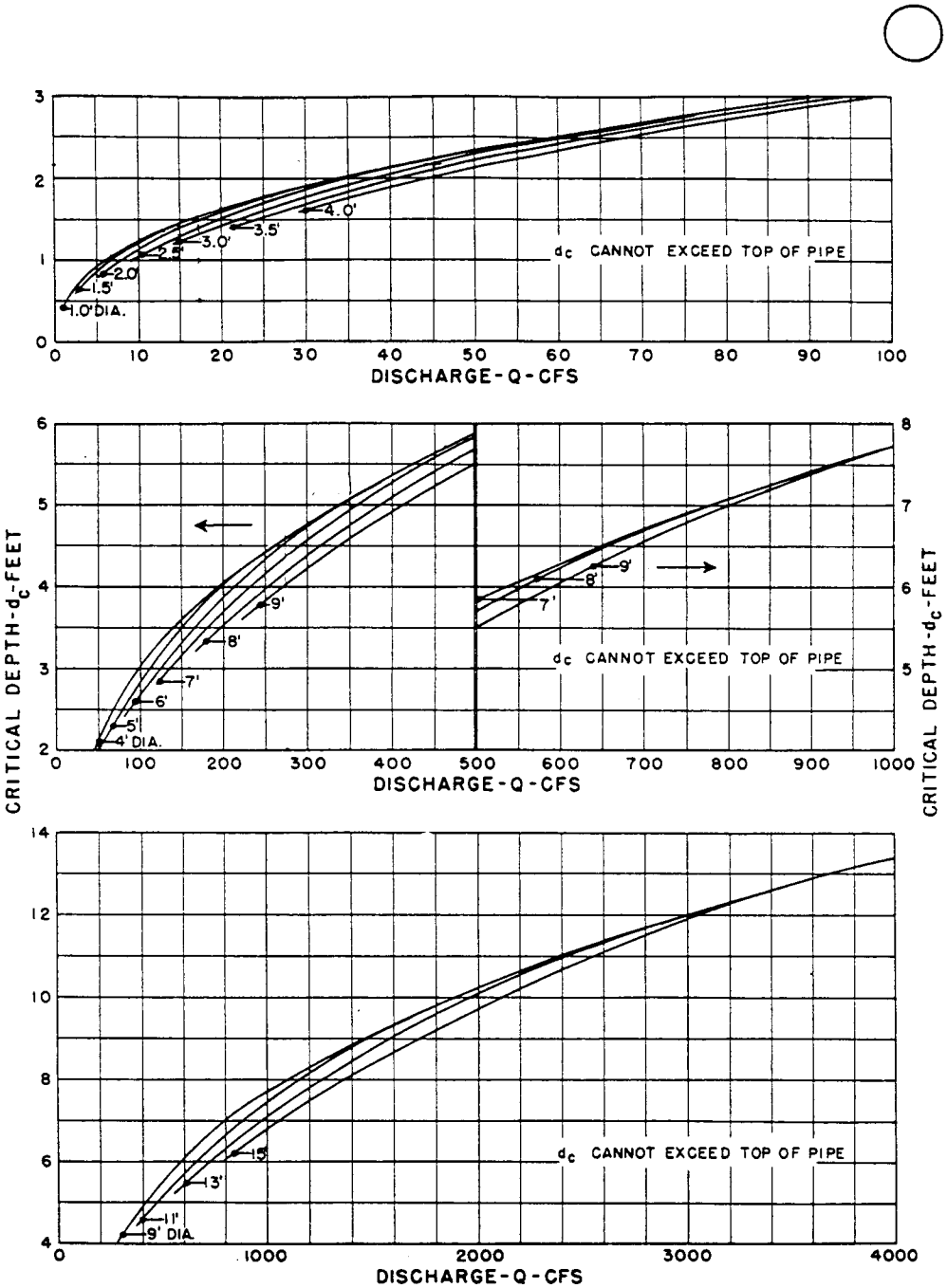


FIGURE 5.24
HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL
n = 0.012
(USDOT, FHWA, HDS-5, 1985)

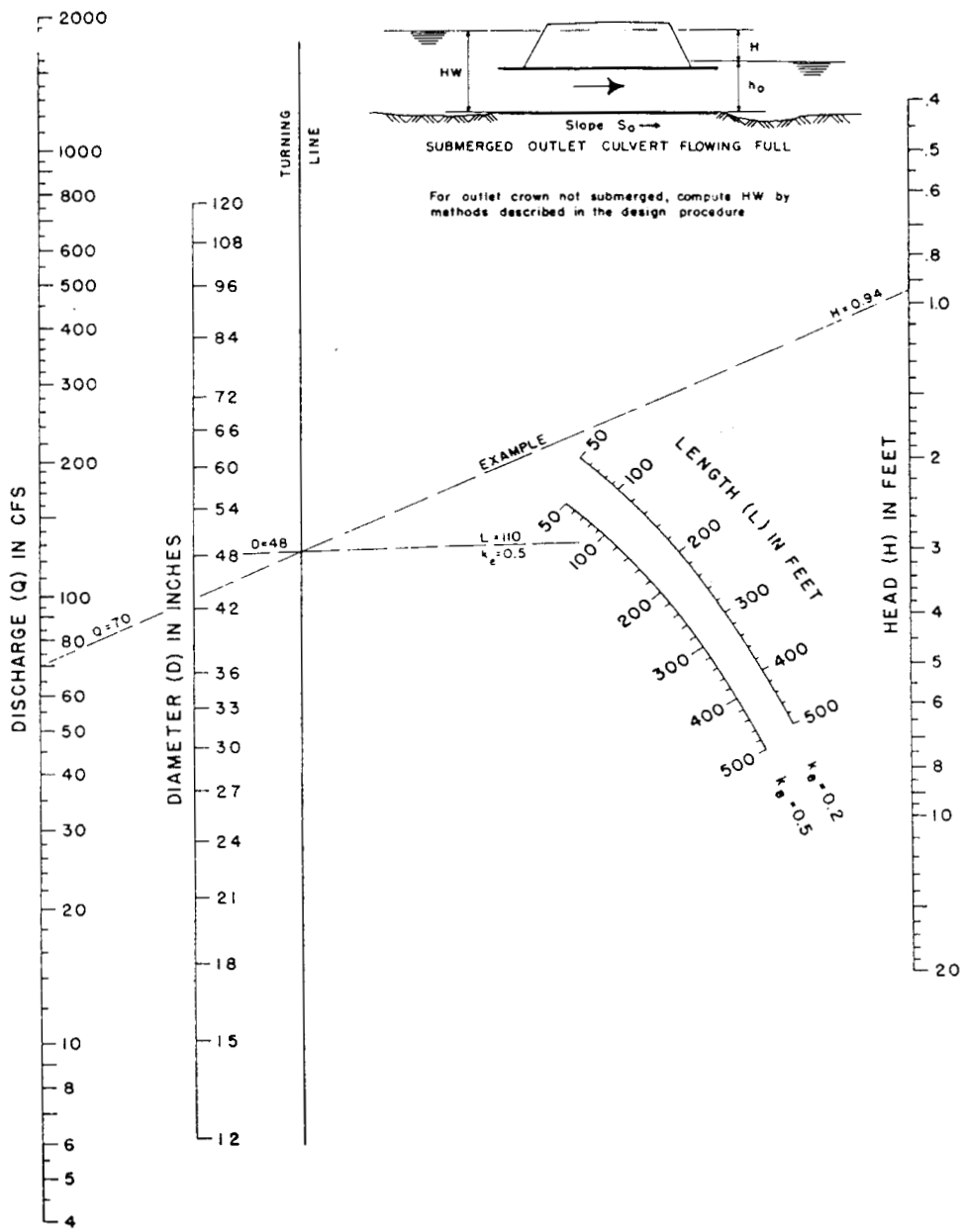


FIGURE 5.25
HEAD FOR C.M. PIPE CULVERTS FLOWING FULL
n = 0.024
(USDOT, FHWA, HDS-5, 1985)

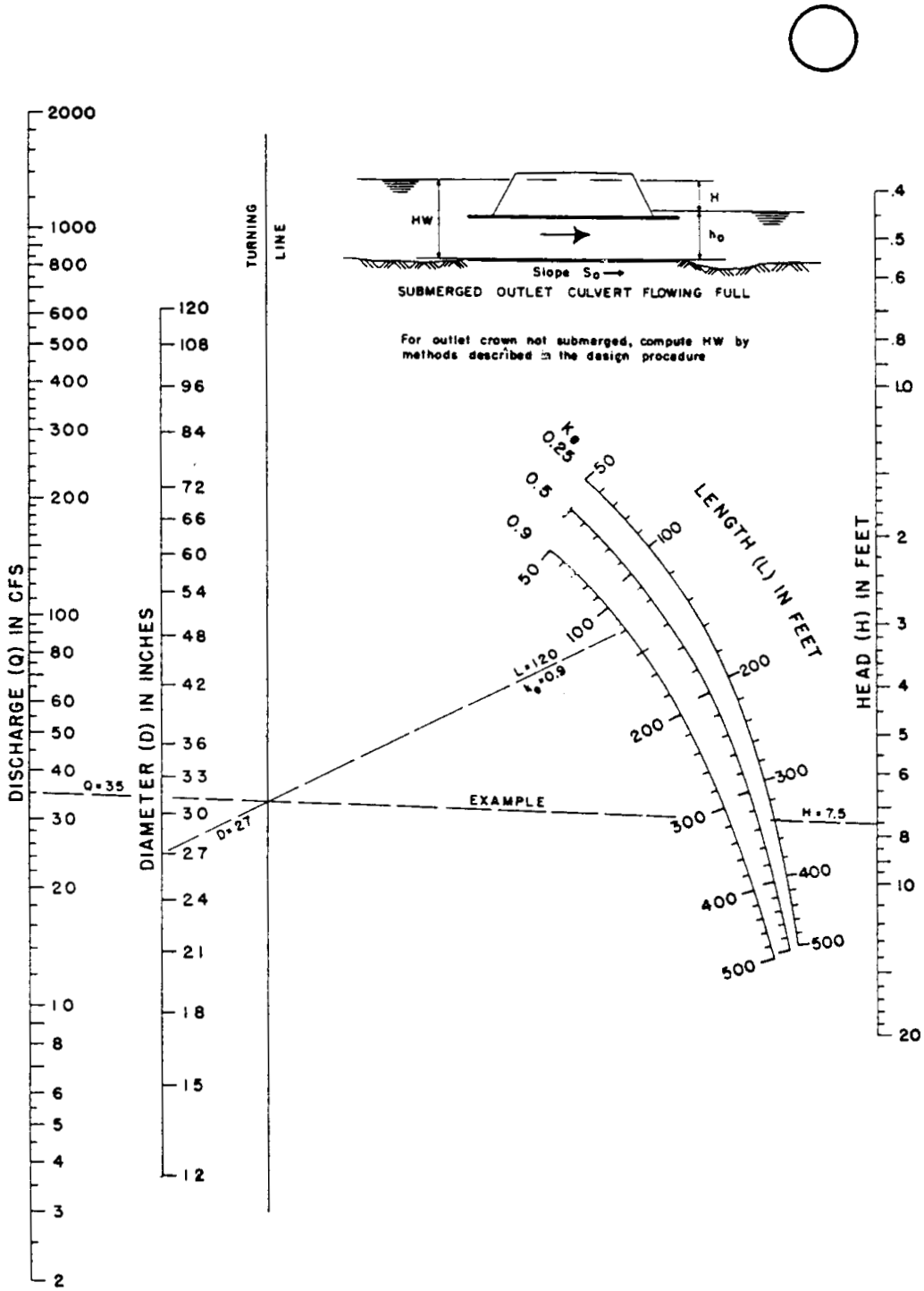


FIGURE 5.26
INLET CONTROL HEADWATER DEPTH FOR BOX CULVERTS
(USDOT, FHWA, HDS-5, 1985)

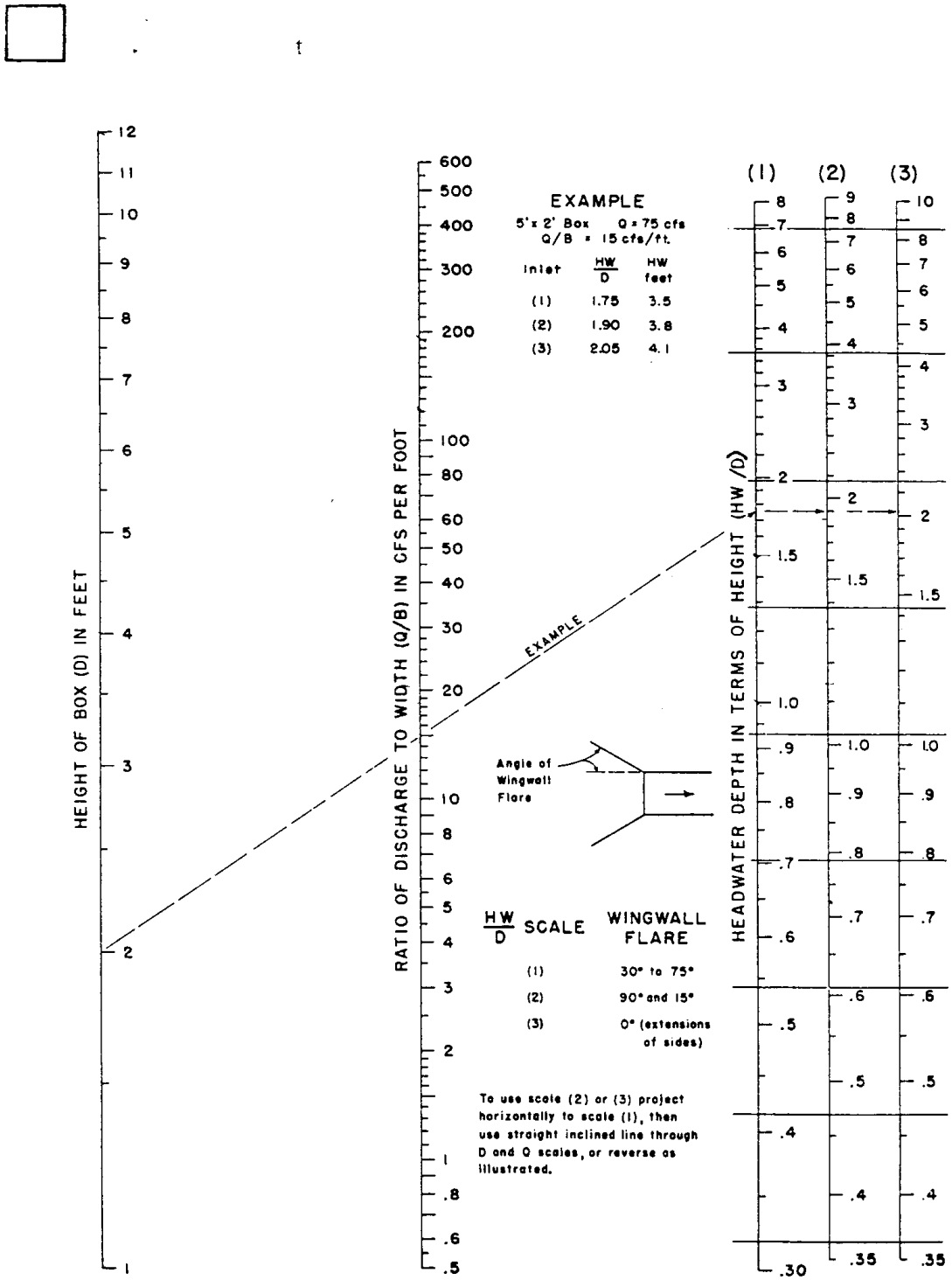


FIGURE 5.27

INLET CONTROL HEADWATER DEPTH FOR RECTANGULAR BOX CULVERT (FLARED WINGWALLS)

Flare Wingwalls (18° to 33.7°, and 45°) and Beveled Edge at the Top of the Inlet

(USDOT, FHWA, HDS-5, 1985)

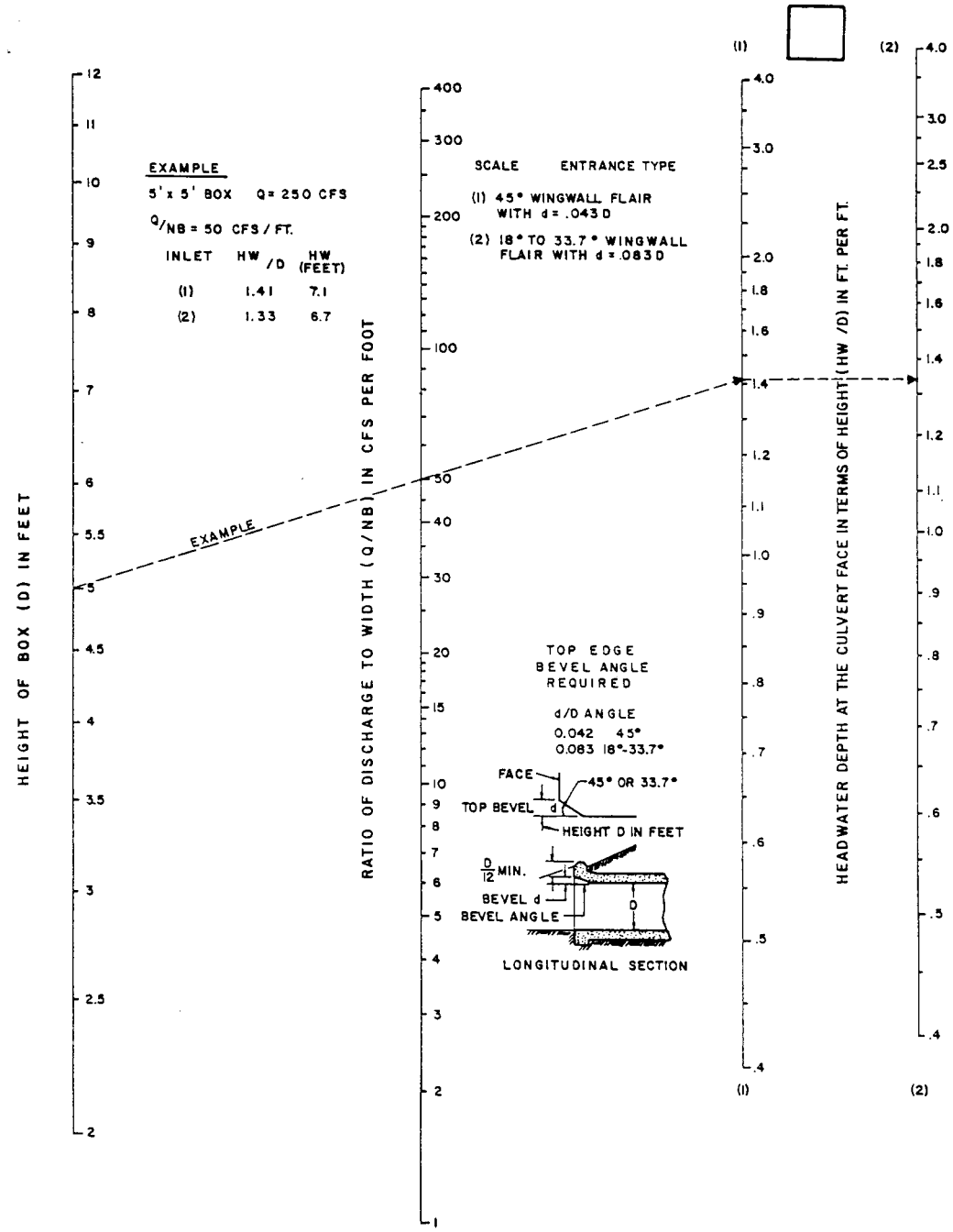


FIGURE 5.28

INLET CONTROL HEADWATER DEPTH FOR RECTANGULAR BOX CULVERT (90° HEADWALL)

90° Headwall - Chamfered or Beveled Inlet Edges

(USDOT, FHWA, HDS-5, 1985)

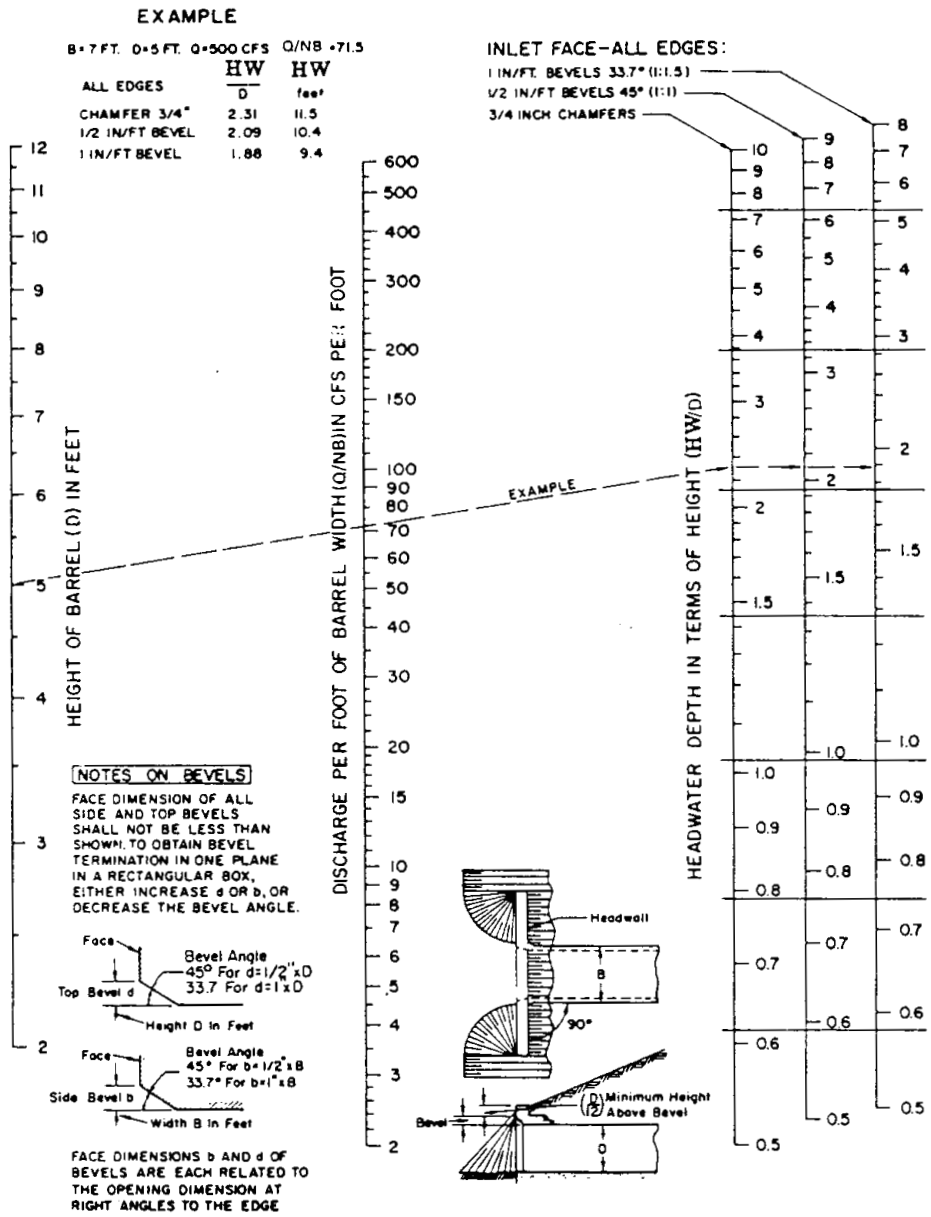


FIGURE 5.29
CRITICAL DEPTH RECTANGULAR SECTION
(USDOT, FHWA, HDS-5, 1985)

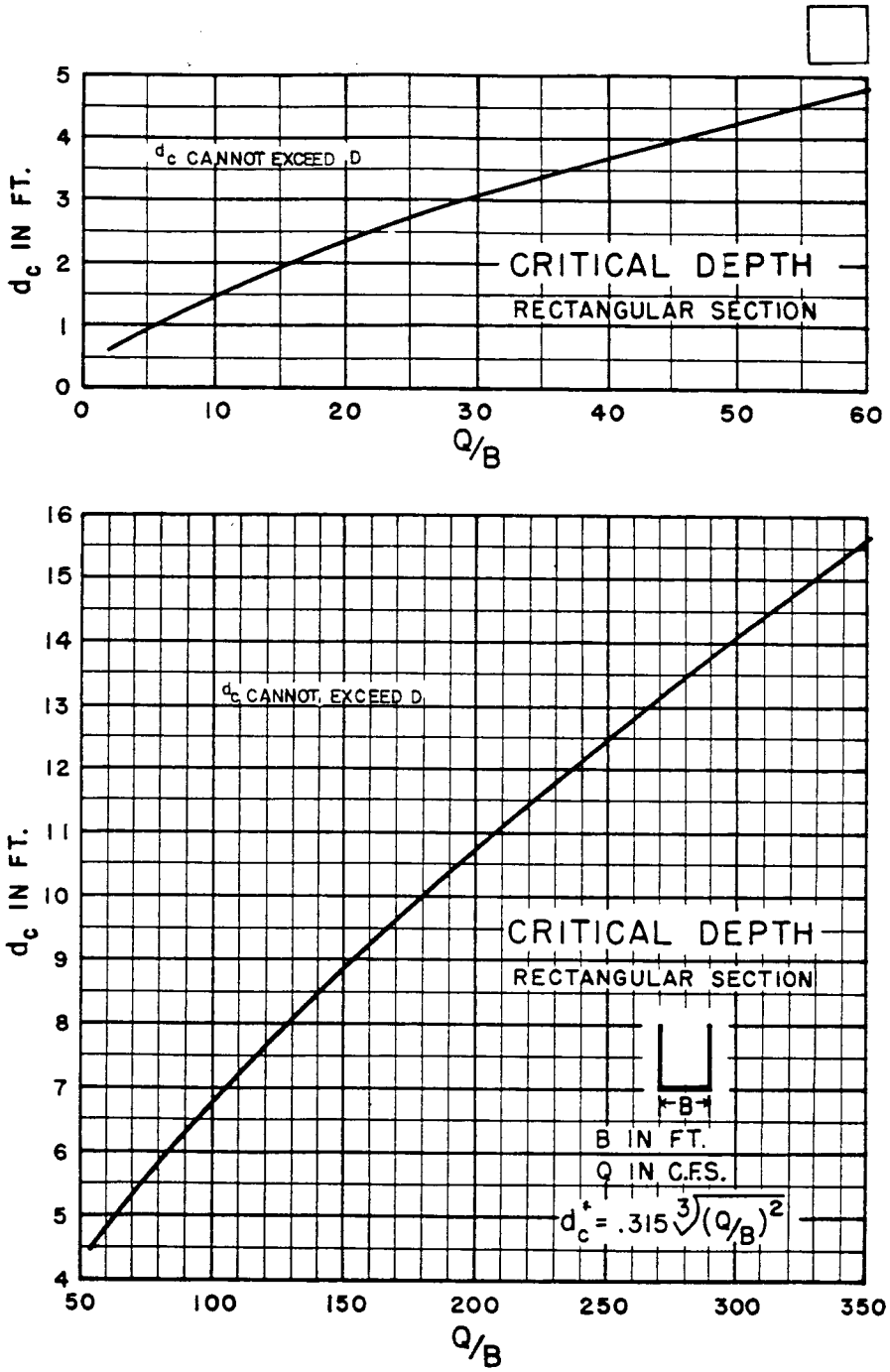


FIGURE 5.30
HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL
n = 0.012
(USDOT, FHWA, HDS-5, 1985)

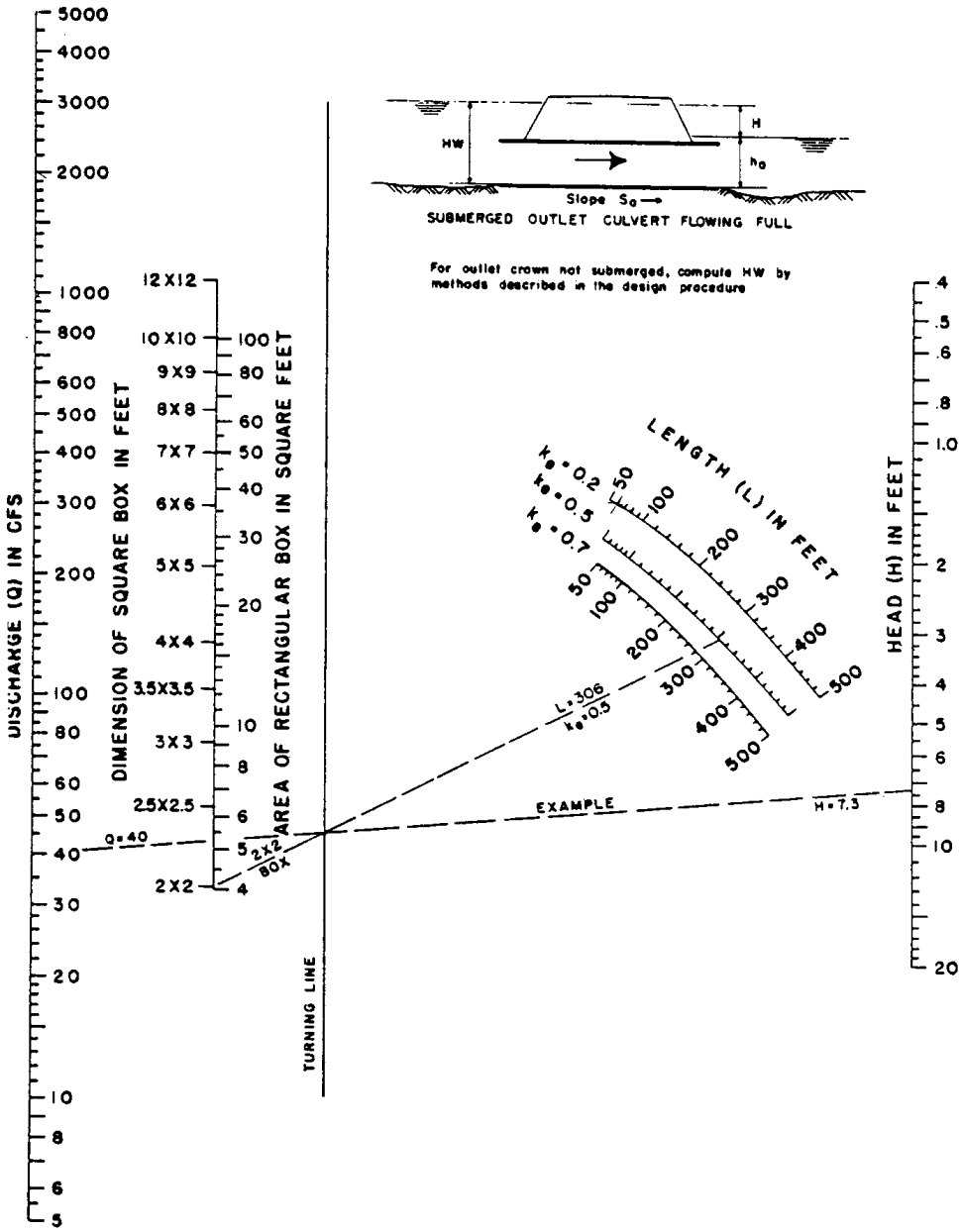


FIGURE 5.31
INLET CONTROL HEADWATER DEPTH FOR OVAL CONCRETE PIPE - LONG AXIS HORIZONTAL
(USDOT, FHWA, HDS-5, 1985)

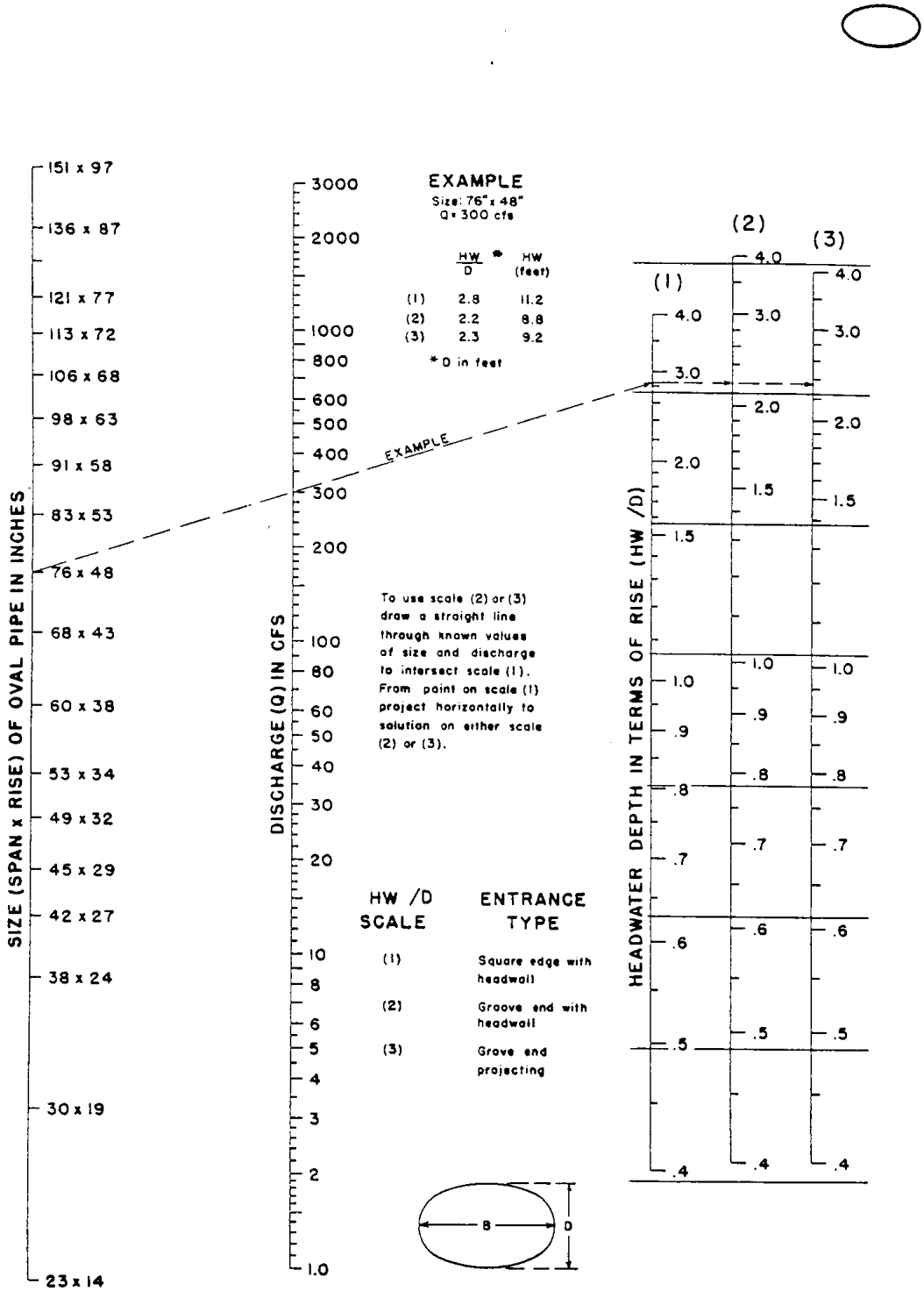


FIGURE 5.32
INLET CONTROL HEADWATER DEPTH FOR OVAL CONCRETE PIPE - LONG AXIS VERTICAL
(USDOT, FHWA, HDS-5, 1985)

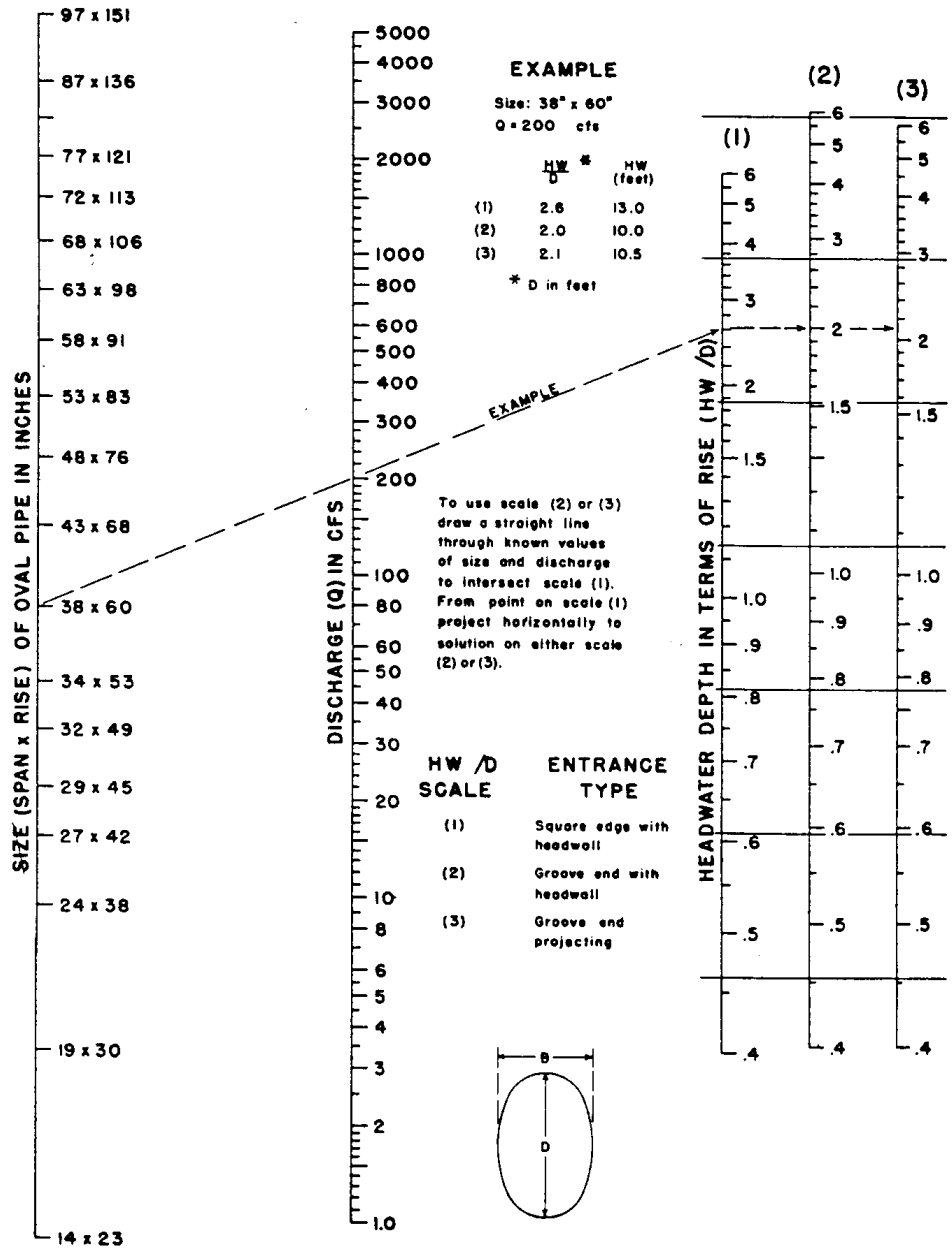


FIGURE 5.33
CRITICAL DEPTH FOR AN OVAL CONCRETE PIPE - LONG AXIS HORIZONTAL
(USDOT, FHWA, HDS-5, 1985)

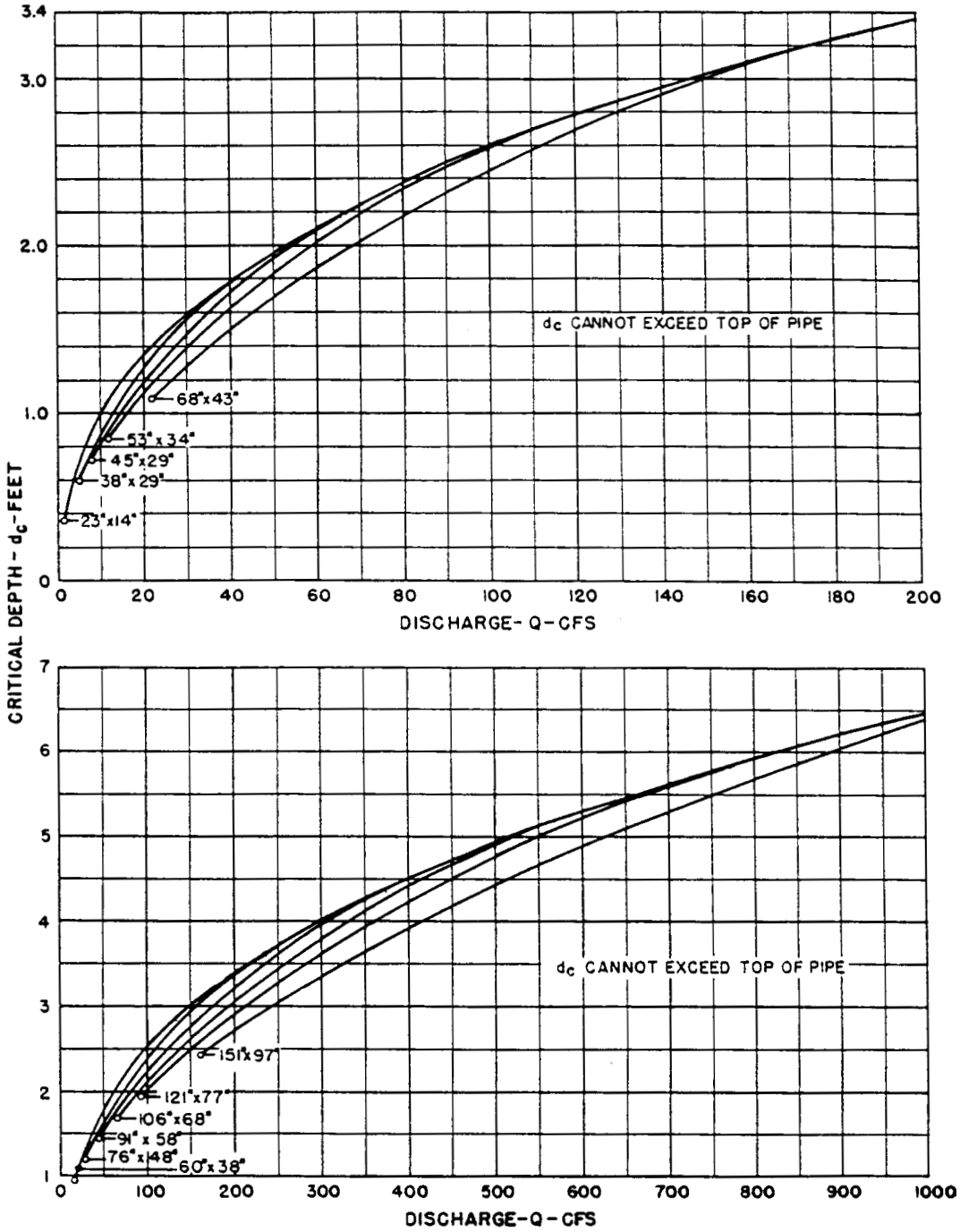


FIGURE 5.34
CRITICAL DEPTH FOR AN OVAL CONCRETE PIPE - LONG AXIS VERTICAL
(USDOT, FHWA, HDS-5, 1985)

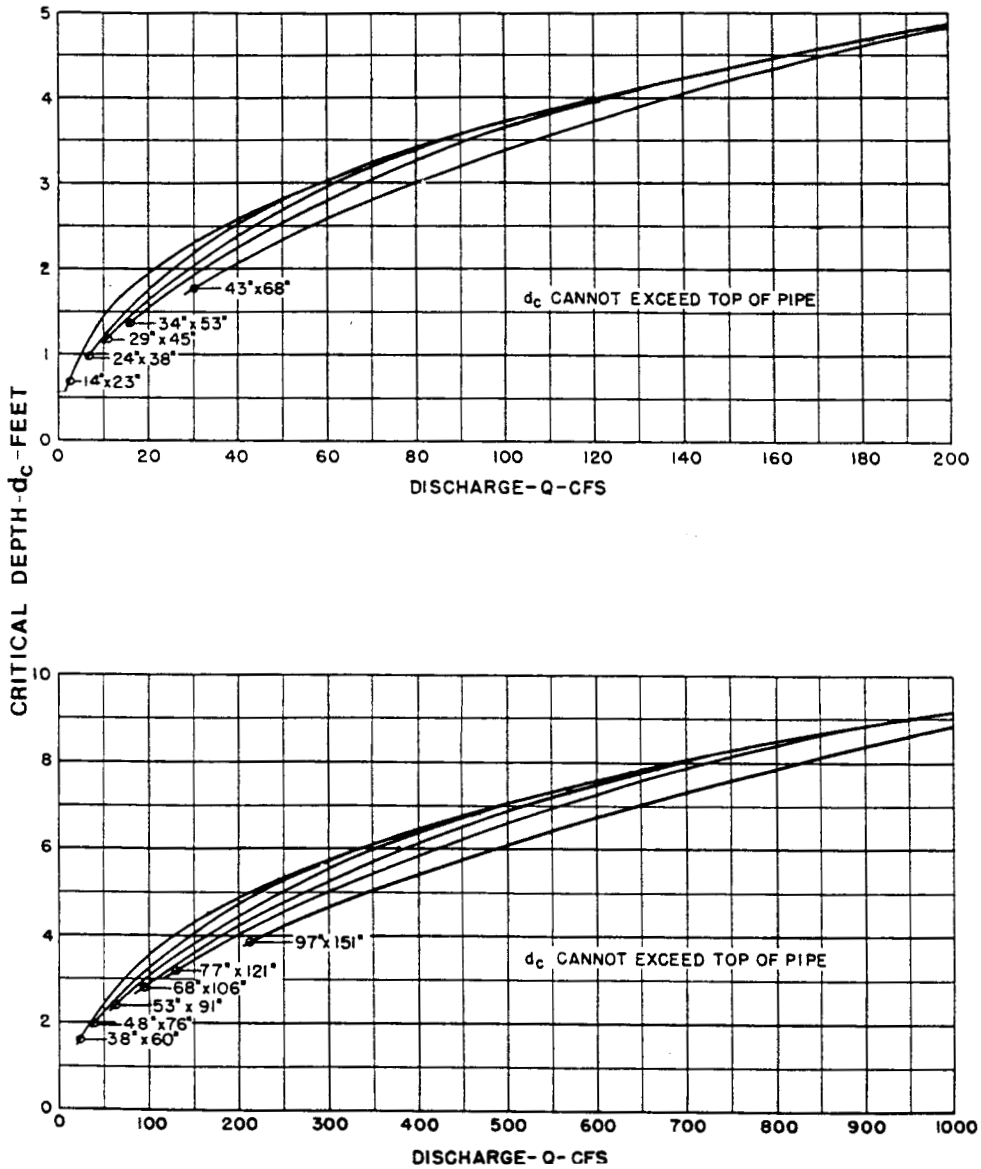


FIGURE 5.35
HEAD FOR CONCRETE PIPE FLOWING FULL - LONG AXIS HORIZONTAL OR VERTICAL
n = 0.012
(USDOT, FHWA, HDS-5, 1985)

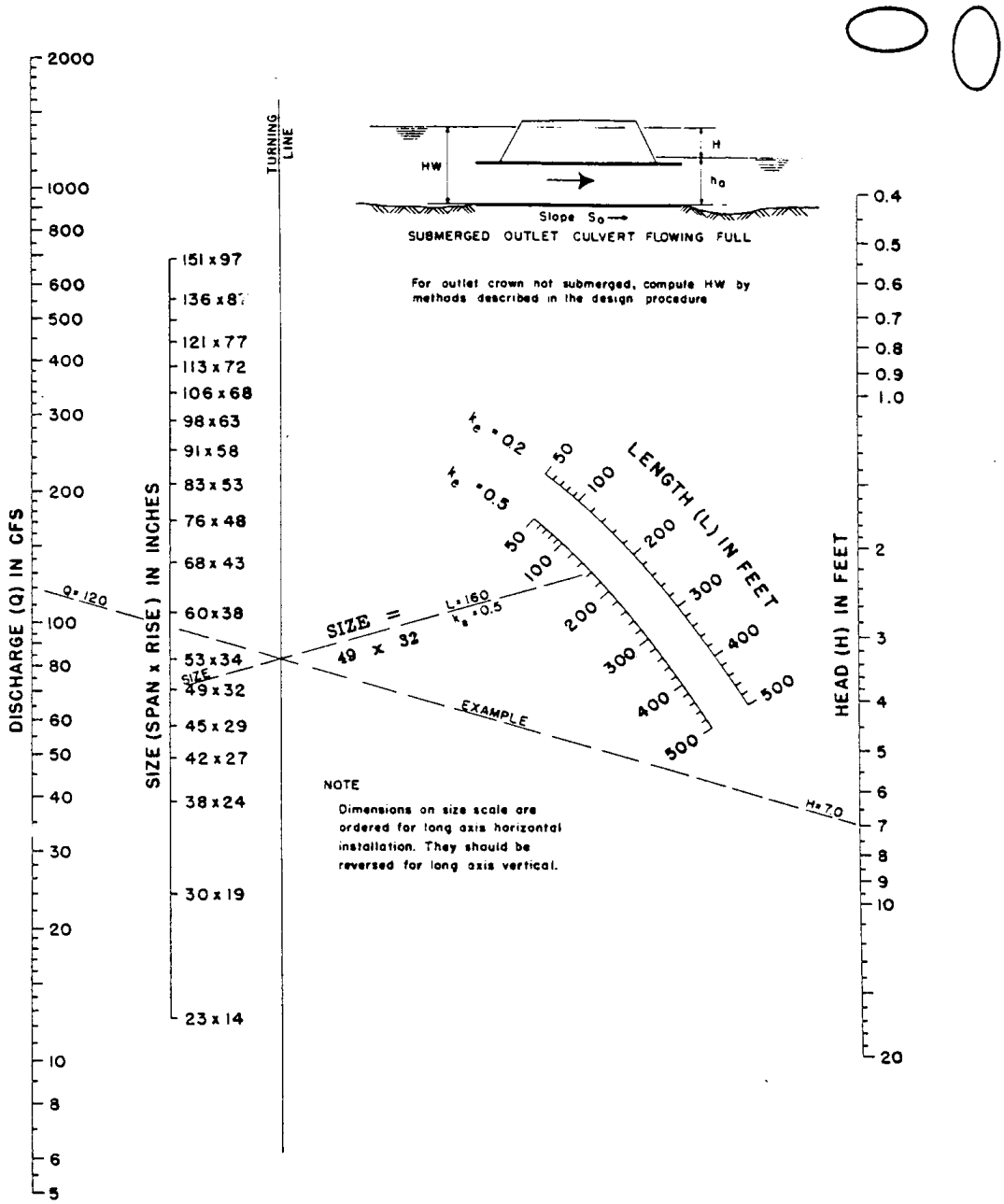


FIGURE 5.36
HEADWATER DEPTH FOR C.M. PIPE - ARCH CULVERT WITH INLET CONTROL
(USDOT, FHWA, HDS-5, 1985)

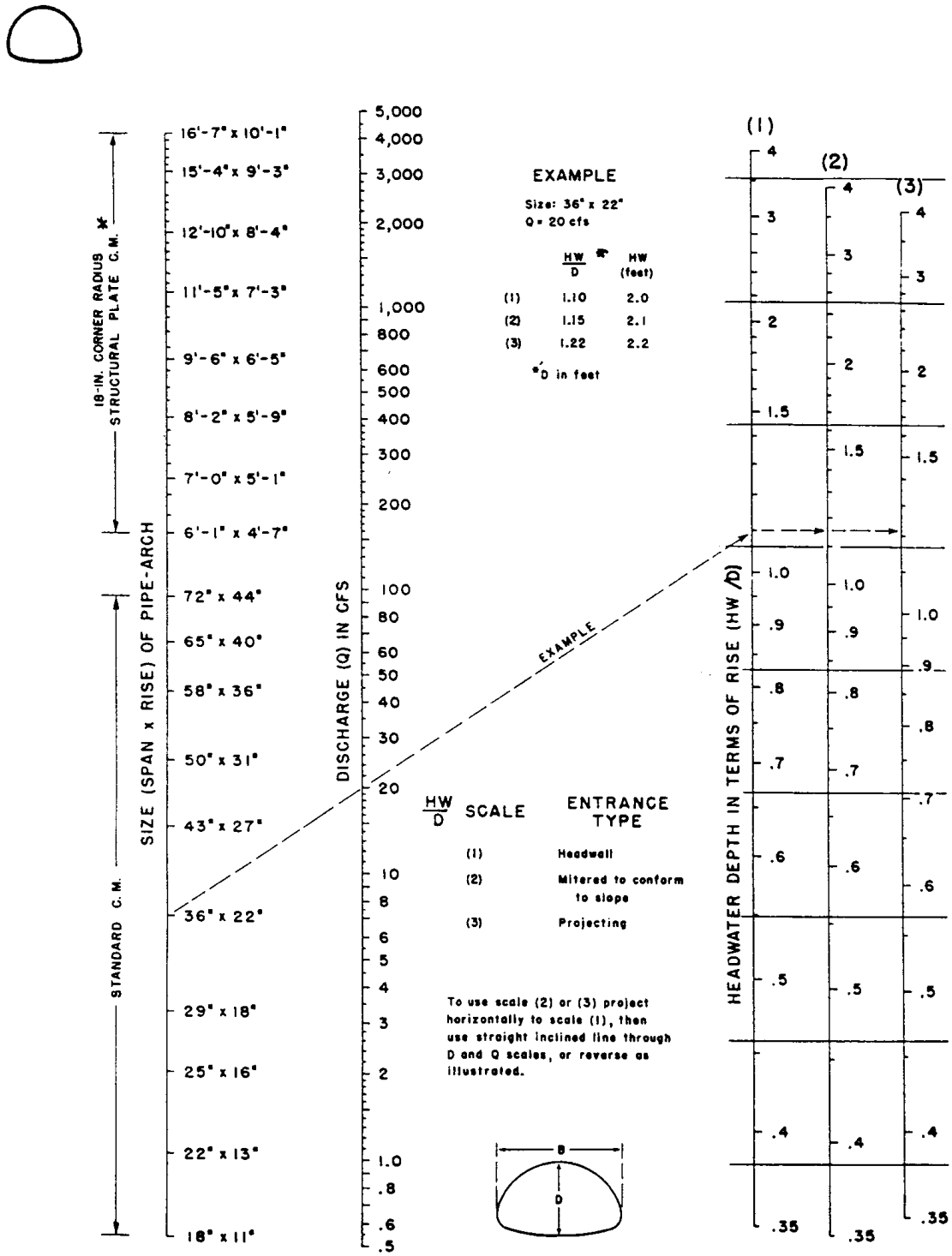
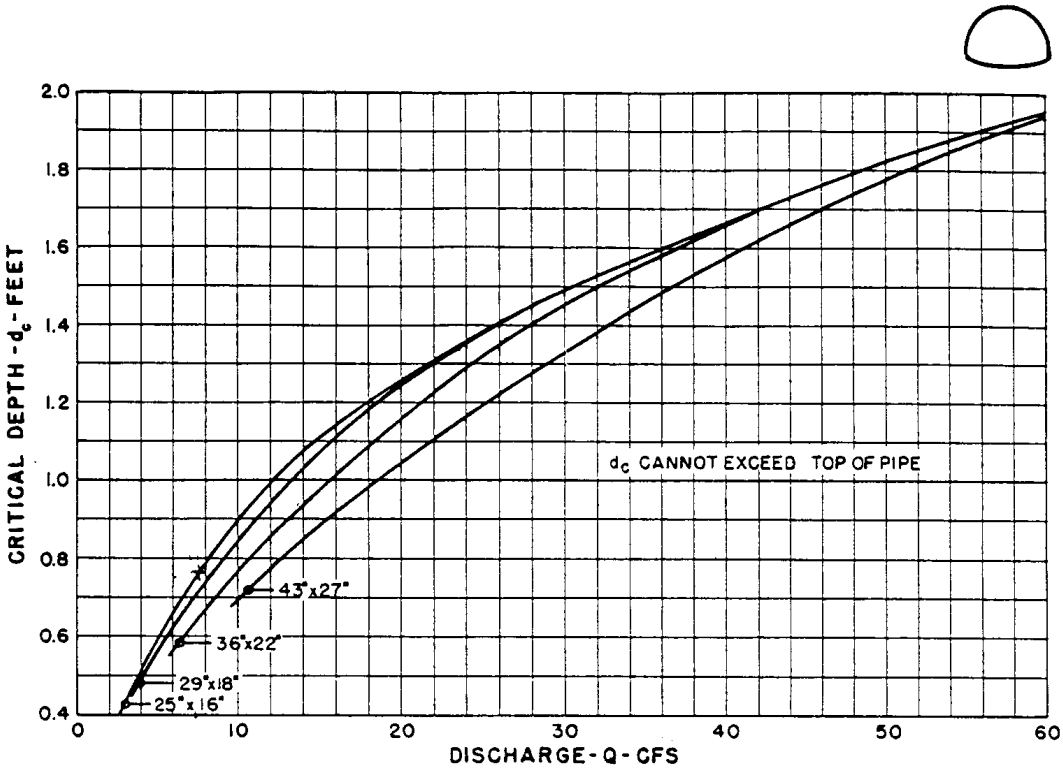


FIGURE 5.37
CRITICAL DEPTH FOR STANDARD C.M. PIPE - ARCH
(USDOT, FHWA, HDS-5, 1985)



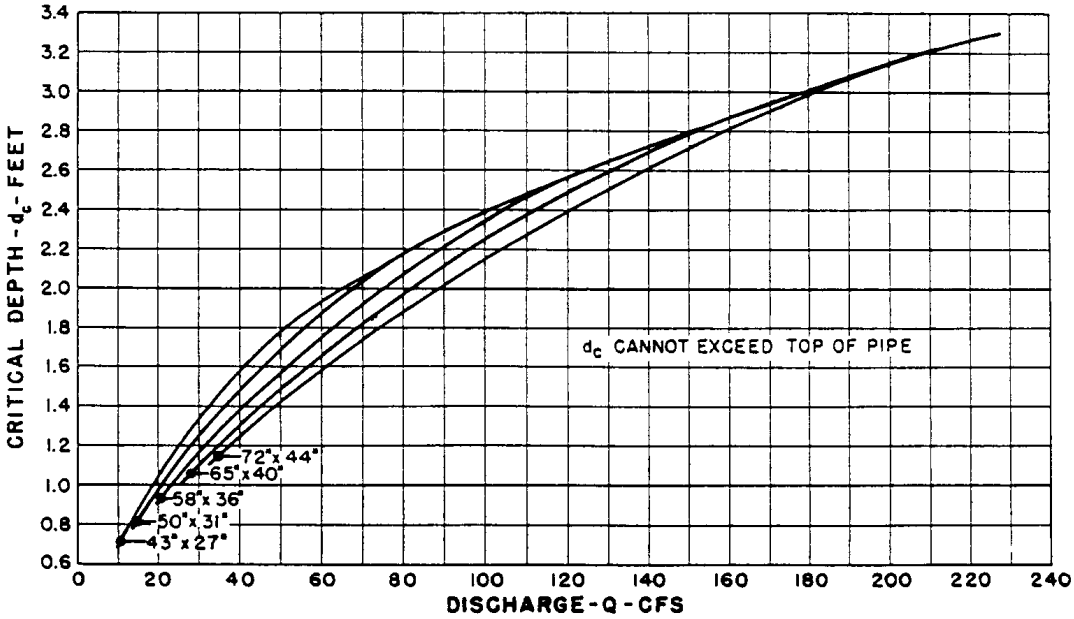
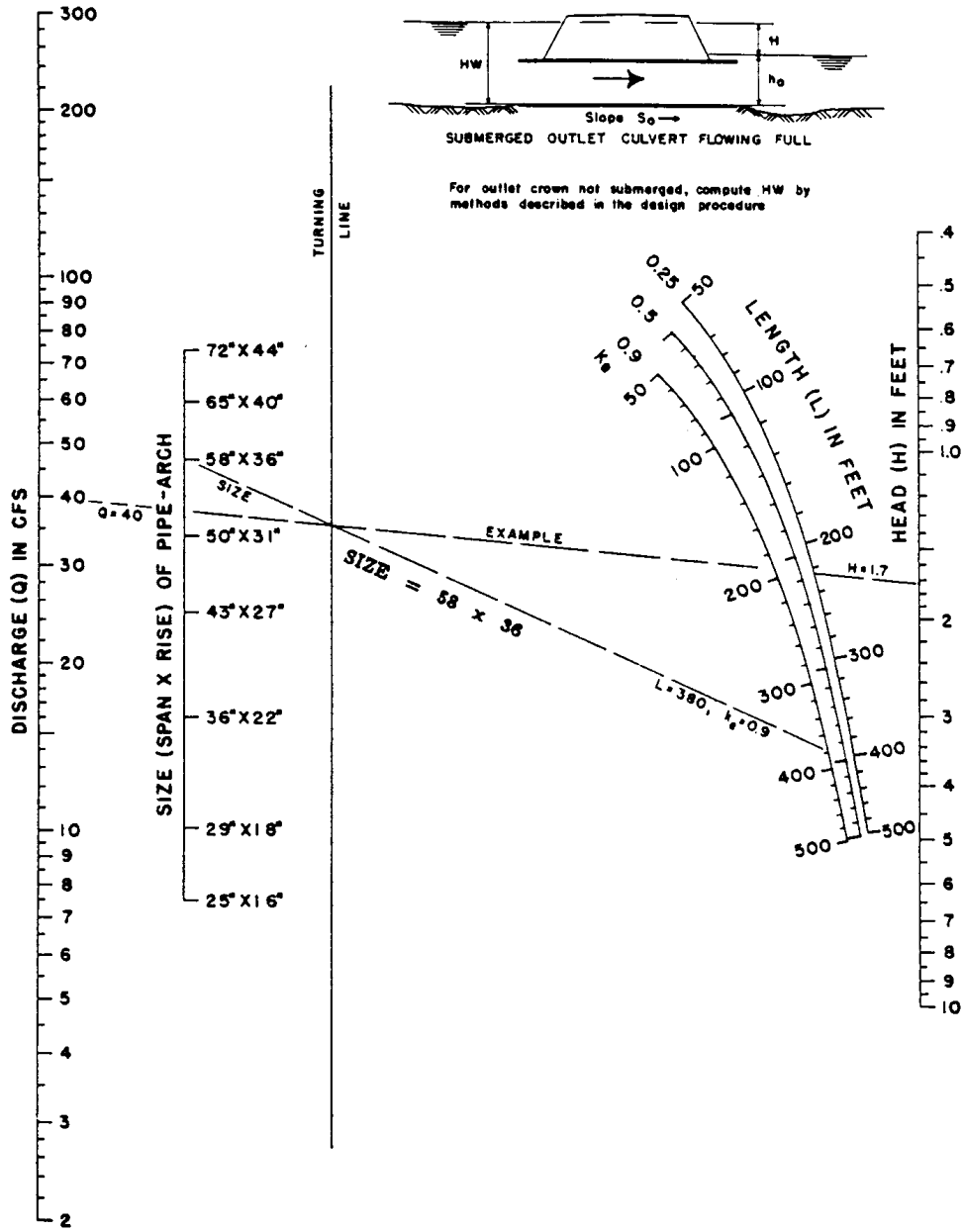


FIGURE 5.38
HEAD FOR STANDARD C.M. PIPE - ARCH CULVERTS FLOWING FULL
n = 0.024
(USDOT, FHWA, HDS-5, 1985)



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 or Section 7.4, 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

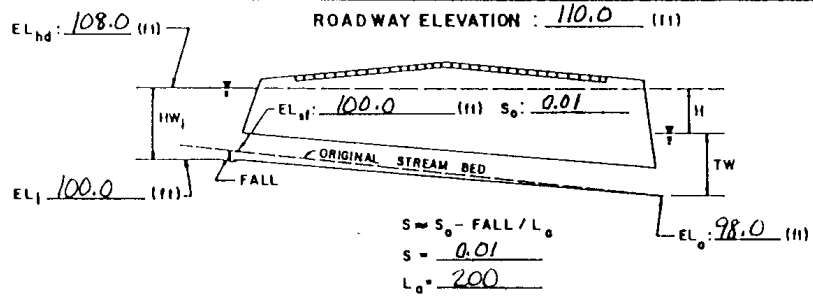
CULVERT DESIGN FORM
DESIGNER / DATE: WJJ / 7/18
REVIEWER / DATE: JMN / 7/19

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)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
CMP - CIRC. - 72 IN. - BEVEL 45° IN HEADWALL	200	200	0.96	58	-	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 = $HW_i - (EL_{hd} - EL_{ef})$; 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
- ef. 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° 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 or Section 7.4, 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)

CULVERT DESIGN FORM

PROJECT: EXAMPLE PROBLEM No. 2
CHAPTER 3, HDS No. 5

STATION: 1+00
SHEET 1 OF 1

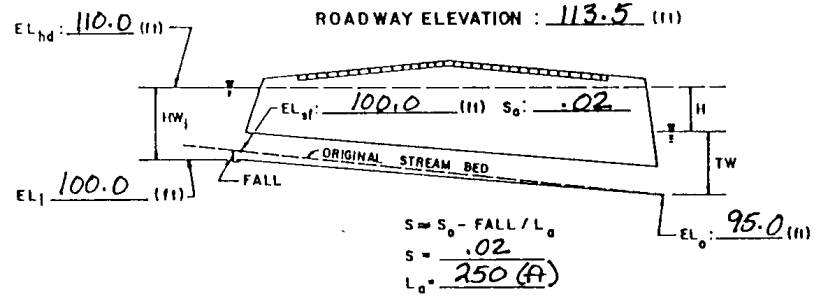
DESIGNER / DATE: WJT / 7/18
REVIEWER / DATE: JMN / 7/19

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)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
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 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

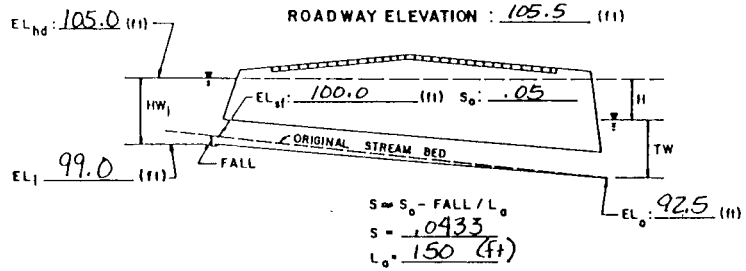
HYDROLOGICAL DATA

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)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_hi (4)	OUTLET CONTROL TW (5)	d_c	(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

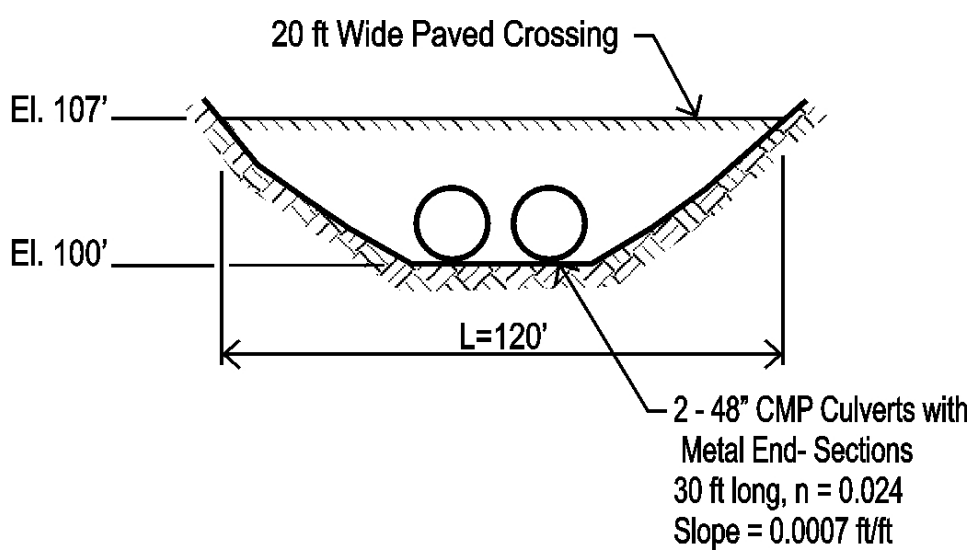


FIGURE 5.43
EXAMPLE 4 CULVERT DESIGN FORM

PROJECT: Example
Roadway Overtopping – Performance Curve

STATION: _____
SHEET 1 OF _____

CULVERT DESIGN FORM
DESIGNER / DATE: EJF / 3-3-
REVIEWER / DATE: _____ / _____

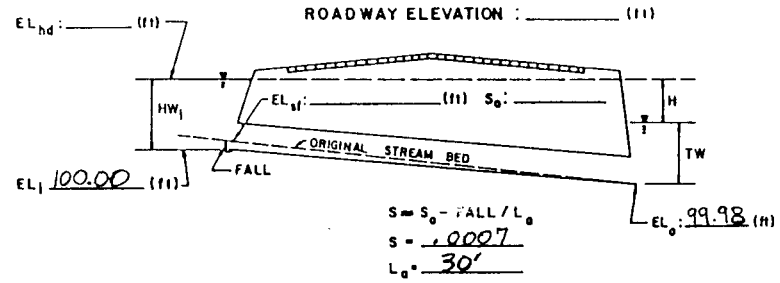
HYDROLOGICAL DATA

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)



$S = S_o - FALL / L_o$
$S = .0007$
$L_o = 30'$

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_hi (4)	OUTLET CONTROL TW (5)	d_c	(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) $H = \left[1 + k_e + (29n^2 L)/R^{1.33}\right] V^2/2g$

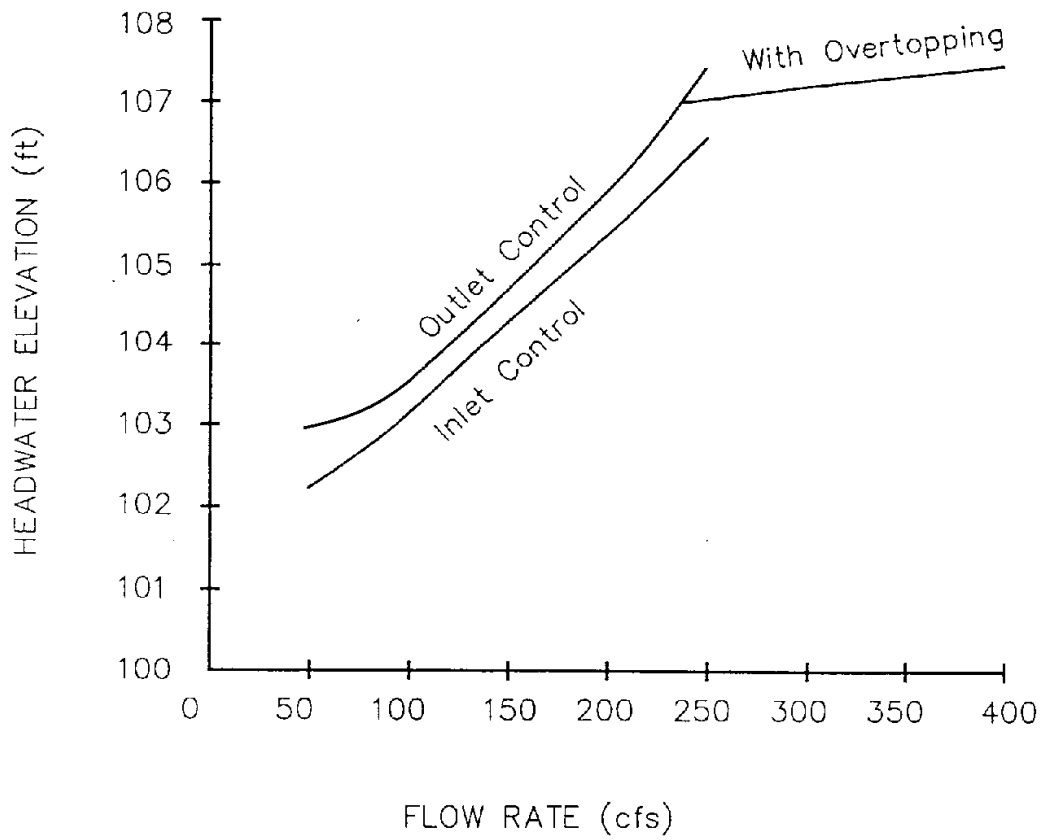
(8) $EL_{ho} = EL_o + H + h_o$

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 stormdrain system, culvert type inlets are frequently used. The stormdrain hydraulic grade line must be considered when estimating the inlet capacity for culvert type inlets. The stormdrain 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 stormdrains. 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, 1983).

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 5.4.3.

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 7, 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 6.1, 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 shoud 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 7, 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 the following equation from *Hydraulic Design of Energy Dissipators for Culverts and Channels* (USDOT, FHWA, HEC-14, 1983).

$$\frac{d_s}{y_e} = \alpha \left(\frac{Q}{\sqrt{g}\, y_e^{2.5}} \right)^{\beta} \left(\frac{t}{t_o} \right)^{\theta} \tag{5.8}$$

where:

d_s = depth of scour hole, ft

y_e = $(A/2)^{0.5}$ = equivalent depth, ft

A = flow area, sq ft

Q = discharge, cfs

g = gravitation constant, 32.2 ft/sec^2

t = time of scour, set at 30 minutes.

t_o = base time (= 316 minutes) used in the experiments in deriving the coefficients α, β, and θ;

DI = discharge intensity

$$DI = \left(\frac{Q}{\sqrt{g}\, y_e^{2.5}} \right) \tag{5.9}$$

For uniform sand with D_{50} = 0.2 mm (fine sand), values of the coefficients α, β, and θ are:

α = 2.72

β = 0.375

θ = 0.100

For uniform sand with $D_{50} = 2.0$ mm (very coarse sand), values of the coefficients α, β, and θ are:

α = 1.86

β = 0.45

θ = 0.09

The following guidelines, applicable to cutoff walls, are based on the computed scour depth analysis identified above.

1. The depth of the cutoff wall should be equal to or greater than the maximum depth of scour.

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 riprap is used at culvert outlets.

Topics on scour are presented in Chapter 10, 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 7, Section 7.4 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 ths 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 10, 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 10, Sedimentation for scour analysis protocol.

5.7 REFERENCES

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6 OPEN CHANNELS

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6.1 SYMBOLS

The following symbols will be used in equations throughout Chapter 6.

α	=	Velocity head coefficient
θ	=	The bank angle with the horizontal, degrees
β	=	Momentum coefficient
Φ	=	Angle of repose, 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
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
d	=	Depth of flow, or hydraulic depth, ft
d_{ave}	=	Average depth of flow in the main channel, ft
d_i	=	The average diameter of a rock particle for which "i" percent of gradation is finer by weight, mm
D	=	Diameter, ft
E	=	Specific energy, 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
K_1	=	Bank angle correction factor
KE	=	Kinetic energy, ft
l	=	Length along channel, 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	=	Discharge, cfs
R	=	Hydraulic radius, ft
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
V or $\bar{v}$	=	Average velocity of a section, ft/sec
v	=	Vectoral velocity, ft/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
z	=	Distance from water surface to the centroid of the section, 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, often termed the thalweg, 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 constructed channels, such as determination of freeboard and toe down requirements;
- Section 6.6 - Design guidelines are recommended for various types of engineered 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 10, Sedimentation.

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 infor-

mation 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 (). 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 incli-

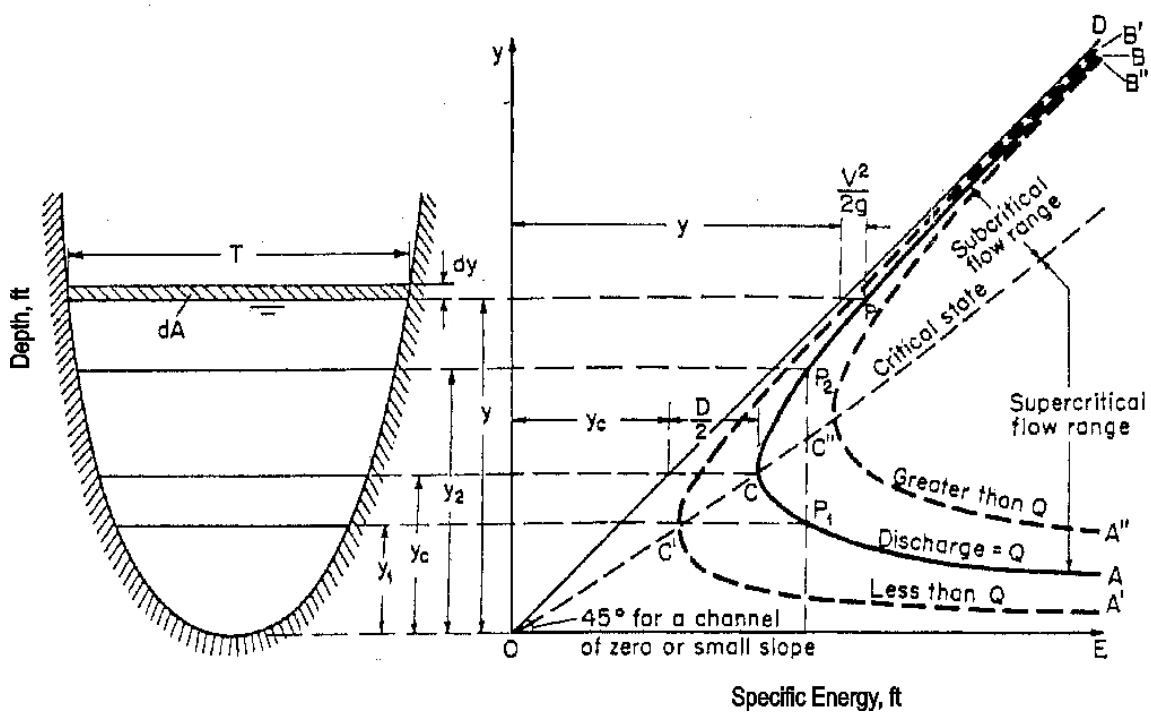
nation equal to 45°. At any point P on this curve, the ordinate represents the depth of flow, and the abscissa represents the specific energy that is equal to the sum of the pressure head, y, and the velocity head, $V^2/2g$. The curve shows that, for a given specific energy, there are two possible depths, 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 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.

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?

$$\frac{400^2}{32.2} = \frac{(10y_c)^3}{10}$$

$$y_c = 3.68 \quad \text{ft}$$

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 nor-

mal to the direction of flow (sq ft). Under steady flow conditions, the discharge is constant and:

$$Q = A_1 V_1 = A_2 V_2 \tag{6.5}$$

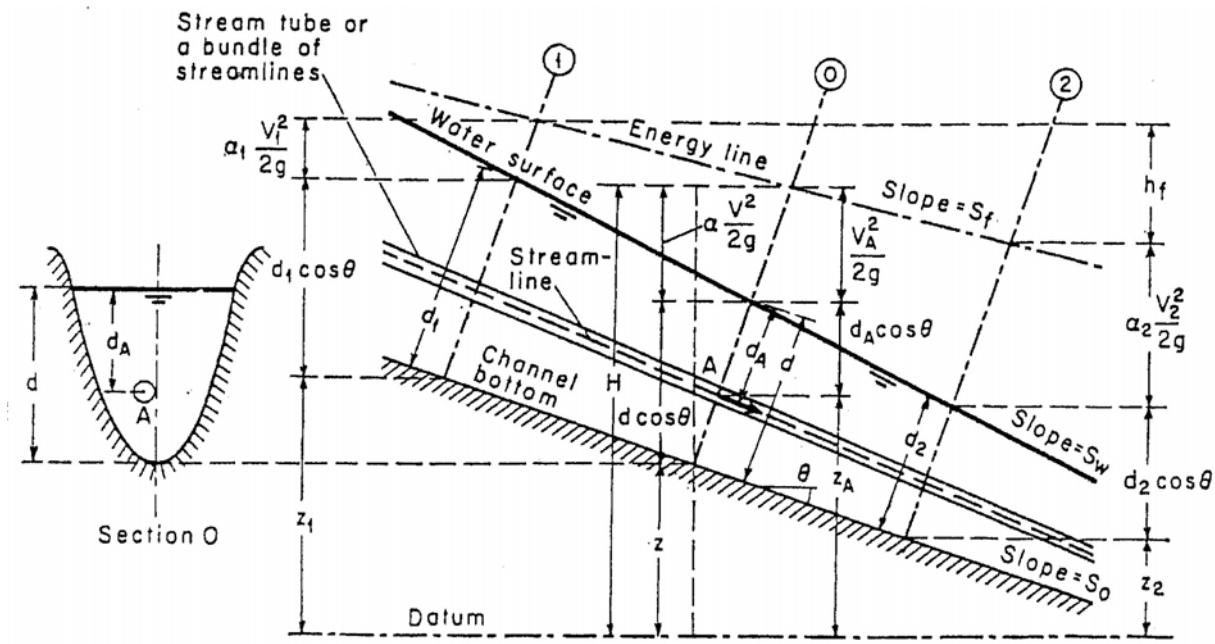
The subscripts denote different channel sections. Equation (6.5) is known as the Continuity Equation and is applicable to the flow conditions addressed in this chapter.

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

Newton's Second 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:

$$E_T = \alpha_1 \frac{V_1^2}{2g} + y_1 + z_1 = \alpha_2 \frac{V_2^2}{2g} + y_2 + z_2 + h_f \tag{6.6}$$

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:

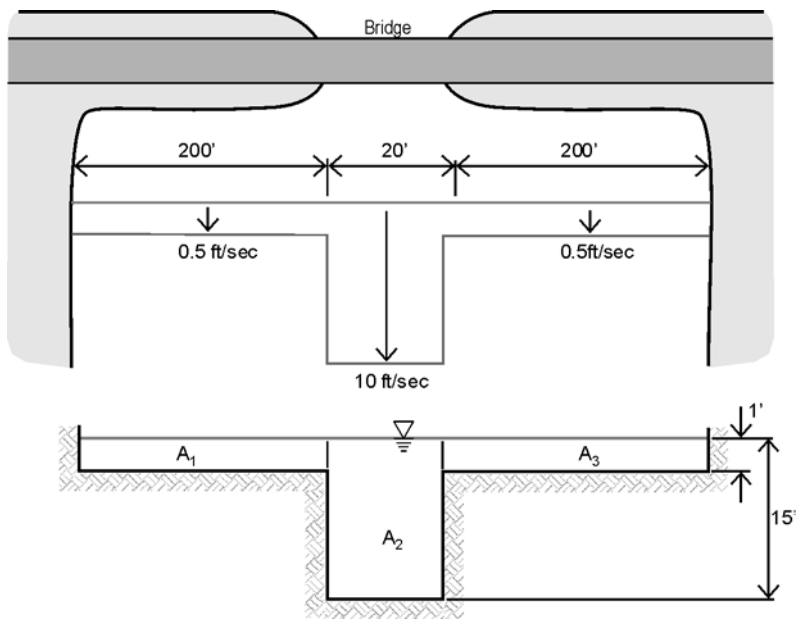
$$\alpha = \frac{\int v^3 dA}{\overline{v}^3 A} \tag{6.8}$$

Computational form:

$$\alpha = \frac{\sum v^3 \Delta A}{\overline{v}^3 A} \tag{6.9}$$

Example 6.2: The flow in a river downstream of a bridge constriction is as shown. Calculate the energy coefficient.

$Q_T = (.5)(1)(200) + (10)(15)(20) + (.5)(1)(200) = 3200$ cfs



$$A = A_1 + A_2 + A_3 = 200 + 300 + 200 = 700 \text{ ft}^2$$

$$\overline{v} = \frac{Q}{A} = \frac{3200}{700} = 4.57 \text{ ft/sec}$$

$$\alpha = \frac{\Sigma v^3 \Delta A}{\overline{v}^3 A} = \frac{(.5)^3(200) + (10)^3(300) + (.5)^3(200)}{(4.57)^3(700)}$$

$\alpha = 4.49$

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 $\beta wQV/g$. 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:

$$z_1 + y_1 + \beta_1 \frac{V_1^2}{2g} = z_2 + y_2 + \beta_2 \frac{V_2^2}{2g} + h_f' \tag{6.11}$$

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.

$$M = \beta m v \qquad \beta \geq 1.0 \tag{6.12}$$

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

$$\beta = \frac{\Sigma v^2 \Delta A}{\bar{v}^2 A} = \frac{(.5)^2(200) + (10)^2(300) + (.5)^2(200)}{(4.57)^2(700)}$$

$$\beta = 2.06$$

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,

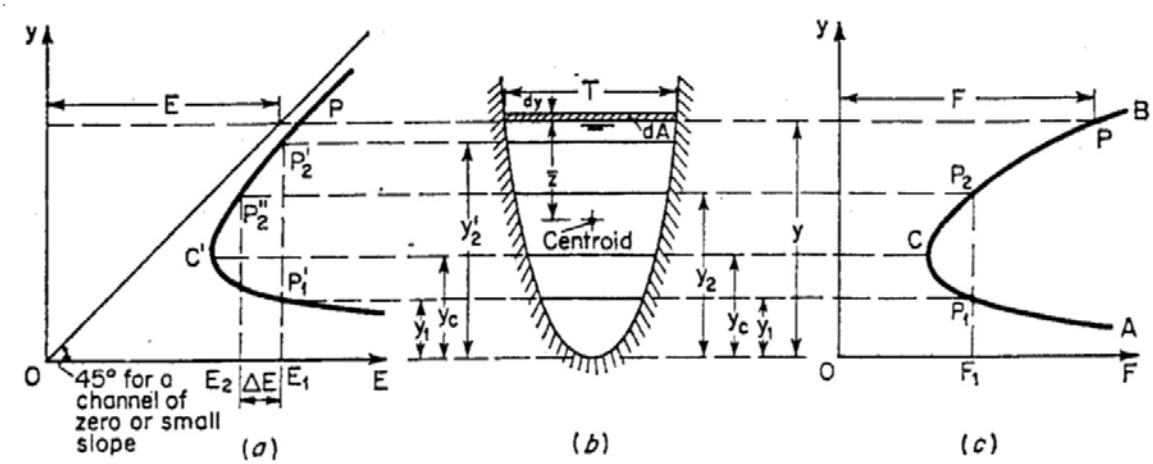
$$P_{h_1} = wz_1A_1 \tag{6.16}$$

$$P_{h_2} = wz_2A_2 \tag{6.17}$$

with z as the distance to the centroids of the respective water areas below the surface of flow. With constant steady flow and $V = Q/A$,

$$\frac{Q^2}{gA_1} + z_1A_1 = \frac{Q^2}{gA_2} + z_2A_2 \tag{6.18}$$

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 (for purposes, herein).

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. Additional information concerning Manning's roughness coefficients can be found in Phillips and Ingersoll (1998), Thomsen and Hjalmarson (1991), Davidian (1984), Aldridge and Garret (1973) and Barnes (1967).

Typical values of roughness coefficients are given in Table 6.1. 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 6.1, 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 6.1 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. In addition, Simons and Richardson (1966) observed that on natural sand-bed streams, resistance to flow might decrease significantly whenever large flows occur. As the depth of flow increases, resistance to flow due to bed roughness decreases. Flows that are sufficient to damage vegetation also reduce resistance to flow.

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.

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 Barnes (1967) and Thomsen and Hjalmarson (1991) for discussion of the estimation of n values for 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 6.2 lists the algebraic expressions for computing the hydraulic geometry for typical channel sections.

TABLE 6.1
MANNING'S ROUGHNESS COEFFICIENTS
(1)

Channel Material	Roughness Coefficient (n)		
	Minimum	**Normal**	**Maximum**
Concrete:			
Trowel finish	0.011	0.013	0.015
Float finish	0.013	0.015	0.016
Unfinished	0.014	0.017	0.020
Shotcrete, good section	0.016	0.019	0.023
Shotcrete, wavy section	0.018	0.022	0.025
Soil cement	0.018	0.020	0.025
Constructed channels with earthen bed			
Clean earth; straight	0.018	0.022	0.025
Earth with grass and forbs	0.020	0.025	0.030
Earth with sparse trees and shrubs	0.024	0.032	0.040
Shotcrete	0.018	0.022	0.025
Soil cement	0.022	0.025	0.028
Concrete	0.017	0.020	0.024
Riprap	0.023	0.032	0.036

1. From: Simons, Li and Associates, 1988. Adapted from Chow (1959) and Aldridge and Garret (1973)

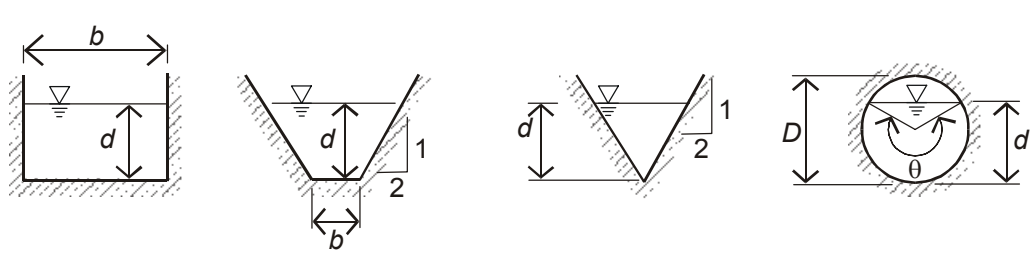
TABLE 6.2
ELEMENTS OF CHANNEL SECTIONS
(1)

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	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2\ zd$
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

(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 landscaped 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.

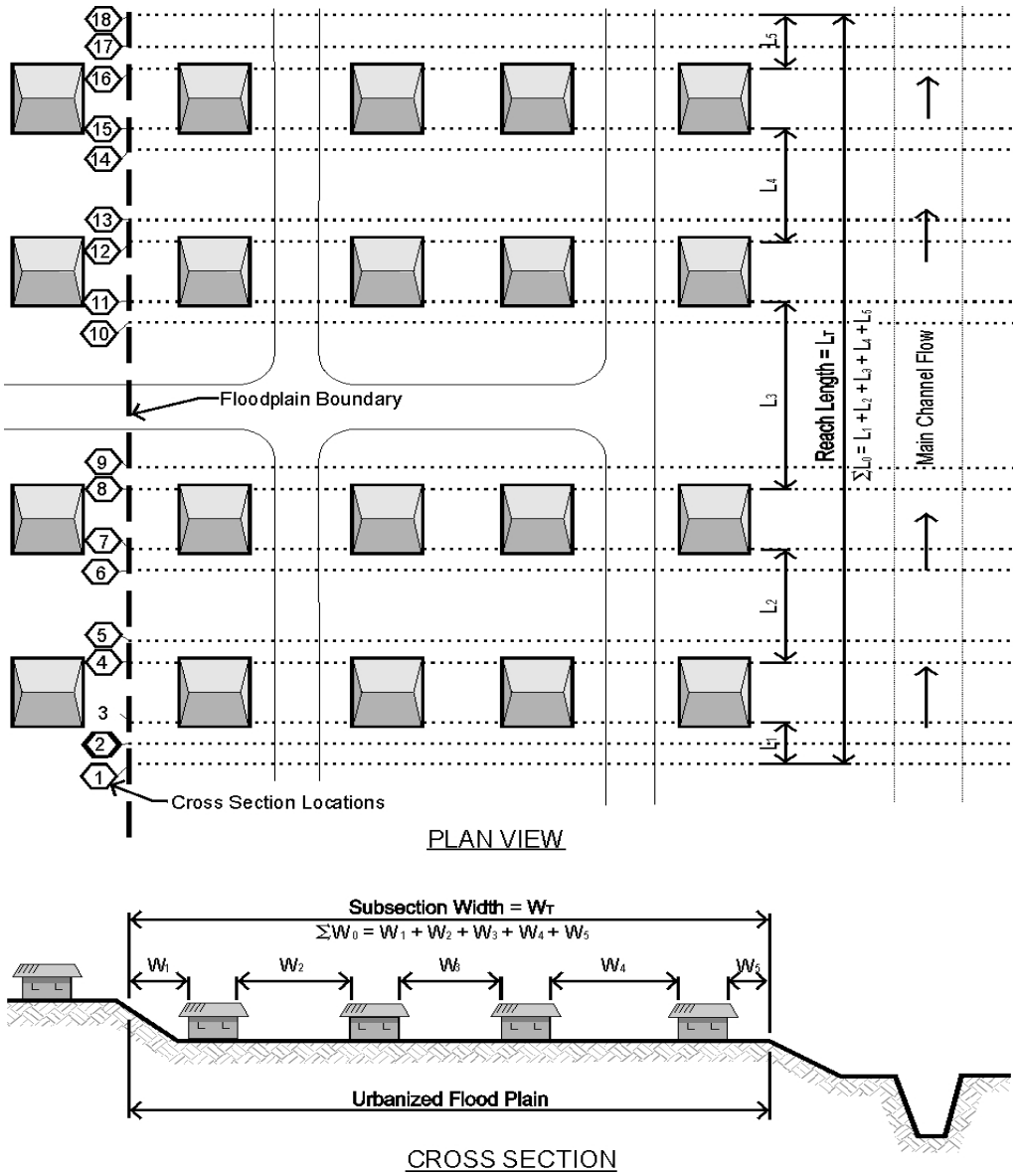
In the urban setting, it is not unusual for buildings and other structures to occupy a significant portion of any given hydraulic cross section. Under these circumstances, it is often difficult to estimate both the effective width of the cross-section and the Manning's roughness coefficient for the overbank 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 6.2 for the cross section area and the hydraulic radius:

$A = bd + zd^2 = (20)(4) + (2)(42) = 112 \text{ ft}^2$

$$R = \frac{bd + zd^2}{b + 2d\sqrt{(2^2 + 1)}} = \frac{(20)(4) + (2)(4^2)}{20 + (2)(4)\sqrt{(2^2 + 1)}} = \frac{112}{37.89} = 2.96 \text{ ft}$$

Select the appropriate Manning's roughness coefficient from Table 6.1. Substituting these values in Equation (6.22):

$$Q = \frac{1.49}{n} A R^{2/3} S_f^{1/2}$$

$$Q = \frac{1.49(112.0)(2.96)^{2/3}(0.0016)^{1/2}}{0.022} = 625.5 \text{ cfs}$$

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):

$$AR^{2/3} = \frac{Qn}{1.49 S^{1/2}} = \frac{(625)(0.022)}{(1.49)(0.0016^{1/2})} = 230.70 =$$

$$(20d + 2d^2) \times \left[\frac{(20d + 2d^2)}{20 + (2d)\sqrt{(2^2 + 1)}}\right]^{2/3}$$

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:

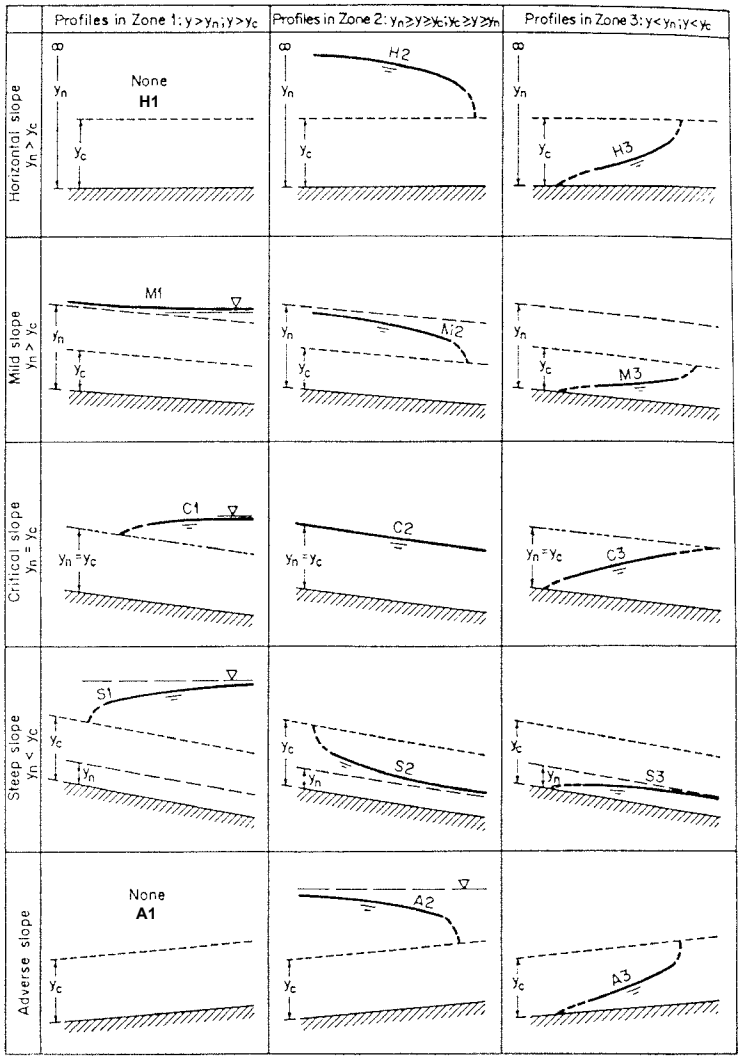
- The **letter** is descriptive of the slope, i.e., **H** for horizontal, **M** for mild, **C** for critical, **S** for steep (supercritical), and **A** for adverse slope; and

- 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, critical depth should be used for determining the water surface profile. Using the critical depth will produce higher, and thus more conservative, water surface elevations for design purposes. 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 the references listed at the end of this chapter.

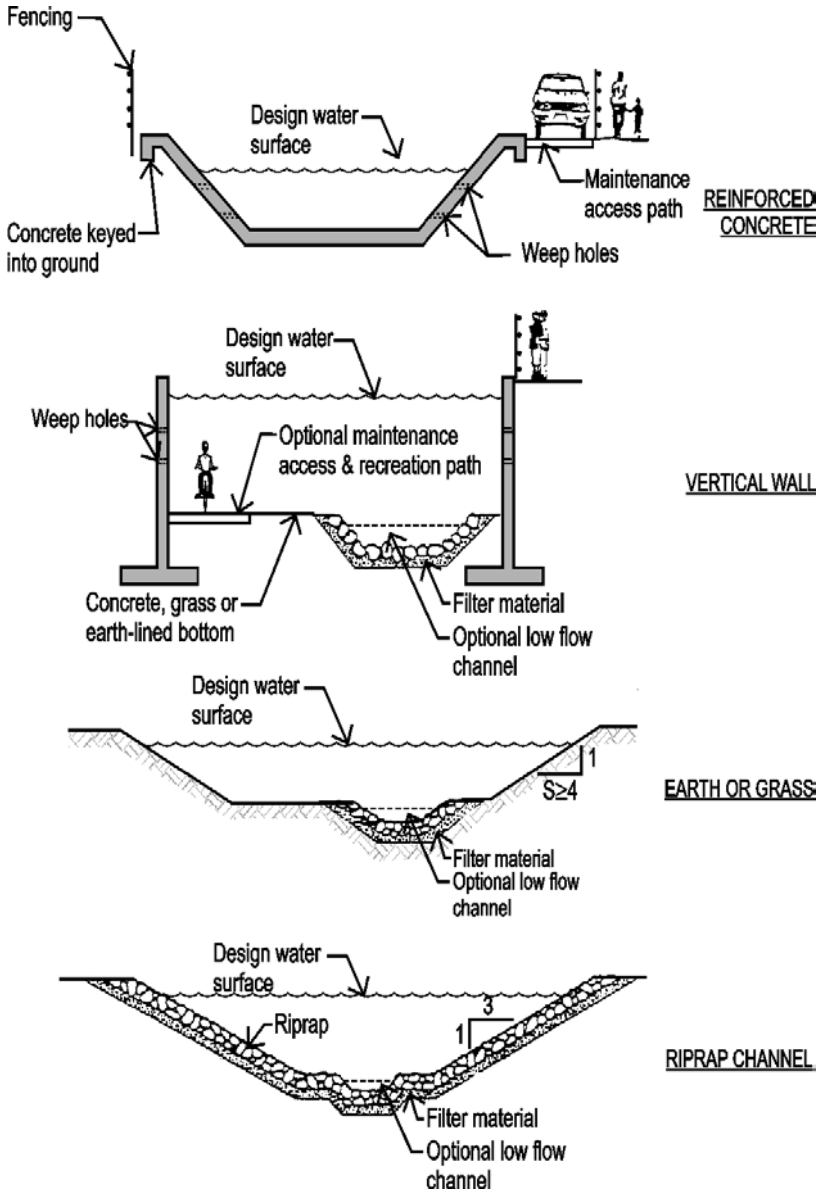
Grade control is a critical factor in the long-term behavior of non-rigid channels. By definition, grade control is any natural or man-made structure within a channel that limits or prevents vertical movement of the channel bed, either degradation or aggradation. Examples include rock outcroppings, culverts 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. At a minimum, the grade control structure should extend to a depth sufficient to prevent upstream migration of the headcut. 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.

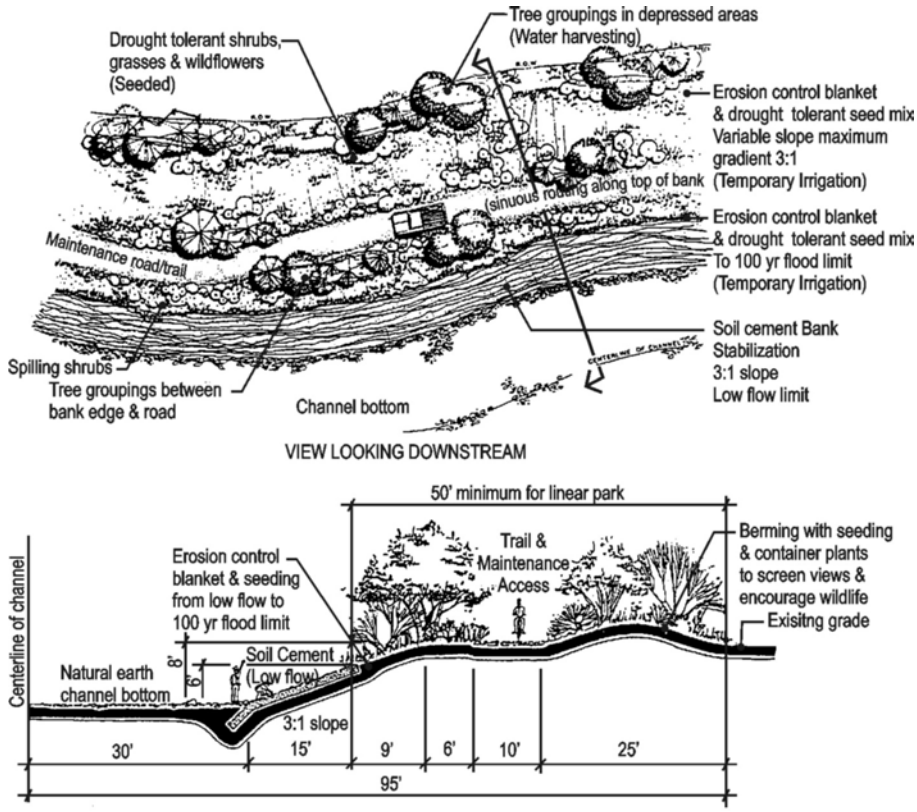
Grass Lined Channels

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. 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 6.6.4.

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 and, thus, may not function satisfactorily long-term when used in the bottom of channels. 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. It is recommended that low flow channels be provided whenever the following condition exists:

$$\frac{b}{Vy} \geq 1.40 \qquad (6.24)$$

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 the jurisdictions *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.

6.5 DESIGN FACTORS FOR CONSTRUCTED 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 10.

6.5.3 Maximum Velocity

For earthen or grass lined channels, maximum permissible velocities should be governed by Table 6.3 and Table 6.4, respectively. If the natural channel slope would cause excessive velocity, employ drop structures, checks, riprap (USDOT, FHWA HEC-11), or other suitable velocity control design features .

TABLE 6.3
MAX. PERMISSIBLE VELOCITIES FOR ROADSIDE DRAINAGE CHANNELS W/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 10. 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).

TABLE 6.4
ROADSIDE CHANNELS WITH 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).

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.

6.5.7 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 must be made of the maximum scour expected to occur over the design life of the structure. Second, a means of protection must be provided for the maximum scour. The first parameter, scour depth estimation, requires specialized analysis techniques by a qualified engineer. References for scour and sediment transport analysis are included in Chapter 10. Mitigation measures for providing protection for the maximum scour are presented in this section.

The two methods of providing toe protection in erodible channels are:

1. To extend protection to the maximum estimated depth of scour; 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 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 6.8a shows the desirable configuration for a weighted riprap toe. Other materials utilized are gabion mattresses (see Figure 6.8b). These mattresses are anchored to the bank protection and their riverward ends are allowed to lower as scour occurs. Studies by Linder (1976) and the U.S. Army Corps of Engineers (1981) 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 6.8a.

3. Complex toe designs that are difficult to construct are not necessary.

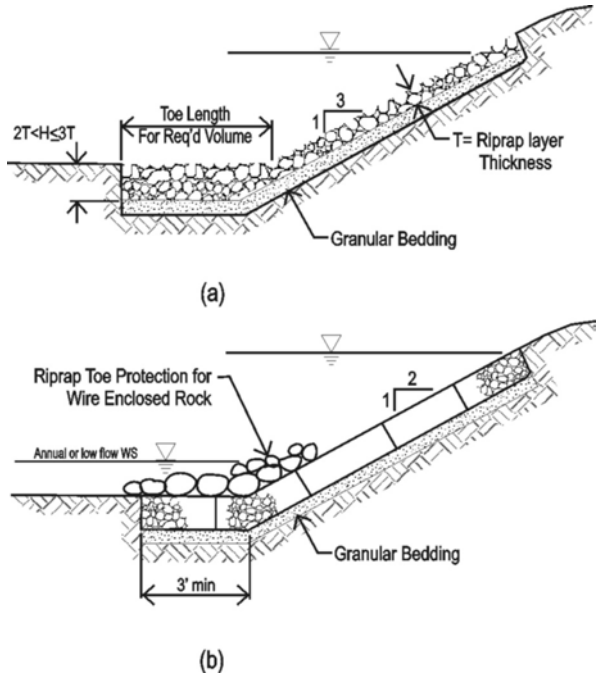
4. Downslope rock movement occurred without significant movement in the downstream direction.

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. Toe volume in the physical model was approximately equal to the volume needed to extend the bank protection to the maximum scour depth at a 2:1 slope. Linder (1976) recommends a toe volume equal to 1.5 times the volume of extending the bank protection to the maximum scour depth.

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 such flow conditions.

FIGURE 6.8
TOE PROTECTION CHANNEL LINING
(Wright-McLaughlin, 1969)



6.6 DESIGN GUIDELINES FOR ENGINEERED 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 sub-grade; 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 supercritical flow conditions and/or when velocities exceed five feet per second for earth lined channels. 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

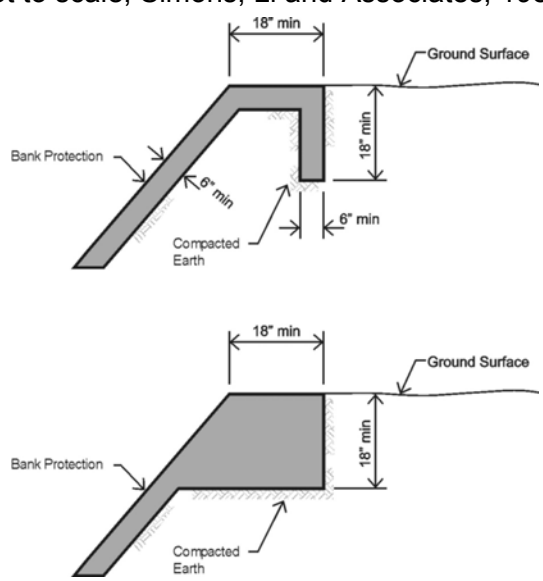
any curvature in a supercritical channel. Careful attention must be taken to prevent or control excessive oscillatory waves that may extend the entire length of the channel from only minor obstructions upstream. Imperfections at joints may rapidly cause a deterioration of the joints, in which case a 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 stormdrain 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 overbanks as shown in Figure 6.9.

FIGURE 6.9
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. 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.

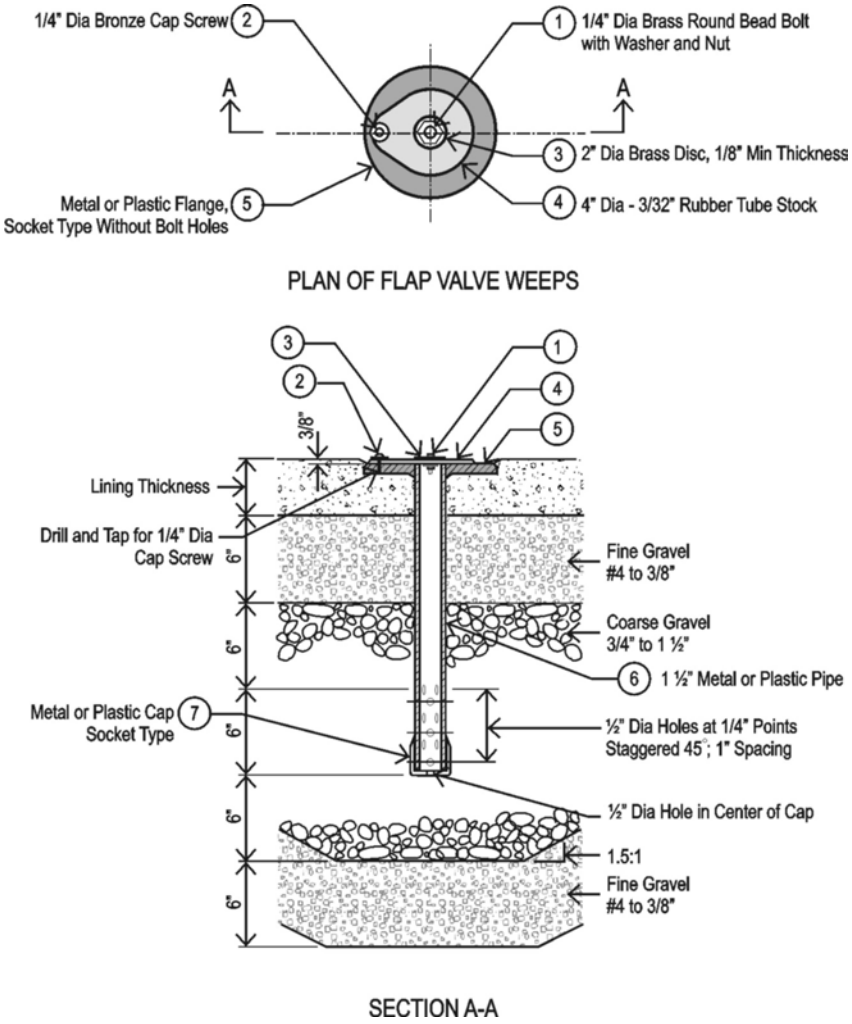
Long-term stability of concrete lined channels depends in part on proper bedding. Undisturbed soils often are satisfactory for a foundation for lining without further treatment. Expansive clays are usually an extreme hazard to concrete lining and should be avoided. A filter underneath the lining is recommended 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 10.

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 6.10 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).

FIGURE 6.10
FLAP VALVE INSTALLATION FOR A CHANNEL UNDERDRAIN
(Simons, Li and Associates, 1981)



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 ground water 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 (1985) 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.

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 6.11 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 6.12 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 permissable.

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 6.9. 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 stormdrain 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. Further design information may be found in ACI 230.1, *State Of The Art Report On Soil Cement*. Additional information on design and construction is available from the Portland Cement Association, Skokie, IL.

FIGURE 6.11
SOIL CEMENT PLACEMENT DETAIL
(Not to Scale)

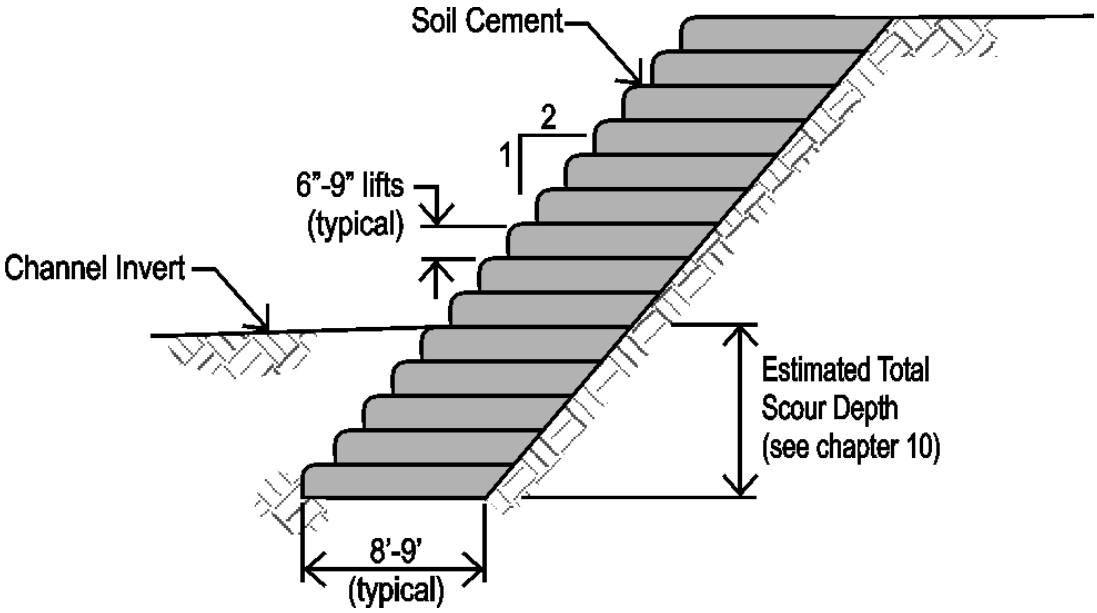
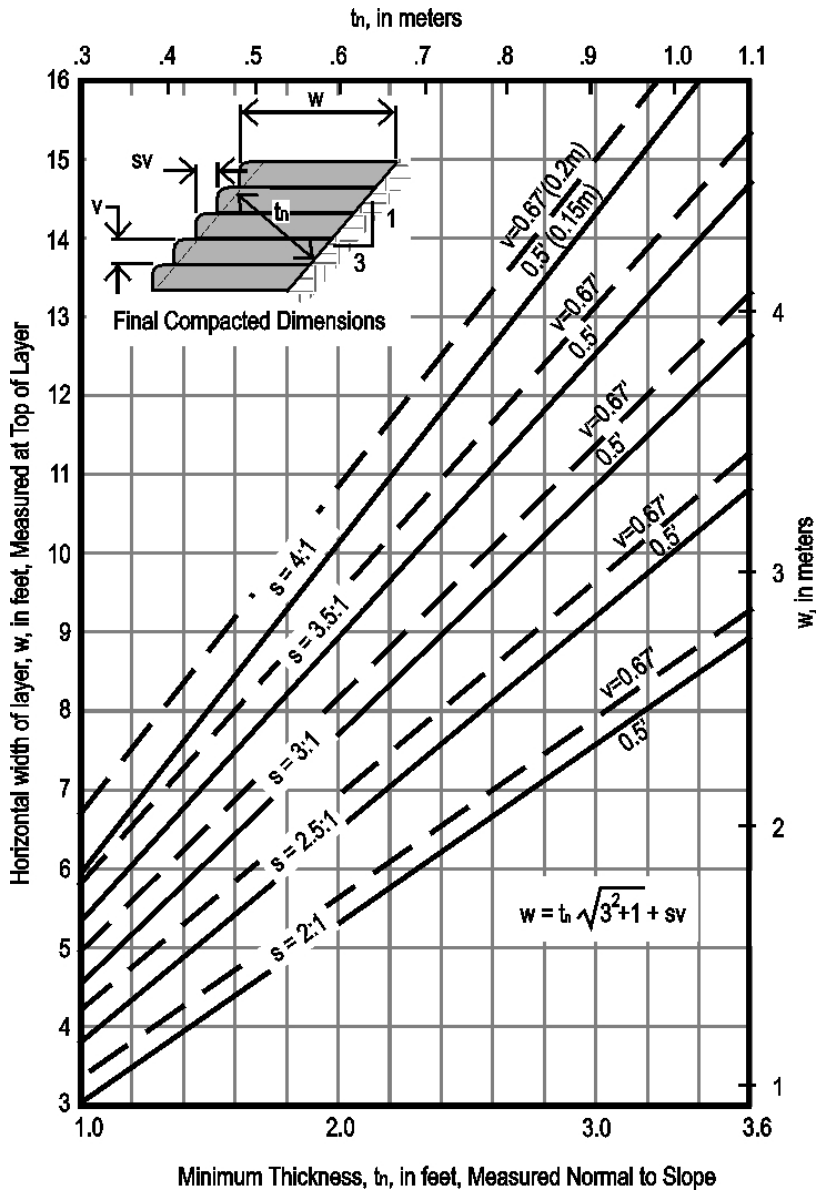


FIGURE 6.12
RELATIONSHIPS FOR SOIL CEMENT LINING
SLOPE, FACING THICKNESS, LAYER THICKNESS, AND HORIZONTAL LAYER WIDTH
(Portland Cement Association, 1987)



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 (Density) - The design stone size for a channel depends on the particle weight, which is a function of the density or specific gravity of the rock material. A typical value of specific gravity in Maricopa County is 2.4. All stones composing the riprap should have a specific gravity equal to or exceeding 2.4, following the standard test ASTM C127.

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, 1995).

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 6.5 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 6.5
RIPRAP GRADATION LIMITS
(USDOT, FHWA, HEC-11)

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 6.6 outlines recommended standard gradations.

The Type-I and Type-II bedding specifications shown in Table 6.6 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 6.6 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 6.7.

TABLE 6.6
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 (4.75 mm)	95 to 100	0 to 20
#16 (1.18 mm)	45 to 80	-
#50 (0.30 mm)	10 to 30	-
#100 (0.15 mm)	2 to 10	-
#200 (0.075 mm)	0 to 2	0 to 3

(1) Percent passing by weight

TABLE 6.7
THICKNESS REQUIREMENTS FOR GRAVEL BEDDING

Riprap Size Classification, inches	Minimum Bedding Thickness, inches		
	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).

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 6.6.4 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.

Riprap Sizing - Several reference sources are available for design procedures. Two recommended sources are:

1. *Design of Riprap Revetment* (Federal Highway Administration, Hydraulic Engineering Circular No. 11, Publication No. FHWA-IP-89-016, March 1989)

2. *Hydraulic Design of Flood Control Channels* (Corps of Engineers, EM-1110-2-1601, 1991)

The riprap sizing method presented here is from HEC-11 (for a complete discussion on this method the designer is referred to the above referenced documents). This method is based on tractive force (shear stress) theory but with velocity as its primary design parameter. This is a blend between the two approaches of permissible velocity and permissible tractive force. The hydraulic assumptions are uniform, steady, subcritical flow. However, adjustments to the design equation are provided for other regimes and conditions such as gradually varying flow and approaching rapidly varying flow. In this method, the riprap size is selected such that the flow induced tractive force does not exceed the critical shear stress of the riprap. The critical shear is based on Shield's relationship, a function of specific weight of water, specific weight of the riprap material, the median rock size (d_{50}), Shields parameter, and a factor that is a function of the bank angle and riprap's material angle of repose. The average shear stress or tractive force exerted by flowing water is the product of unit weight of water, energy grade line slope and hydraulic radius. These two equations are combined to develop the design tractive force relationship in terms of a stability factor (SF). The stability factor is defined as the ratio of the average tractive force exerted by the flow field and the riprap materials critical shear stress. Therefore if the stability factor is greater than 1.0, the critical shear stress is greater than the flow induced tractive stress and the riprap is considered stable.

For the HEC-11 method the d_{50} (ft) is determined by:

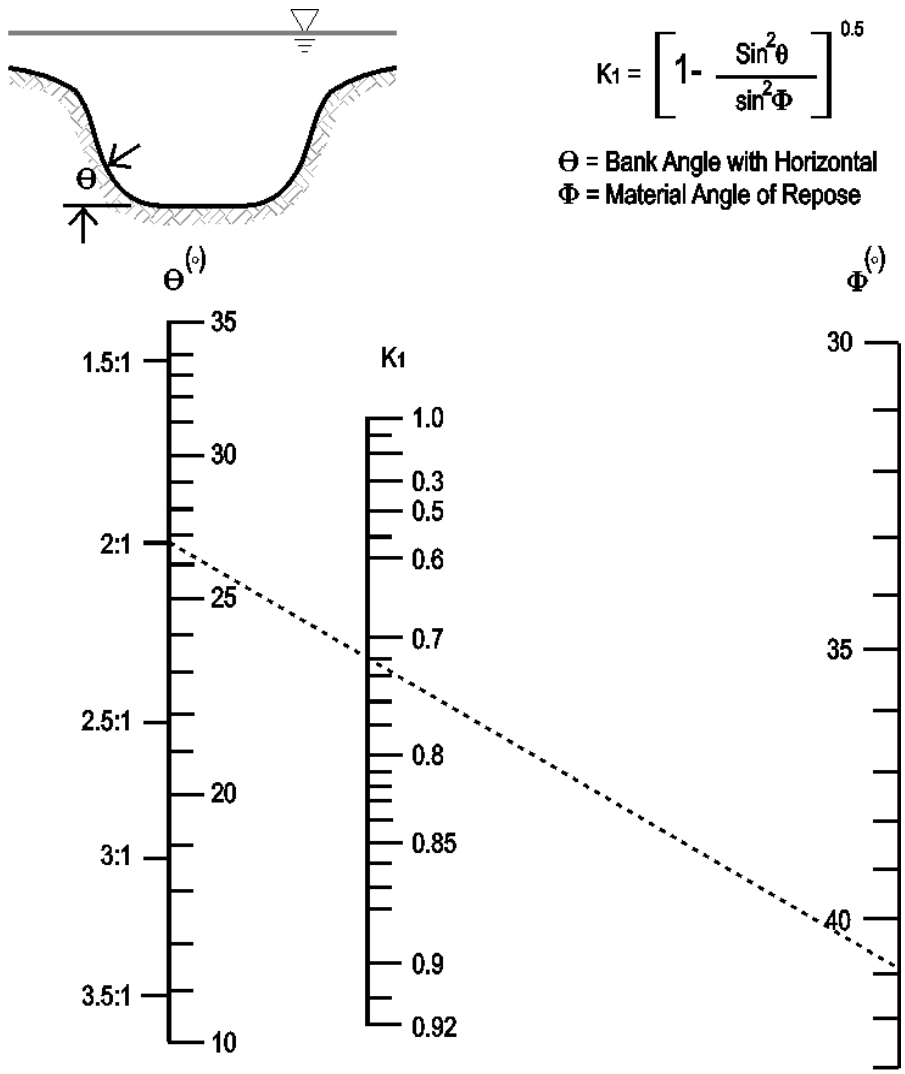
$$d_{50} = \frac{0.001 V_a^3}{d_{avg}^{0.5} K_1^{1.5}} \tag{6.33}$$

Where V_a (ft/sec) is the average velocity in the main channel, d_{avg} (ft) is the average flow depth in the main channel, and K_1 is the bank angle correction factor. The bank angle correction factor is determined using Equation (6.34).

$$K_1 = \left[1 - \frac{\sin^2\theta}{\sin^2\Phi}\right]^{0.5} \tag{6.34}$$

Where θ is the bank angle with the horizontal, Φ is the riprap material's angle of repose. The bank angle correction factor can also be determined using Figure 6.13. The riprap material's angle of repose can be determined using Figure 6.14.

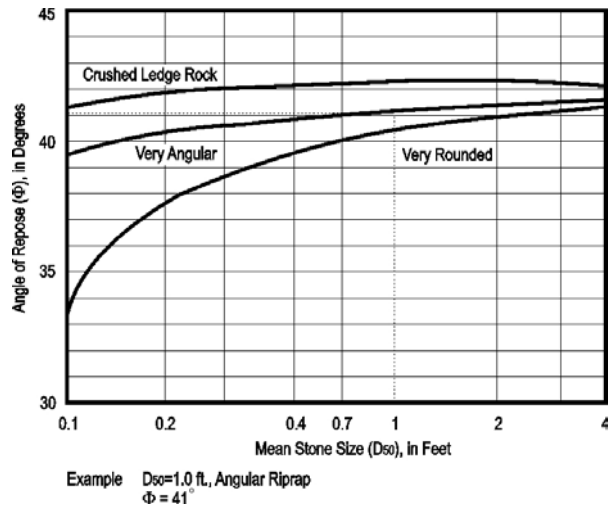
FIGURE 6.13
BANK ANGLE CORRECTION FACTOR, K_1
(USDOT, FHWA, HEC-11, 1989)



Example

Given:	Find:	Solution:
θ = 2:1	K_1	K_1 = 0.73
Very angular		
Φ = 41°		

FIGURE 6.14
ANGLE OF REPOSE OF RIPRAP IN TERMS OF MEAN SIZE AND SHAPE OF STONE
(USDOT, FHWA, HEC-11, 1989)



Equation (6.33) is based on a rock riprap specific gravity of 2.65, and a stability factor of 1.2. Equation (6.35) and Equation (6.36) present correction factors for other specific gravities and stability factors.

$$C_{sg} = \frac{2.12}{(S_s - 1)^{1.5}} \tag{6.35}$$

Where S_s is the specific gravity of the rock riprap.

$$C_{sf} = \left(\frac{SF}{1.2}\right)^{1.5} \tag{6.36}$$

Where (SF) is the stability factor to be applied. Table 6.8 presents guidelines for the selection of an appropriate value for the stability factor.

The correction factors computed using Equation (6.34) and Equation (6.35) are multiplied together to form a single correction factor C. This correction factor is then multiplied by the riprap size computed from Equation (6.33) to arrive at a stable riprap size.

The stability factor is used to reflect the uncertainty in the hydraulic conditions at a particular site. Equation (6.33) is based on the assumption of uniform or gradually varying flow. In many instances, this assumption is violated or other uncertainties come to bear. For example, debris and/or ice impacts, or the cumulative effect of high shear stresses and forces from wind and/or boat generated waves. The stability factor is used to increase the design rock size when these conditions must be considered. Typically, the minimum thickness of riprap linings should be the greater of $1 \times d_{100}$ or $1.5 \times d_{50}$.

TABLE 6.8
STABILITY FACTORS
(USDOT, FHWA, HEC-11, 1989)

Condition	Stability Factor Range
Uniform Flow: Straight or mildly curving reach (curve radius/channel width > 30); Impact from wave action and floating debris is minimal; Little or no uncertainty in design parameters.	1.0 - 1.2
Gradually Varying Flow: Moderate bend curvature (30 > curve radius/ channel width > 10); Impact from wave action and floating debris is moderate.	1.3 - 1.6
Approaching rapidly varying flow: Sharp bend curvature (10 > curve radius/channel width); Significant impact potential from floating debris and/ or ice; Significant wind and/or boat generated waves (1-2 ft); High flow turbulence; Turbulently mixing flow at bridge abutments; Significant uncertainty in design parameters.	1.6 - 2.0

Grouted Rock

Grouted rock 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. The grout must extend the full thickness of the rock blanket, with the face rocks exposed for a maximum of one-fourth to one-third of their depth.

This lining type is often suggested as a substitute for adequately sized riprap. It is not an equivalent product because it is neither rigid nor flexible. Any movement or settlement of the subgrade immediately results in cracks in the matrix which, in turn, allow water to enter behind the lining and greatly accelerate the lining's destruction. Some jurisdictions do not accept this alternative and its use is discouraged with two exceptions; riprap designed by the guidelines contained herein can be grouted to 1) minimize vandalism and/or 2) to inhibit the growth of volunteer vegetation and to aid in maintenance.

6.6.4 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 which, under normal conditions here in the arid southwest, is considered to be about 35 years. In applications where the gabions are subjected to frequent wet conditions, the life span diminishes to about 15 years (Myers, 2000). 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. 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_o 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.

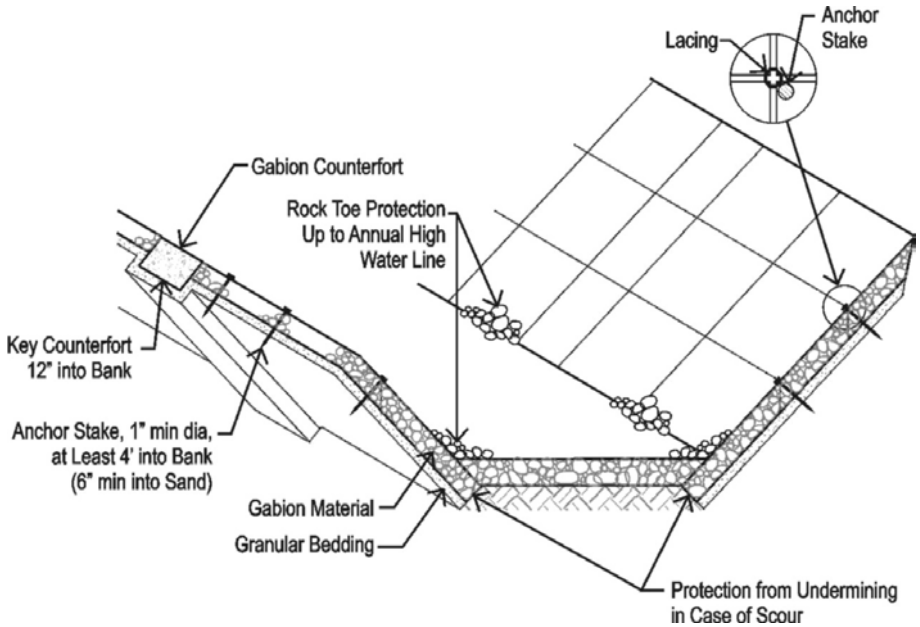
Design Considerations

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 6.15 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 6.15
SLOPE MATTRESS LINING
(Wright-McLaughlin Engineers, 1969)



6.6.5 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 2000 cfs.

Open Channel Hydraulics

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, 2002*). 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. 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 *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.

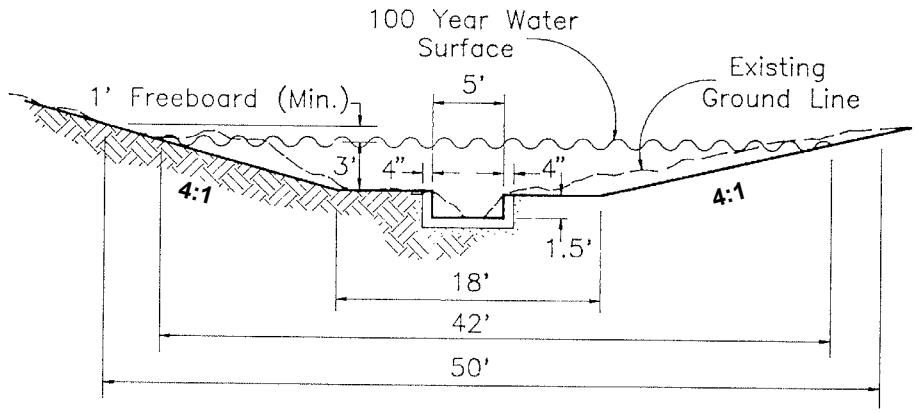
The analysis shall address sediment transport, scour, lateral migration, and river mechanics as discussed in Chapter 10.

Plans submitted for review shall include profiles showing the top of levee protection, toe-down, hydraulic grade line, 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.

6.6.6 Design Example

Problem Description

Improve a small unlined channel by adding an incised low flow channel and providing bank and edge erosion protection. 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.



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})].

$$A_T = A_{lf} + A_{mc}$$

$$= (1.5)(5) + 3((18 + 42)/2)$$

$$= 7.5 + 90 = 7.5 + 90$$

$$A_T = 97.5 \text{ sq ft}$$

2. Find wetted perimeter and indicate a 4 inch thickness for low flow wall.

$$P_T = P_{lf} + P_{mc}$$

$$= \left[2(0.33) + 2(1.5) + 5\right] + \left[(18 - 5.67) + 2(3)\left(1^2 + 4^2\right)^{0.5}\right]$$

$P_T = 47.5$ ft

3. Find hydraulic radius.

$$R = A_T / P_T$$

$$= 97.5/45.7$$

$$R = 2.13\ ft$$

4. Determine Manning's "n" from Table 6.1.

Find composite "n" value:

Concrete lined low flow: n = 0.015

Grass-lined main channel: n = 0.025

$$n = [\ \Sigma_i \frac{(P_i\, n_i^{1.5})}{P_T}\ ]^{0.67}$$

$$n = [\ \frac{(P_{lf})(n_{lf}^{1.5}) + (P_{mc})(n_{mc}^{1.5})}{P_T}\ ]^{0.67}$$

$$n = [\ \frac{8.67(0.015)^{1.5} + 37.0(0.025)^{1.5}}{45.7}\ ]^{0.67}$$

$$n = [\ \frac{(0.016 + 0.15)}{45.7}\ ]^{0.67}$$

$$n = 0.023$$

5. Substitute values to solve for slope and multiply Equation (6.22) by A_T and rearrange:

$$Q = (1.49/n) A_T R^{0.67} S_o^{\ 0.5}$$

$$S_o = \left[Qn/\left(1.49\, A_T R^{0.67}\right)\right]^2$$

$[565\ (0.023)/(1.49)(97.5)(2.13)^{0.67}]^2$

$S_o = 0.0029$ 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:

$$A_T R^{0.67} = Qn/1.49 S_o^{0.5}$$

$$= 565(0.023)/1.49(0.006)^{0.5}$$

$$= 112.6 \text{ ft}^3$$

By trial and error,

$$y_n = 2.45 \text{ ft}$$

$$A_T = 7.5 + 2.45((18 + 37.6)/2) = 75.6 \text{ ft}^2$$

$$P_T = 8.67 + ((18 - 5.67) + 2\,(2.45)(1^2 + 4^2)^{0.5}) = 41.2 \text{ ft}$$

$$R = A_T / P_T = 75.6/41.2 = 1.83\,ft$$

$$A_T R^{0.67} = (75.6)(1.83)^{0.67} = 113.3 \approx 112.6 ft^3 \;\therefore \text{ OK.}$$

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_{if}, 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 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 7 for grade control structures.

3. Check channel transitions (see Chapter 7).

C. Check freeboard requirement.

Using the revised slope of S_o = 0.0029 ft/ft, and velocity of V = 5.8 ft/sec.

$FB = 0.25\left(y + V^2/2g\right)$

$= 0.25\left(3 + 5.8^2/2(32.2)\right)$

$= 0.25(3.52)$

$FB = 0.88\,ft$

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 for allowable grass and soil types.

Channel slope = 0.0029 << 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.

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7 HYDRAULIC STRUCTURES

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7.1 SYMBOLS

The following symbols are used in equations throughout Chapter 7:

α	=	Angular variation of sidewall with respect to channel centerline
α_1	=	Kinetic energy correction coefficient
β	=	Standing wave front angle
ρ	=	Density of water, 1.94 lb sec^2/ft^4
τ	=	Shear stress on the bed caused by the flow of water, psi
γ	=	Specific weight, lb/ft^3
A	=	Area, sq ft
A_1	=	Area upstream of the jump, sq ft
A_2	=	Area downstream of the jump, sq ft
a_n	=	Net area through the trashrack bars, sq ft
a_g	=	Gross area of the trashrack bars and supports, sq ft
b	=	Bottom width, ft
b_t	=	Trickle channel width, ft
B	=	Basin depth below downstream channel, ft
C	=	Weir Coefficient
C_d	=	Drag force coefficient
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_2	=	Depth of basin tailwater, ft
d_{50}	=	Grain diameter corresponding to 50% passing, by weight (or mass), ft, mm
d_s	=	Depth of scour, ft
D	=	Jet plunge height, ft
D_b	=	Bedding layer thickness, ft
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_m = Momentum force, 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

h_j = Height of hydraulic jump, ft

h_L = Head loss, ft

h_s = Depth of scour, ft

h_v = Velocity head, ft

h_w = Height of weir, ft

H = Head on the weir or orifice, 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_w = Head on structure for weighted-creep ratio, ft

K_t = Trashrack loss coefficient (empirical)

L_a = Approach length, ft

L_b = Design basin length, ft

L_{bm} = Design basin length, main channel, ft

L_{bt} = Design basin length, trickle channel, ft

L_d = Drop length, ft

L_{dm} = Nappe length, main channel, ft

L_{dt} = Nappe length, trickle channel, 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

L_V = Vertical creep distance along any contact surfaces greater than 45 degrees, ft

M = Mass rate of flow, lb sec/ft

n = Manning roughness coefficient

P = Height of the weir crest above the approach channel, ft

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

r = Channel centerline radius of curvature, ft

R = Hydraulic radius, ft

S_o = Bed or drop slope (S is also used), ft/ft

T = Top width of flow in the channel, 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_g = Velocity between the bars of a trashrack, ft/sec

V_n = Velocity through the net trashrack area, ft/sec

W = Width, ft

W_o = Width of box culvert, diameter of pipe culvert, or span of pipe arch, ft

W_s = Width of scour, ft

y = Depth of flow, 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_m	=	Hydraulic depth, ft
y_n	=	Normal 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_o	=	Brink depth, 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
Z	=	Channel side slope horizontal distance per foot of drop, ft/ft
Z_f	=	Drop face slope, ft/ft

7.2 USE OF STRUCTURES IN DRAINAGE

Hydraulic structures are used in stormdrainage 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 stormdrains 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.

7.2.1 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.

7.2.2 Conduit Outlet Structures

Energy dissipation structures are necessary at the outlets of culverts or stormdrains 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 8 for a more detailed discussion pertaining to how these facilities are incorporated into stormwater basins.

7.2.3 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 tur-

bulent 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.

7.2.4 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.

7.2.5 Trashracks and Access Barriers

Trashracks serve two purposes when utilized in conjunction with stormdrains, 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 stormdrains, culverts, and detention basin outlets to prevent the public from entering the conduit. Access barriers are typically the same configuration as trashracks.

7.2.6 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)).

7.3 CHANNEL DROP STRUCTURES

7.3.1 General

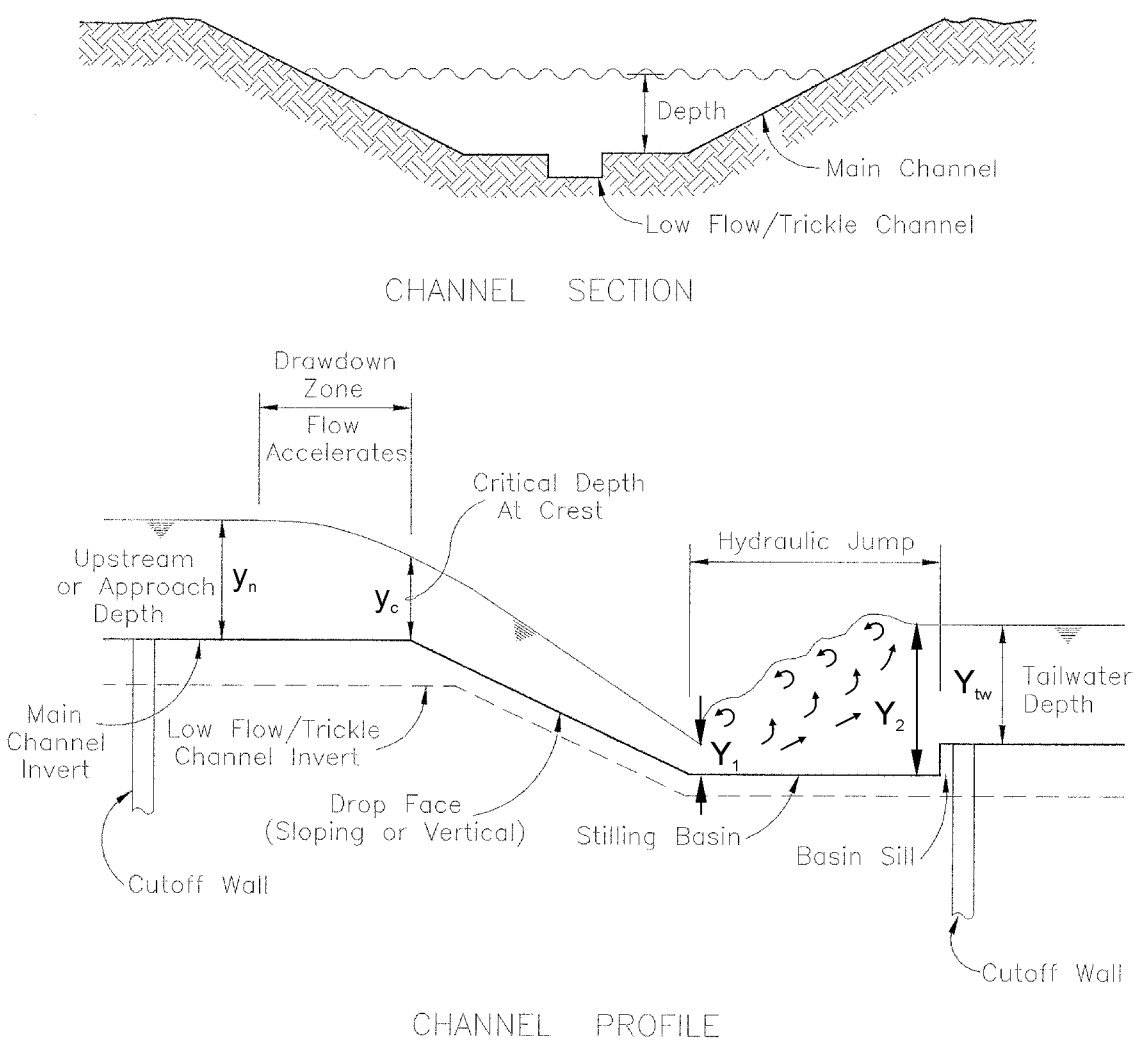
The term drop structure is broadly defined. Included are structures built to restore previously damaged channels, those constructed during new urban development to prevent accelerated erosion caused by increased runoff, and applications in which other specialized hydraulic conditions are created in the 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 7.1 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 7.1
TYPICAL DROP STRUCTURE COMPONENTS
(Adapted from McLaughlin Water Engineers, Ltd. 1986)



Design Considerations

In addition to hydraulic performance (discussed in Section 7.3.2), 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.

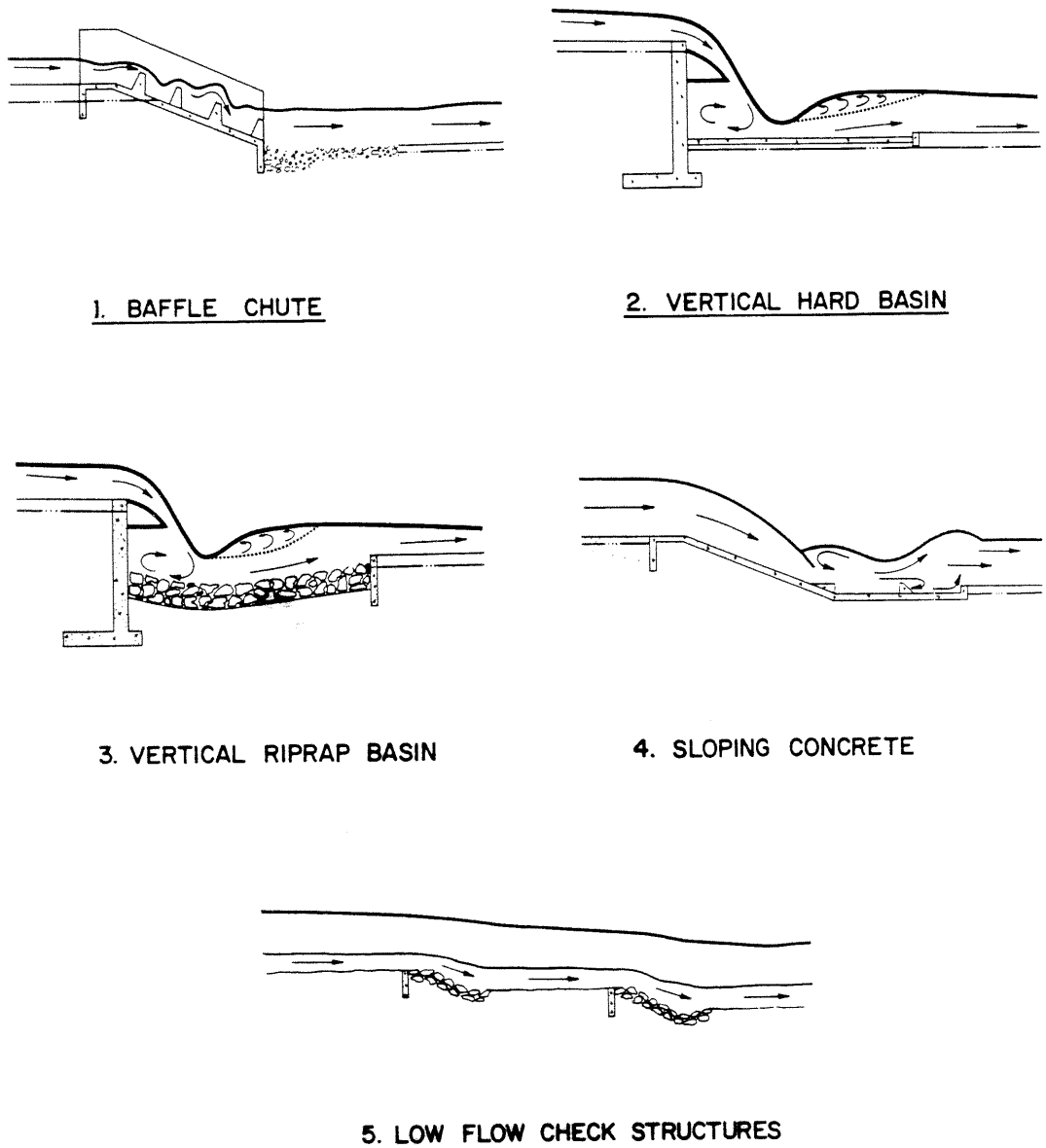
Sociological Considerations - These include public acceptability issues such as safety (Section 7.7), visual appearance (Section 7.8), mosquito breeding in ponded water, etc.

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 7.2 shows schematic profiles of each type.

FIGURE 7.2
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 not permitted.

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 7.3.3 presents considerations for the selection of the appropriate type of drop structure for particular application or site conditions.

7.3.2 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 7.3.3).

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.

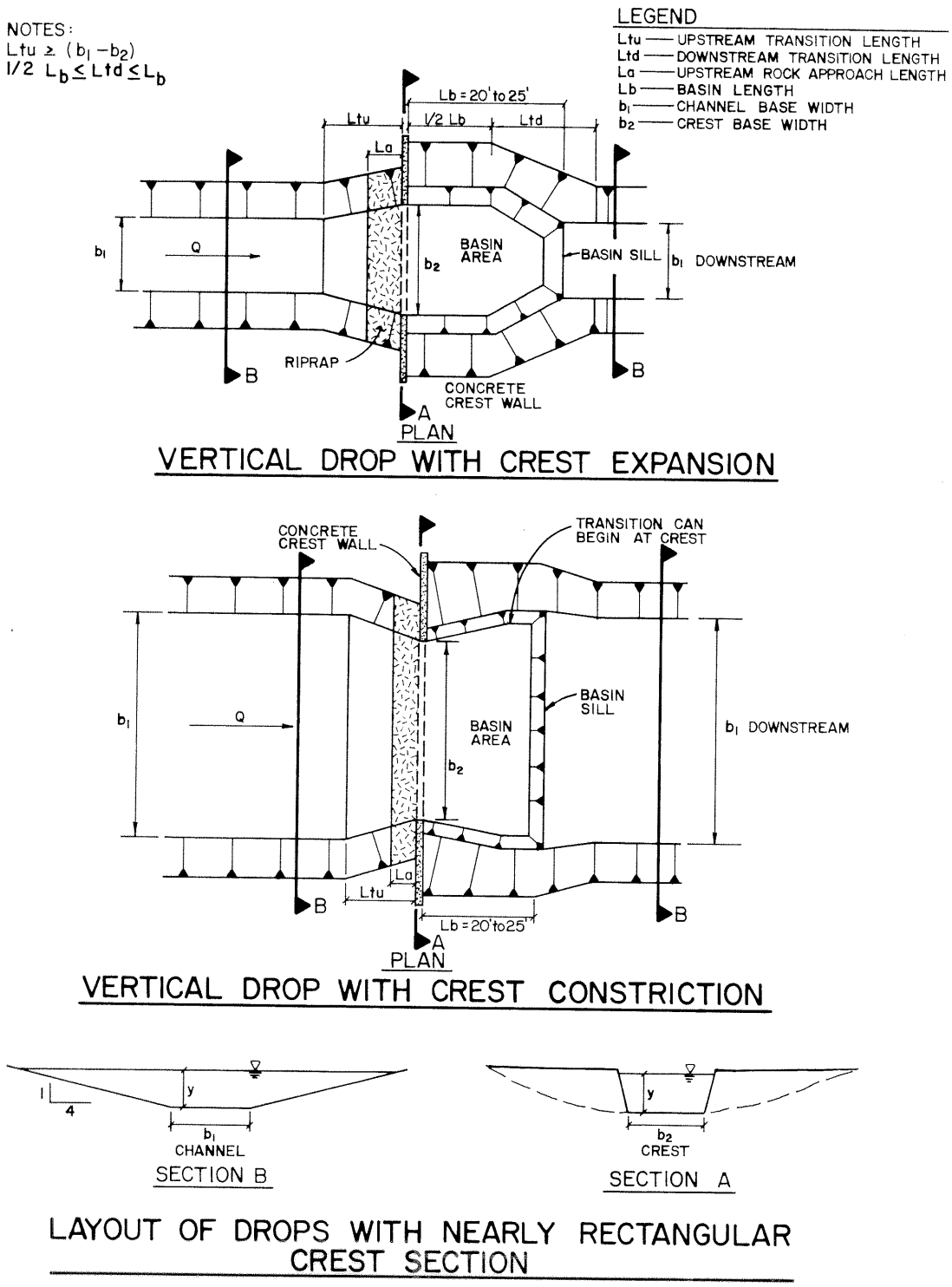
Vertical or Near Vertical Abutments at Drop Crest - Figure 7.3 presents alternative drop crests at a vertical drop structure. In general, the objectives of upstream hydraulics and crest design are:

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.

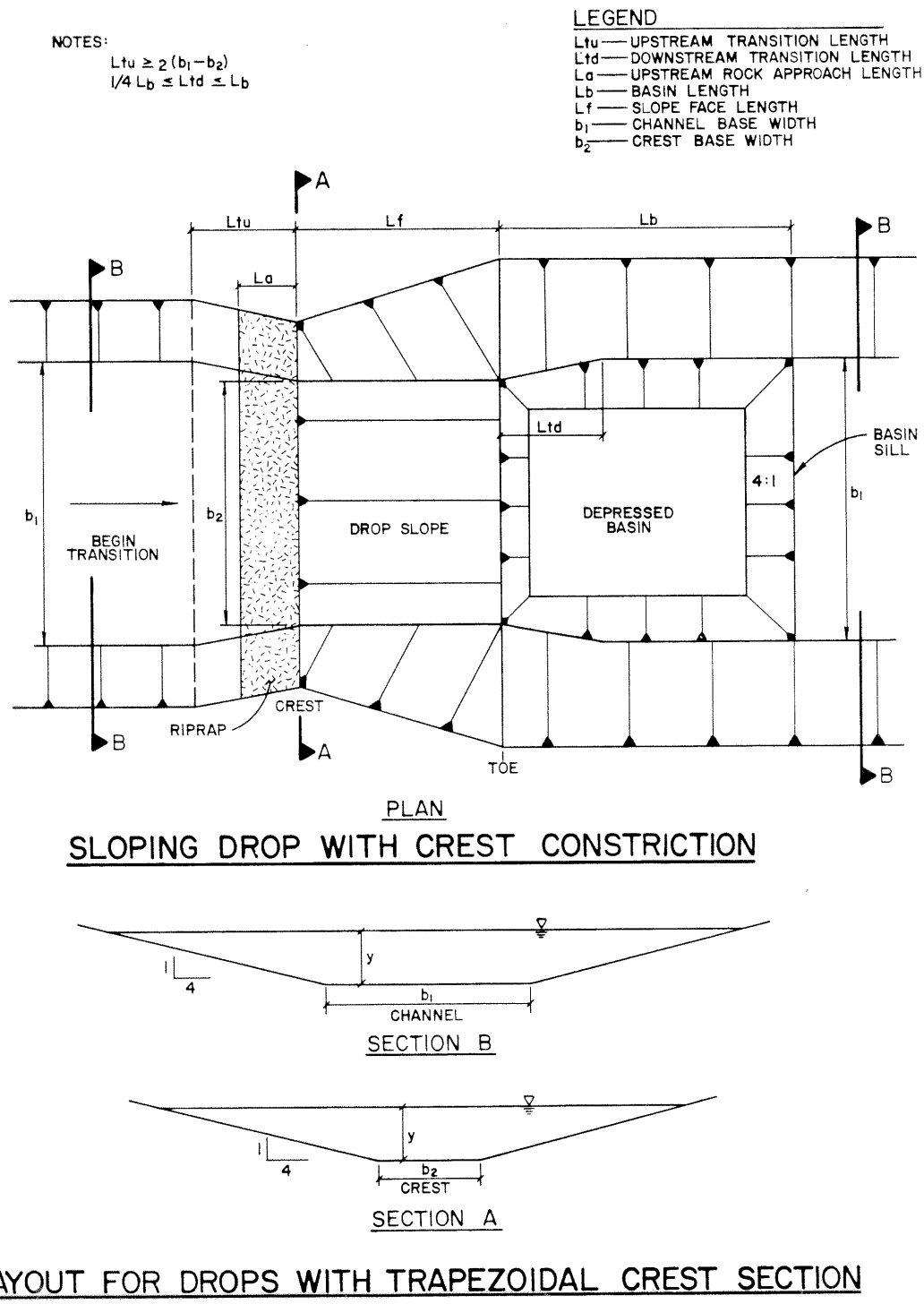
FIGURE 7.3
TYPICAL VERTICAL DROP CREST CONFIGURATION
(McLaughlin Water Engineers, Ltd. 1986)



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 7.4 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 7.4
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 super-critical 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 (7.1) 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.

$$F_s = \frac{q^2}{g\,y} + \frac{y^2}{2} \tag{7.1}$$

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.

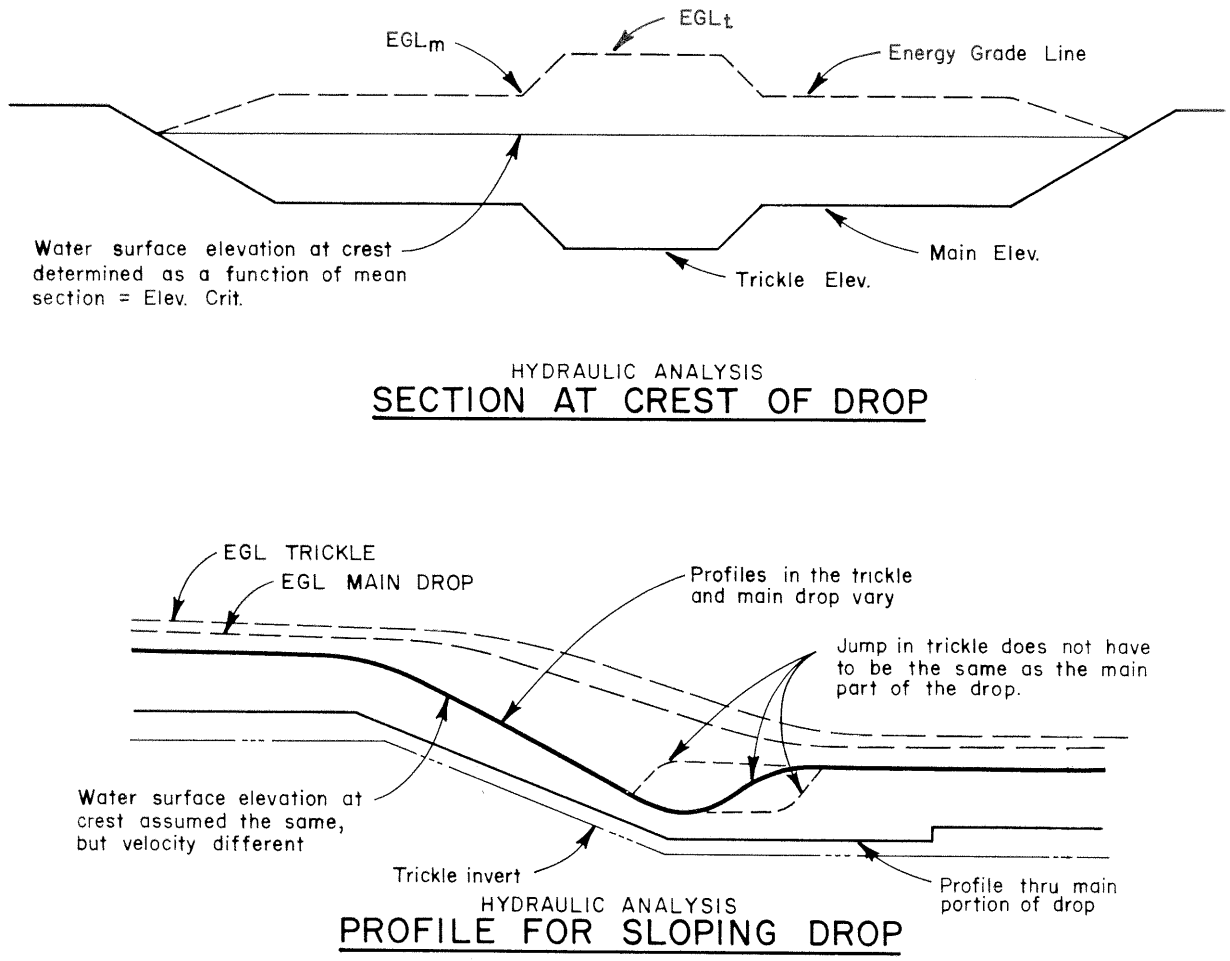
Ideally, for economic considerations, the jump should begin no further downstream than the drop toe. This is generally accomplished in the main drop zone by depressing the basin to a depth nearly as low as the downstream trickle channel elevation.

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 pro-

file in the low flow channel zone has a higher energy grade line profile (Figure 7.5). 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 7.5
TYPICAL SECTION AND PROFILE FOR SLOPING DROP
(McLaughlin Water Engineers, Ltd. 1986)



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 7.1.

TABLE 7.1
TYPES OF HYDRAULIC JUMPS

Upstream Froude Number	Type of Jump	Energy Loss, %
$1.0 < F_{r1} \leq 1.7$	Undular Jump	0 to 5
$1.7 < F_{r1} \leq 2.5$	Weak Jump	5 to 18
$2.5 < F_{r1} \leq 4.5$	Oscillating Jump	18 to 44
$4.5 < F_{r1} \leq 9.0$	Steady Jump	44 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:

$$Y_2 = \frac{1}{2} Y_1 \left[(1 + 8F_{r1}^2)^{\frac{1}{2}} - 1 \right] \tag{7.2}$$

The solution for sequent depth in trapezoidal channels can be obtained from a trial-and-error solution of Equation (7.3), which is derived from momentum equations. It is also acceptable for design purposes to determine the sequent depth in trapezoidal channels from Equation (7.2). Equation (7.2) is much simpler to solve and produces only slightly greater values for sequent depth than does Equation (7.3).

$$\frac{ZY_1^3}{3}+\frac{ZY_1^2}{2}+\frac{Q}{gA_1}=\frac{ZY_2^3}{3}+\frac{bY_2^2}{3}+\frac{Q}{gA_2} \tag{7.3}$$

Figure 7.6 and Figure 7.8 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 (7.3). Therefore, the height of this wave should be determined as follows:

$$\frac{Y_2-Y_1}{Y_1} = F_{r1}^2-1 \tag{7.4}$$

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 7.7 and Figure 7.9.

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 7.10.

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 (7.3). 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 (7.1) 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.

Design Charts and Figures

Figure 7.6 to Figure 7.10 and Table 7.2 have been included as additional aids to the user of this manual.

FIGURE 7.6
HEIGHT OF A HYDRAULIC JUMP FOR A HORIZONTAL RECTANGULAR CHANNEL
(USDOT, FHWA, HEC-14, 1983)

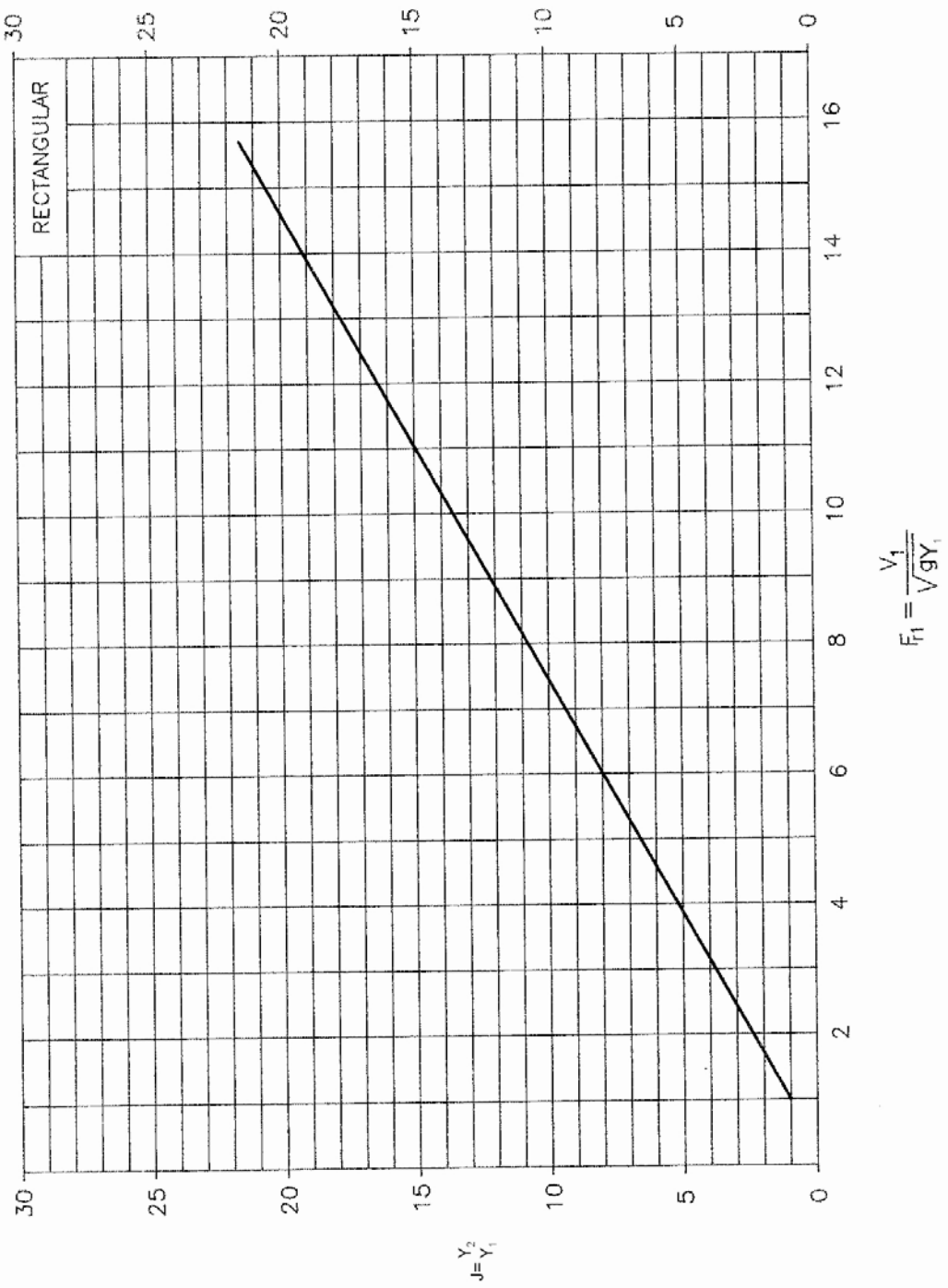


FIGURE 7.7
LENGTH OF HYDRAULIC JUMP FOR RECTANGULAR CHANNELS
(USDOT, FHWA, HEC-14, 1983)

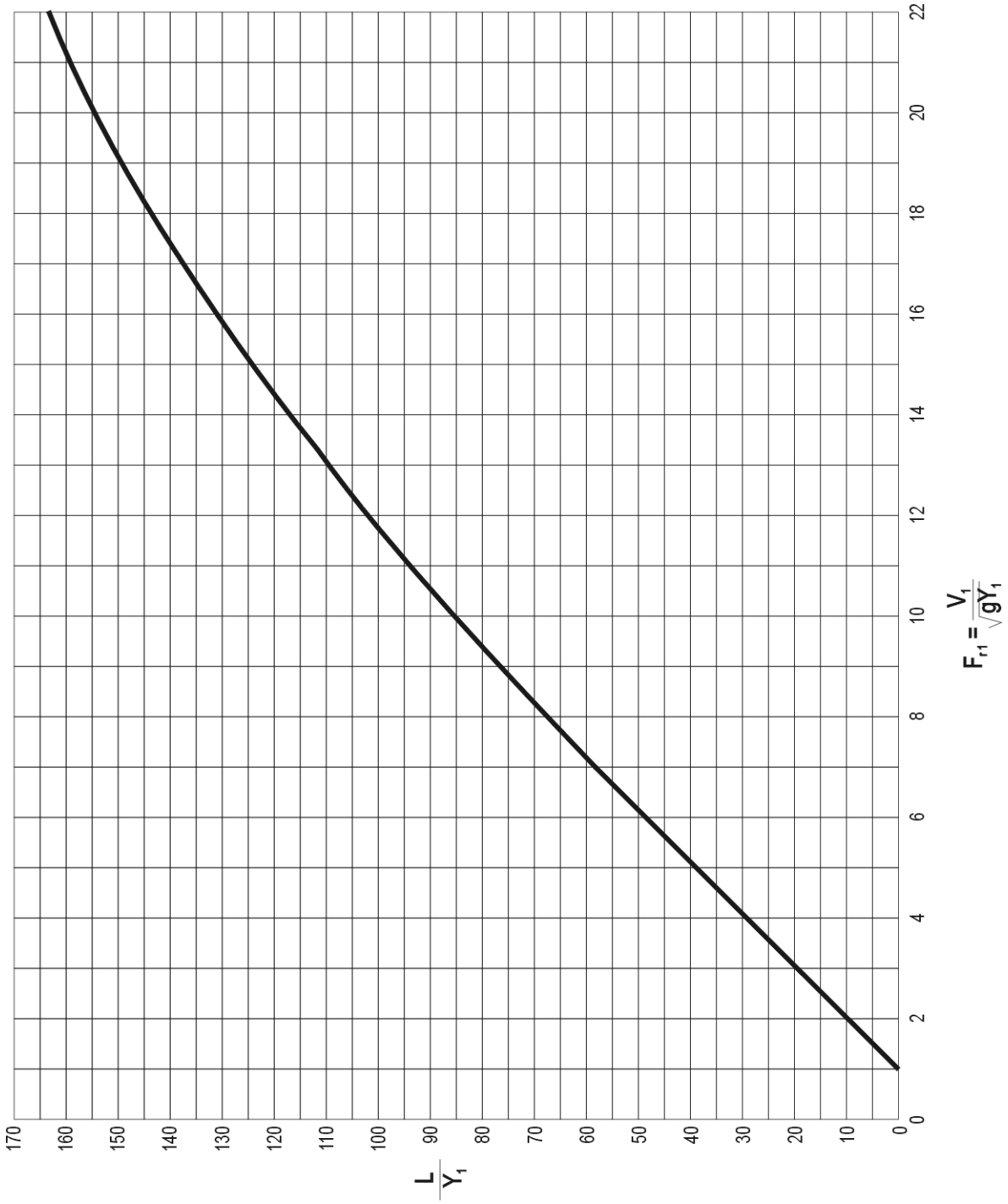


FIGURE 7.8
HEIGHT OF A HYDRAULIC JUMP FOR A HORIZONTAL TRAPEZOIDAL CHANNEL
(USING HYDRAULIC DEPTH)
(USDOT, FHWA, HEC-14, 1983)

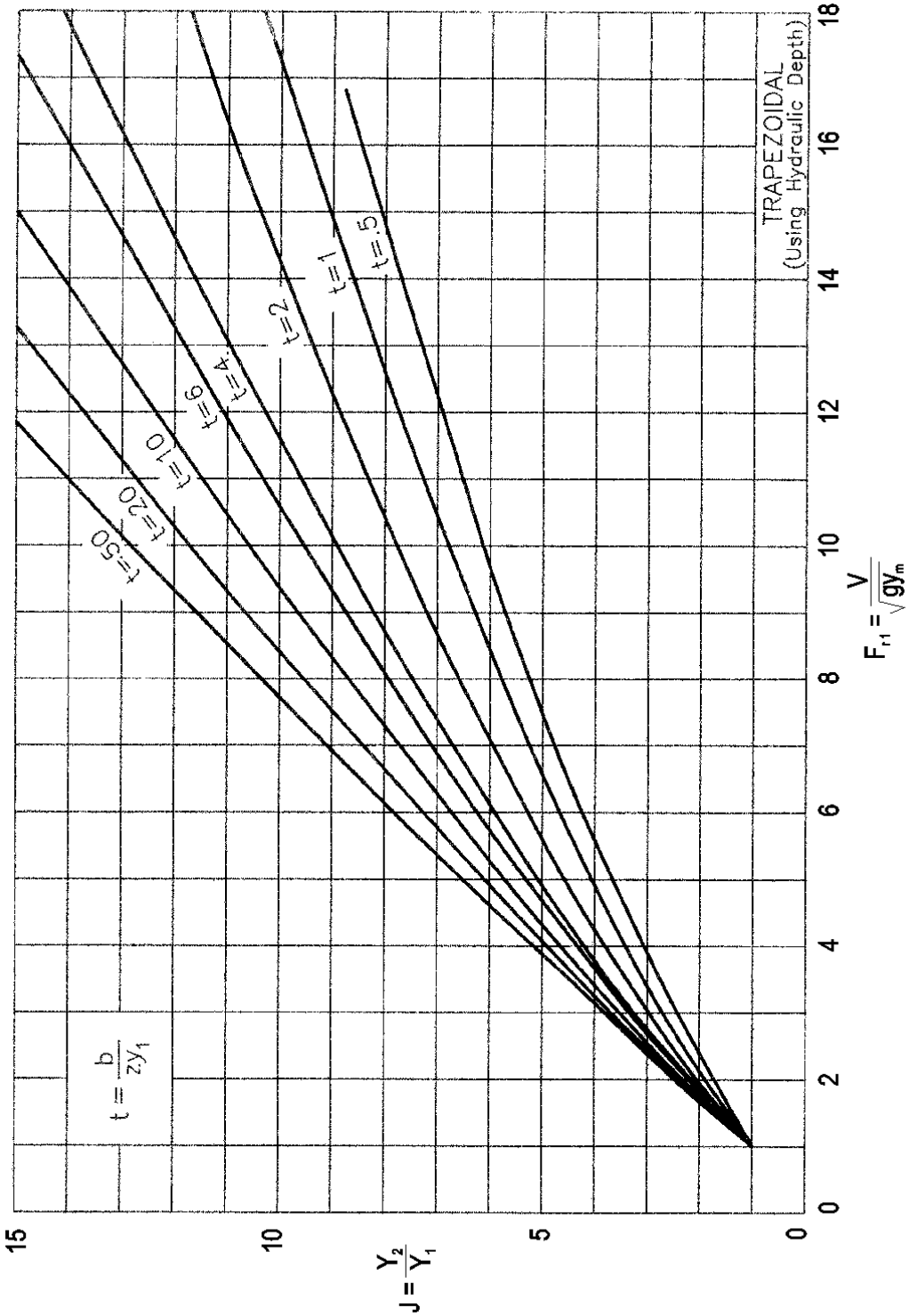


FIGURE 7.9
LENGTH OF A HYDRAULIC JUMP FOR NON-RECTANGULAR CHANNELS
(USDOT, FHWA, HEC-14, 1983)

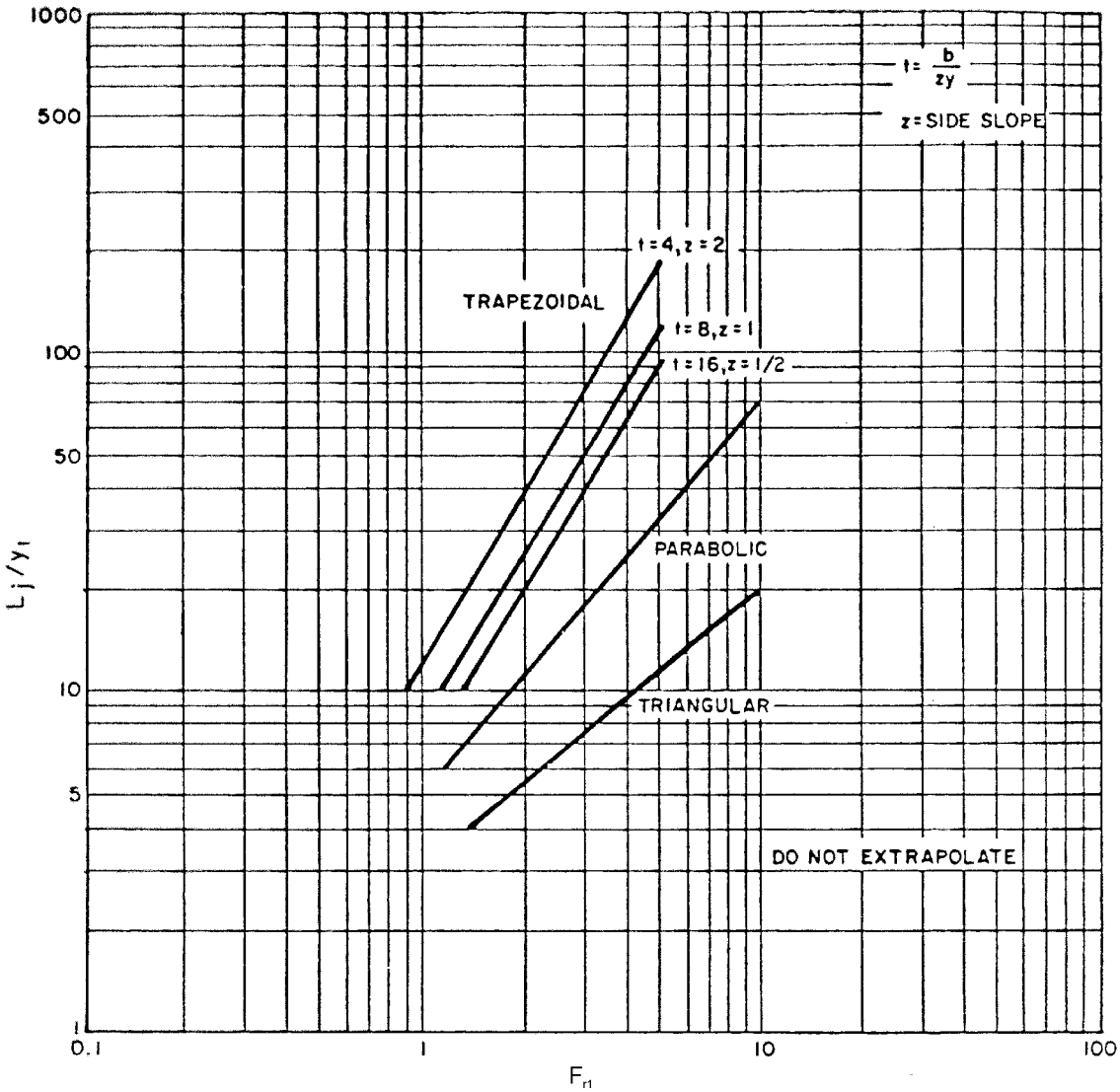


FIGURE 7.10
SURFACE PROFILE OF A HYDRAULIC JUMP IN A HORIZONTAL CHANNEL
(Chow, 1959)

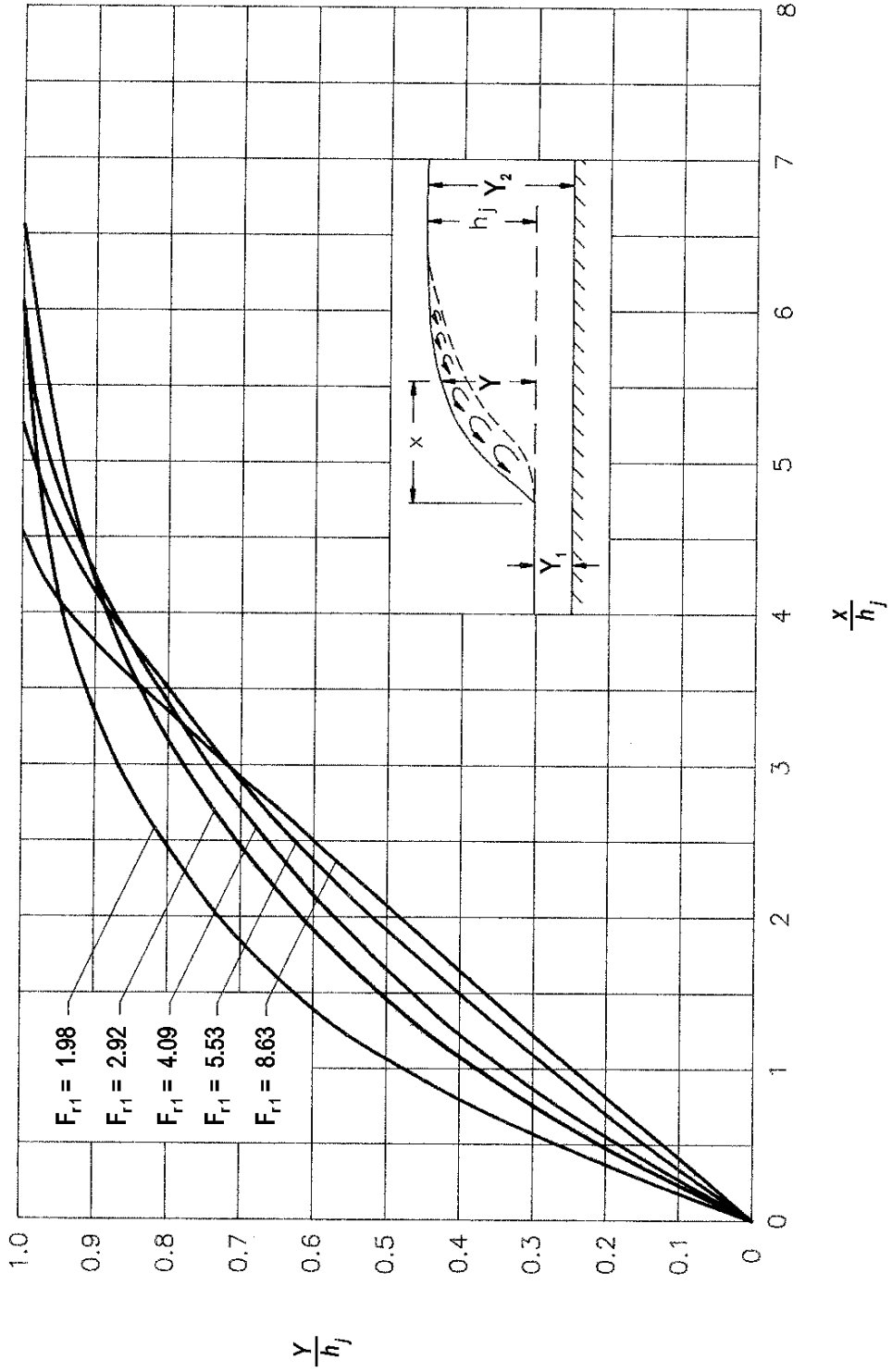


TABLE 7.2
UNIFORM FLOW IN CIRCULAR SECTIONS FLOWING PARTLY FULL
(USDOT, FHWA, HEC-14, 1983)

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:

$$C_w = \frac{(L_H + 3L_v)}{3H_{df}} \tag{7.5}$$

3. Lane's recommended weighted-creep ratios are given for various foundation materials in Table 7.3.

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 lowered 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, 1987). 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 7.3
LANE'S WEIGHTED-CREEP: RECOMMENDED RATIOS

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

7.3.3 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 7.3.2. Additional guidelines are also contained in Section 7.3.4.

Foundation and Seepage Control Systems

Table 7.4 presents some typical foundation conditions and control systems typically used for various drop heights. Table 7.4 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 7.4
GENERAL SEEPAGE CUTOFF TECHNIQUE SUITABILITY

Soil Conditions	Drop Height, feet			
	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 7.5 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 7.5
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 7.5 (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. Synthetic fibers can be added to the concrete mix, to provide additional crack control 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 Cutoff 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 7.5 (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.

7.3.4 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.

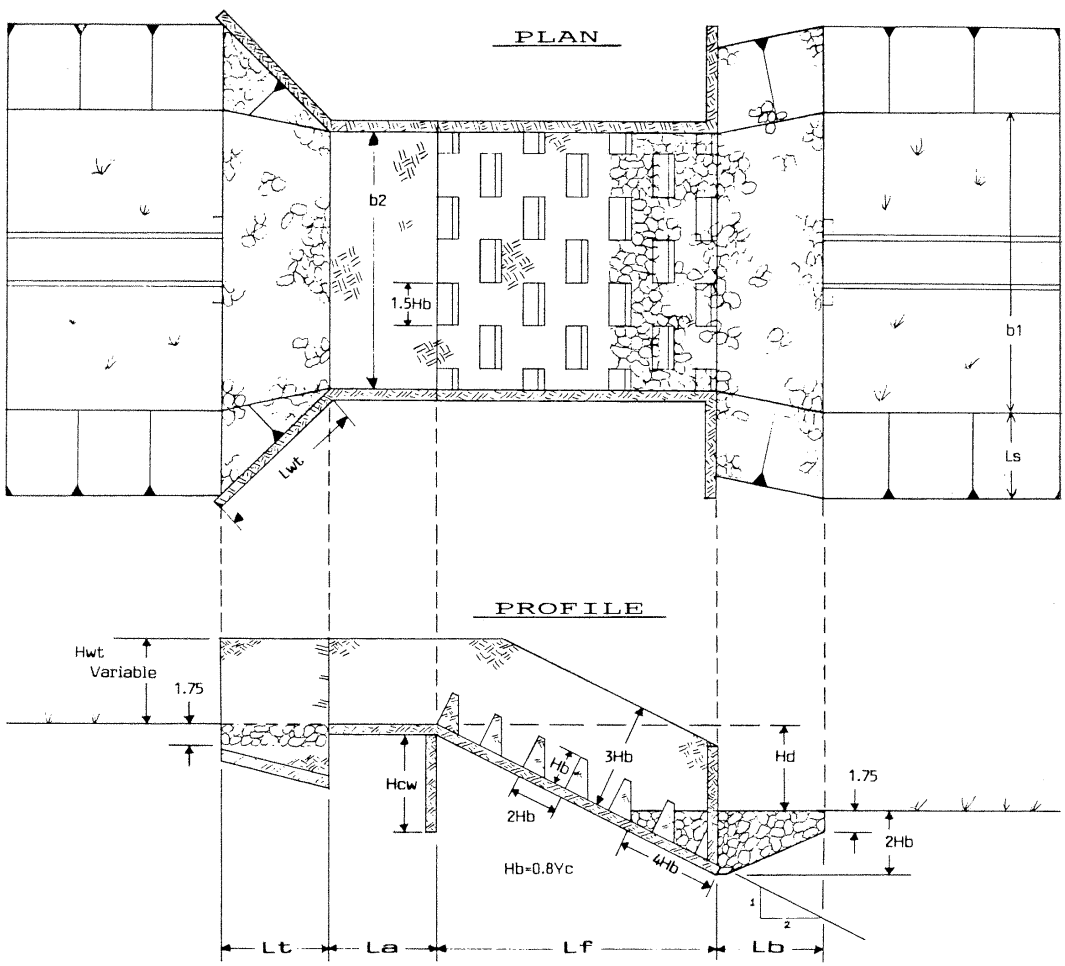
The optimal performance occurs for a unit flow (q) at the chute width of 35 to 60 cfs/ft. Model testing has evaluated discharges up to 300 cfs/ft, and there have been structures built with up to 120 cfs/ft. The USBR states that the recommended design flow of 60 cfs/ft for baffle chute drops has been 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 7.11). 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 7.11
BAFFLE CHUTE DROP
(McLaughlin Water Engineers, Ltd., 1986)



Lt = Transition Length
La = Apron Length
Lf = Face Length
Lb = Basin Rock Length
Lwt = Wing Wall Length
Hwt = Wing Wall Height
Hcw = Cutoff Wall Height
Hd = Height of Drop
Hb = Baffle Height = 0.8 x Critical Depth

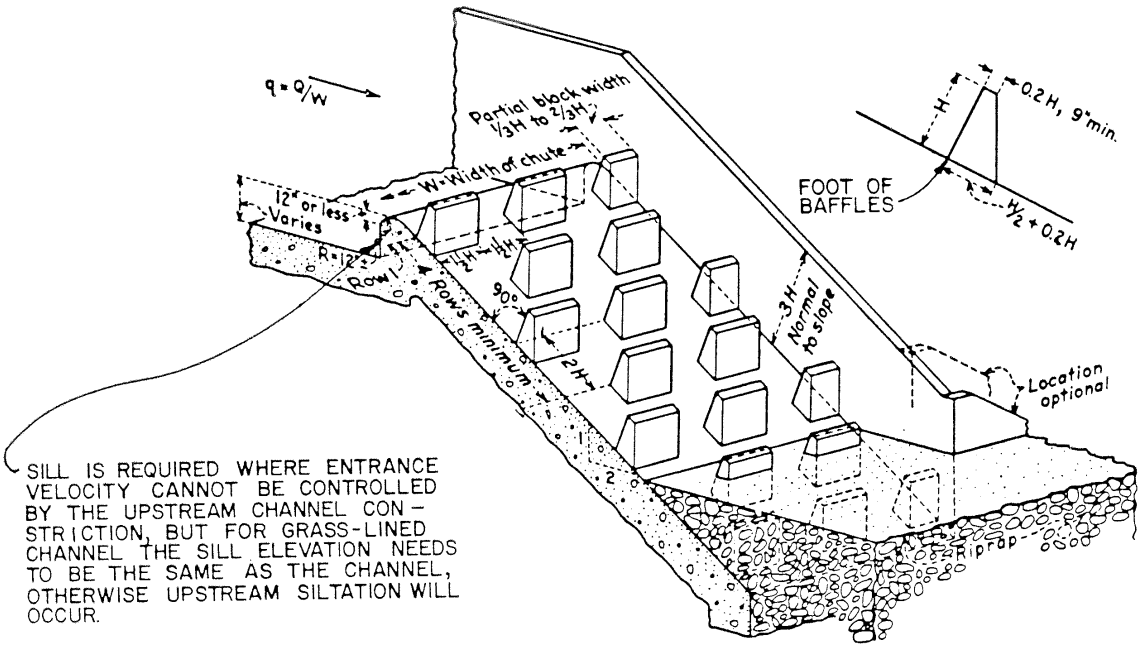
There are fixed costs associated with the upstream wing walls, crest approach section, downstream transition walls and a minimum length of sloping apron (for four baffle rows). Consequently, the baffle chute becomes more economical with increasing drop height.

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.

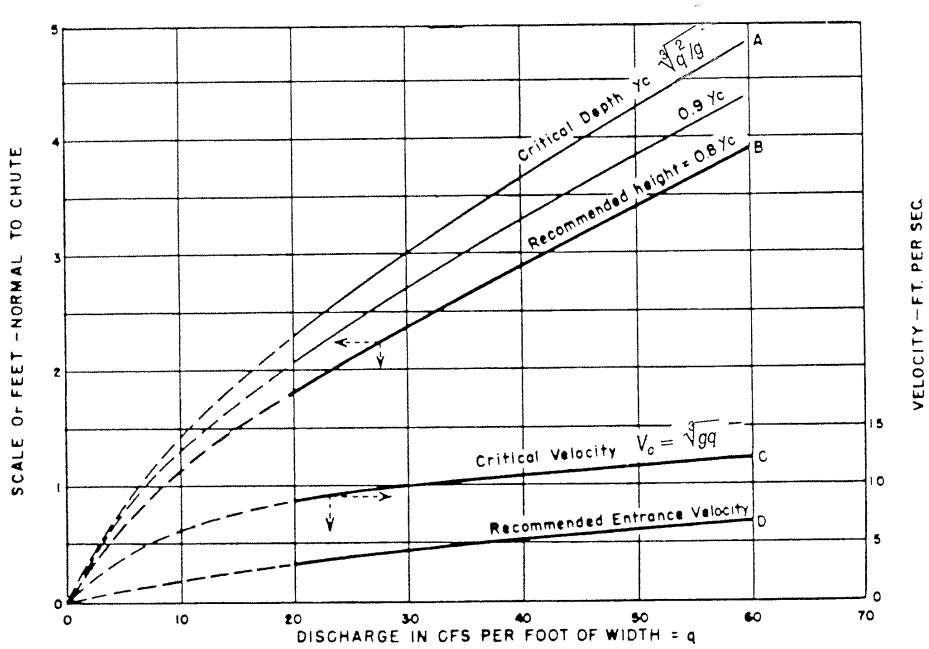
The potential for debris flow must also be considered. Use caution when conditions include streams with heavy debris flow, because the baffles can become clogged between the interstices, resulting in overflow, low energy dissipation, and direct impingement of the erosive stream jet on the downstream 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 7.12 and Figure 7.13. Remaining structural design parameters must be determined for specific site conditions. The recommended design procedures are discussed on the following pages.

FIGURE 7.12
BAFFLE CHUTE DESIGN CRITERIA
(Adapted from: Peterka 1984)

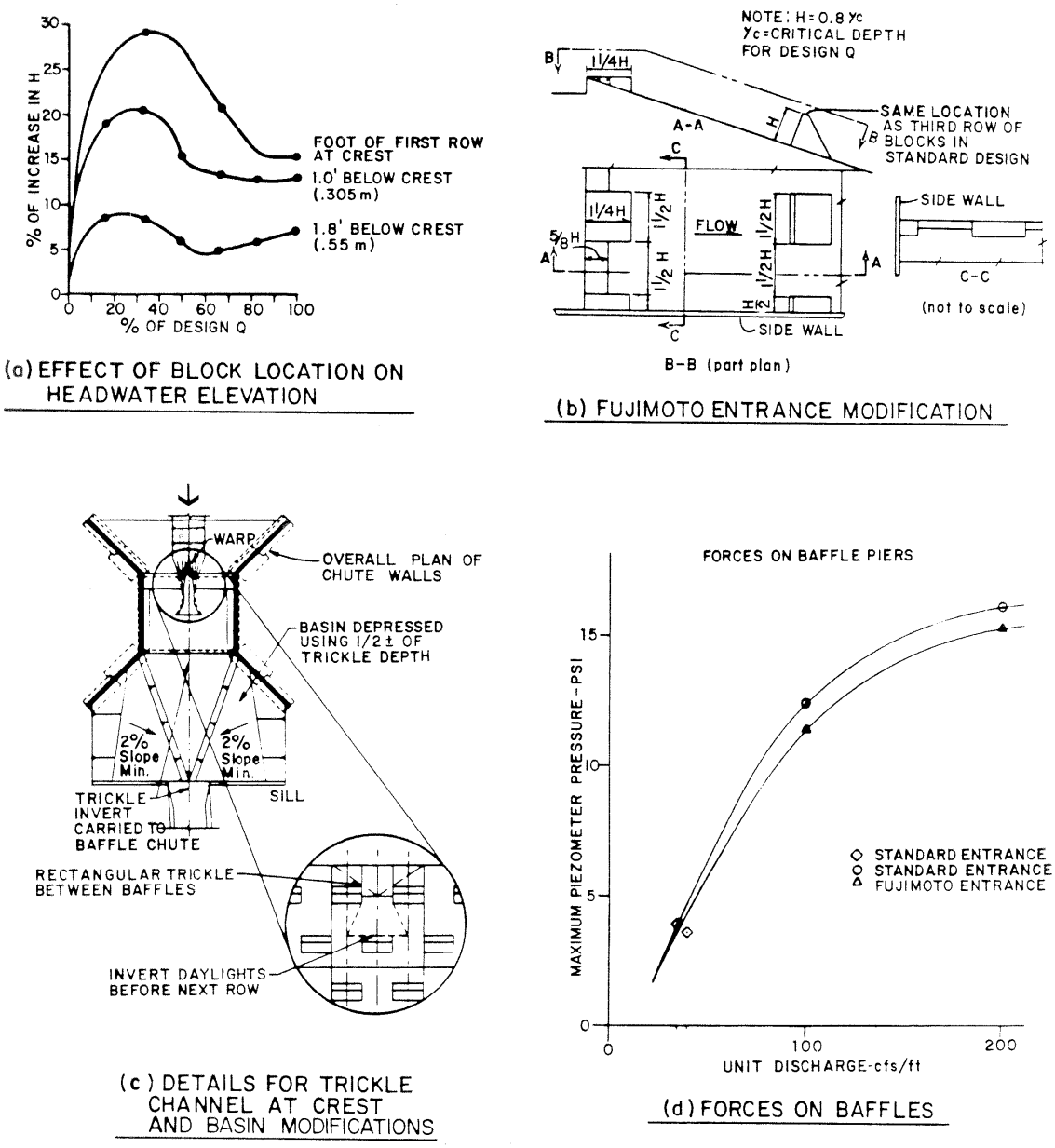


(A) USBR ISOMETRIC



(B) DESIGN CRITERIA

FIGURE 7.13
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:

$$Q = Q / W \tag{7.6}$$

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 7.12(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 7.12(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 7.13(c) illustrates one method of designing the low flow channel through the crest.

2. The conventional design shown in Figure 7.12(a) results in the top elevation of the baffles being higher than the crest, which causes a higher backwater surface effect upstream. Figure 7.12(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 7.13(b). The upper rows of baffles are moved one row increment downstream. The important advantage of this entrance is that there is no backwater 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:

$$y_c = (q^2/g)^{0.33} \tag{7.7}$$

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 7.12(b) may be extrapolated:

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 7.3.2).

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 7.13(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.

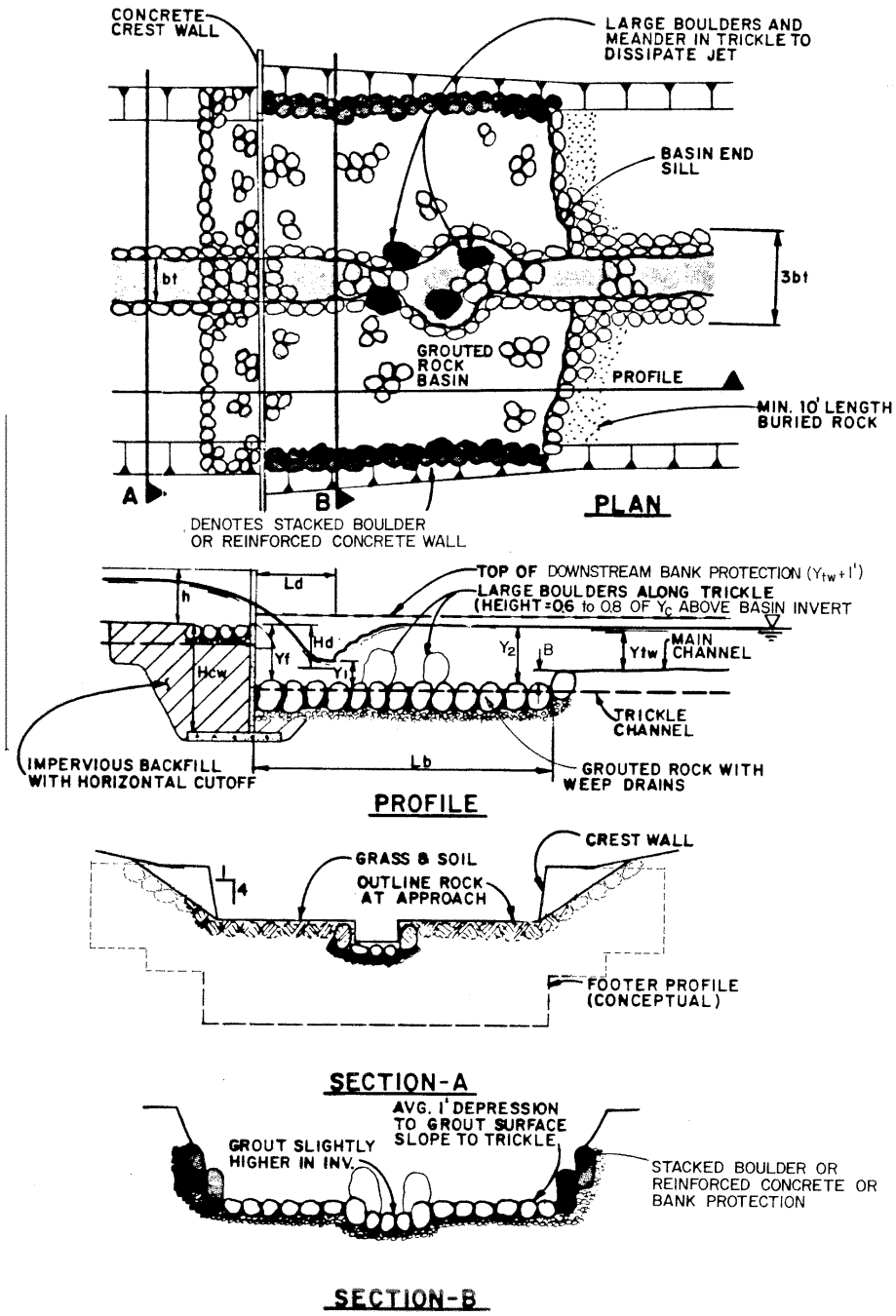
Vertical Hard Basin Drops

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 7.14 shows a vertical drop with a grouted boulder basin (concrete may also be used), and illustrates several important design considerations.

FIGURE 7.14
VERTICAL HARD BASIN DROP
(McLaughlin Water Engineers, Ltd., 1986)



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 7.3.2)

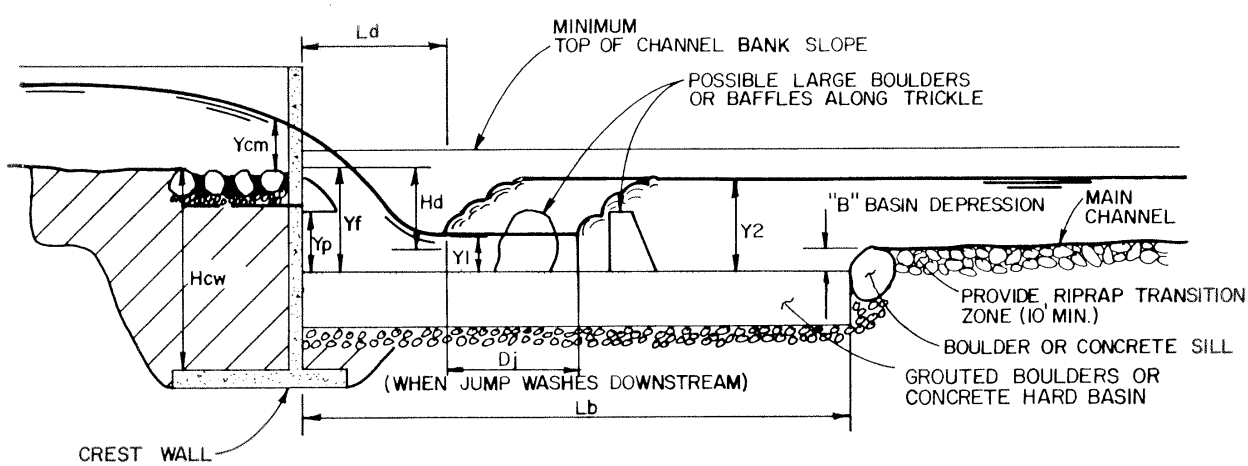
3. Crest wall and footer dimensions are determined by conventional structural methods. Underdrain requirements are determined from seepage analysis.

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 7.3.2. 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 7.15 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:

$$L_b \geq L_d + D_j + 9.6\ Y_2 \tag{7.8}$$

FIGURE 7.15
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:

$$L_d / Y_f = 4.3\, D_n^{0.27} \tag{7.9}$$

where:

$$D_n = q_c^2 / (gY_f^3) \tag{7.10}$$

$$Y_p / Y_f = 1.0\, D_n^{0.22} \tag{7.11}$$

$$Y_1 / Y_f = 0.54\, D_n^{0.425} \tag{7.12}$$

$$Y_2 / Y_f = 1.66\, D_n^{0.27} \tag{7.13}$$

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:

   $$L_{bm} = L_{dm} + D_{jm} + 0.60\,(6Y_2)_m \tag{7.14}$$

   b. At the trickle zone, standard design:

   $$L_{bt} = L_{dt} + D_{jt} + 0.60\,(6Y_2)_t \tag{7.15}$$

   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:

   $$L_{bt} = L_{dt} + 0.60\,(6Y_2)_t \tag{7.16}$$

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 7.16, 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.

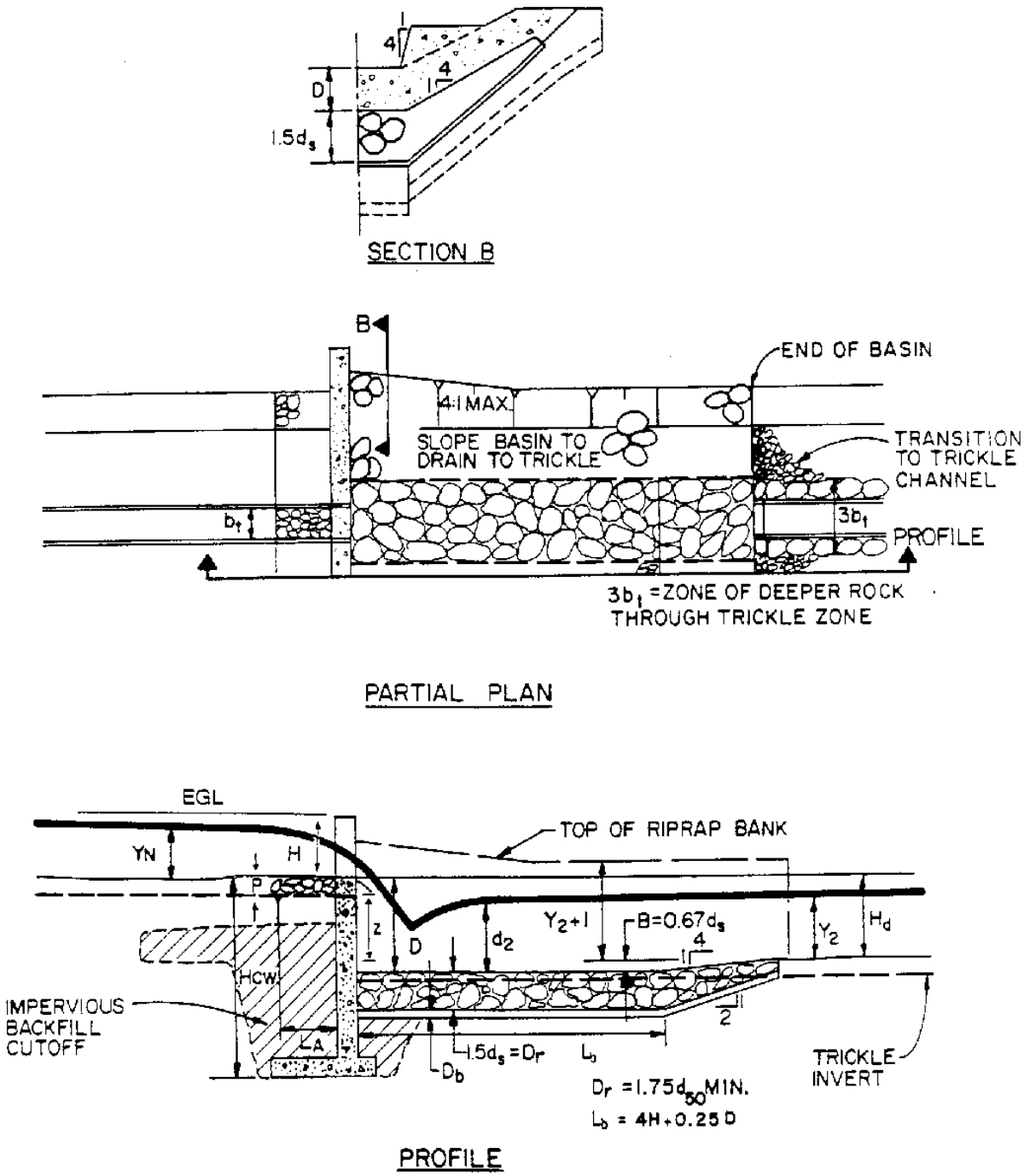
General Hydraulic Design Procedure:

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 7.16 that:

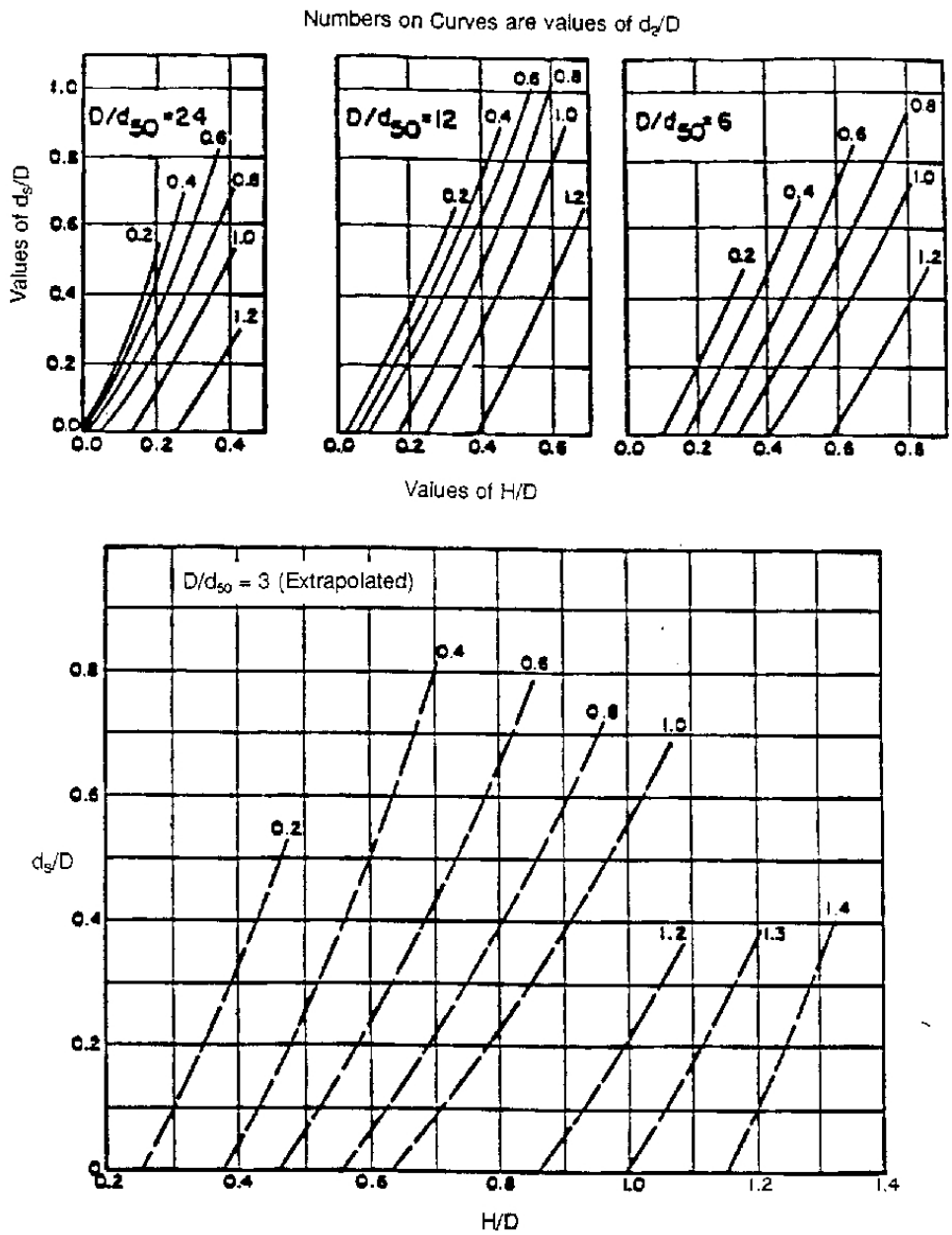
$$H_d = D - 0.67\ d_s \tag{7.17}$$

FIGURE 7.16
VERTICAL RIPRAP BASIN DROP
(Stevens, 1981)



The designer must find the combination of rock size and jet plunge height D that gives a depth of scour which balances Equation (7.16). 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 7.17. As these values will be different in the main drop and the trickle, the design d_{50} and/or d_s will vary.

FIGURE 7.17
CURVES FOR SCOUR DEPTH AT VERTICAL DROP
(Stevens, 1981)



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.

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

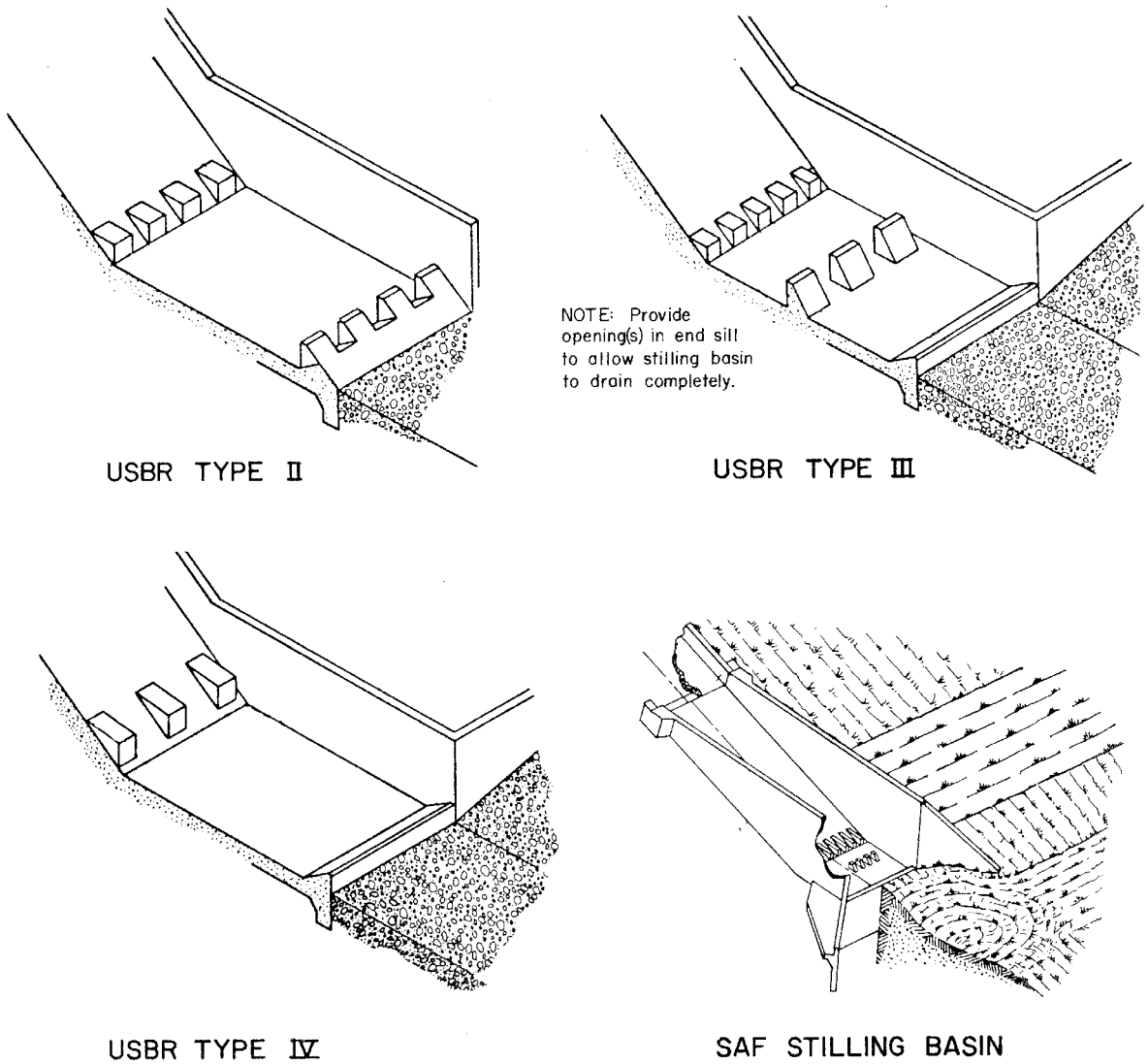
The hydraulic concept of these structures is to dissipate energy by formation of a conventional hydraulic jump, usually associated with a reverse current surface flow as the supercritical flow down the face converts to subcritical flow downstream.

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 7.18 illustrates the various types of stilling basins for use with sloping concrete drops.

FIGURE 7.18
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 7.3.2. 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 7.3.2), 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

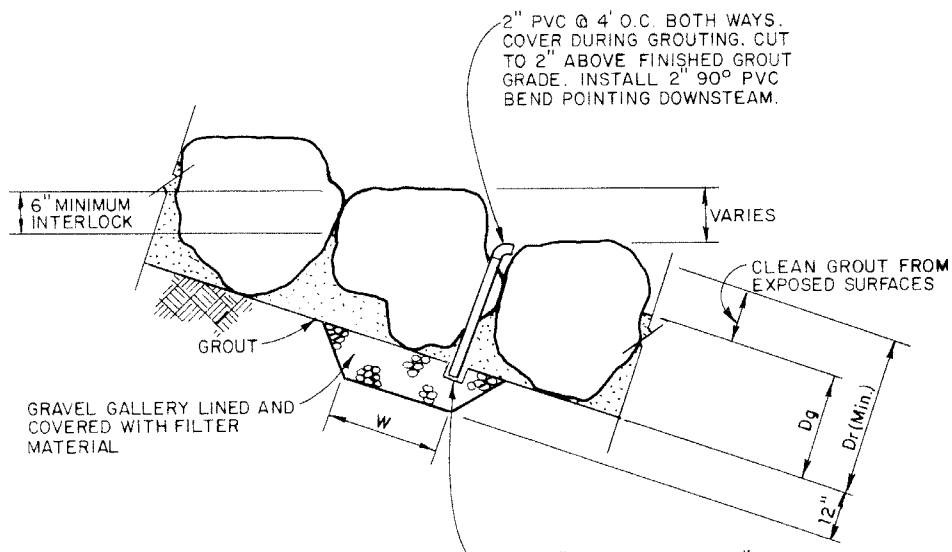
There are numerous other types of drop structures for specific applications in drainage design. The four types of structures presented above are appropriate for the majority of situations to be encountered in Maricopa County. Some possible variations or modifications are presented below along with a 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. Ade-

quate 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 7.19).

FIGURE 7.19
GROUTED BOULDER PLACEMENT
(McLaughlin Water Engineers, Ltd.,1986)



NOTES:

1) CLEAN ALL DIRT FROM VOIDS BEFORE GROUTING.
2) PLACE BOULDERS WITH FLATTEST SURFACE ON TOP AND HORIZONTAL. PLACE BOULDERS IN INTERLOCKING FASHION, TOUCHING ADJACENT BOULDERS AND MINIMIZING VOIDS.
3) DEPTH OF ROCK (Dr) AND DEPTH OF GROUT (Dg) AS CALLED OUT ON PLANS.
4) GROUT TO PENETRATE FULL DEPTH OF ROCK AND FILL ALL VOIDS.
5) CLEAN ALL GROUT FROM EXPOSED SURFACES.
6) GRAVEL GALLERIES ARE CALLED OUT ON PLANS AS "W" (width of gallery) GRAVEL GALLERY (ies) "S" (spacing between galleries) O.C.

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. Addition of synthetic fiber reinforcement is also recommended to provide crack control, increased durability, and increased abrasion resistance.

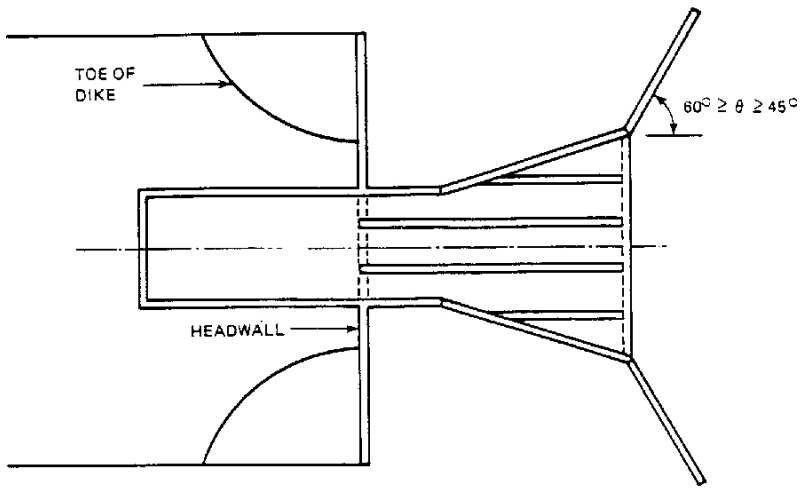
Other USBR Basins - Some other stilling basins developed by the USBR (Peterka, 1984) have limited application. For example, Basin I is basically a horizontal concrete apron downstream of a sloping or vertical drop. This type of basin is applicable only to a concrete lined channel, and, as the USBR states, 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.

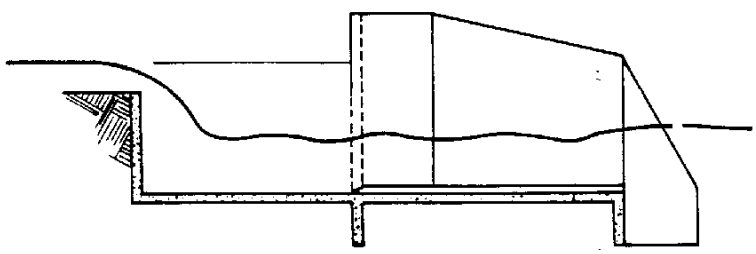
Box Inlet Drop Structure - The box inlet drop structure may be described as a rectangular box open at the top and downstream end (Figure 7.20). Water is directed to the crest of the box inlet by earth dikes and headwalls. Flow enters over the upstream end and two sides and leaves the structure through 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 7.20
BOX INLET DROP STRUCTURE
(Adapted from: FHWA, HEC-14, 1983)



Plan



Profile

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 Chapters 5 and 10 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.

7.4 ENERGY DISSIPATION STRUCTURES

7.4.1 General

Concrete energy dissipation or stilling basin structures are required to prevent scour damages caused by high exit velocities and flow expansion turbulence at conduit outlets. Outlet structures can provide a high degree of energy dissipation and are generally effective even with relatively low tailwater control. Rock protection at conduit outlets is appropriate where moderate outlet conditions exist; however, there are many situations where rock basins are impractical even at low to moderate flow conditions. 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.

7.4.2 Riprap Protection at Conduit Outlets

General Operating Characteristics

A stilling basin constructed of loose, graded riprap can be an effective and economical energy dissipation measure for a conduit outlet. *Hydraulic Design of Energy Dissipators for Culverts and Channels* (USDOT, 1983), contains a design procedure for riprap energy dissipators based on studies conducted at Colorado State University and sponsored by the Wyoming Highway Department. The following conclusions were drawn from an analysis of the experimental data and observed operating characteristics.

- The depth (h_s), length (L_s), and width (W_s) of the scour hole were related to the characteristic size of riprap (d_{50}), discharge (Q), brink depth (Y_o), and tailwater depth (TW).

- The dimensions of a scour hole in a basin constructed with angular rock were approximately the same as those of a scour hole in a basin constructed of rounded material when rock size and other variables were similar.

- When the ratio of tailwater depth to brink depth (TW/Y_o) was less than 0.75 and the ratio of scour depth to size of riprap (h_s/d_{50}) was greater than 2.0, the scour hole functioned very efficiently as an energy dissipator. As the concentrated flow at the culvert brink plunged into the hole, a jump formed against the downstream extremity of the scour hole, and the flow was generally well dispersed as it left the basin.

- The mound of material which formed on the bed downstream of the scour hole contributed to the dissipation of energy and reduced the size of the scour hole; i.e., if the mound from a stable scoured basin was removed and the basin was again subjected to design flow, the scour hole enlarged somewhat.

- For high tailwater basins (TW/Y_o> greater than 0.75) the high velocity core of water emerging from the culvert retained its jet-like character as it passed through the basin, and diffused in a manner very similar to that of a concentrated jet diffusing in a large body of water. As a result, the scour hole was much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rock-lined basin.

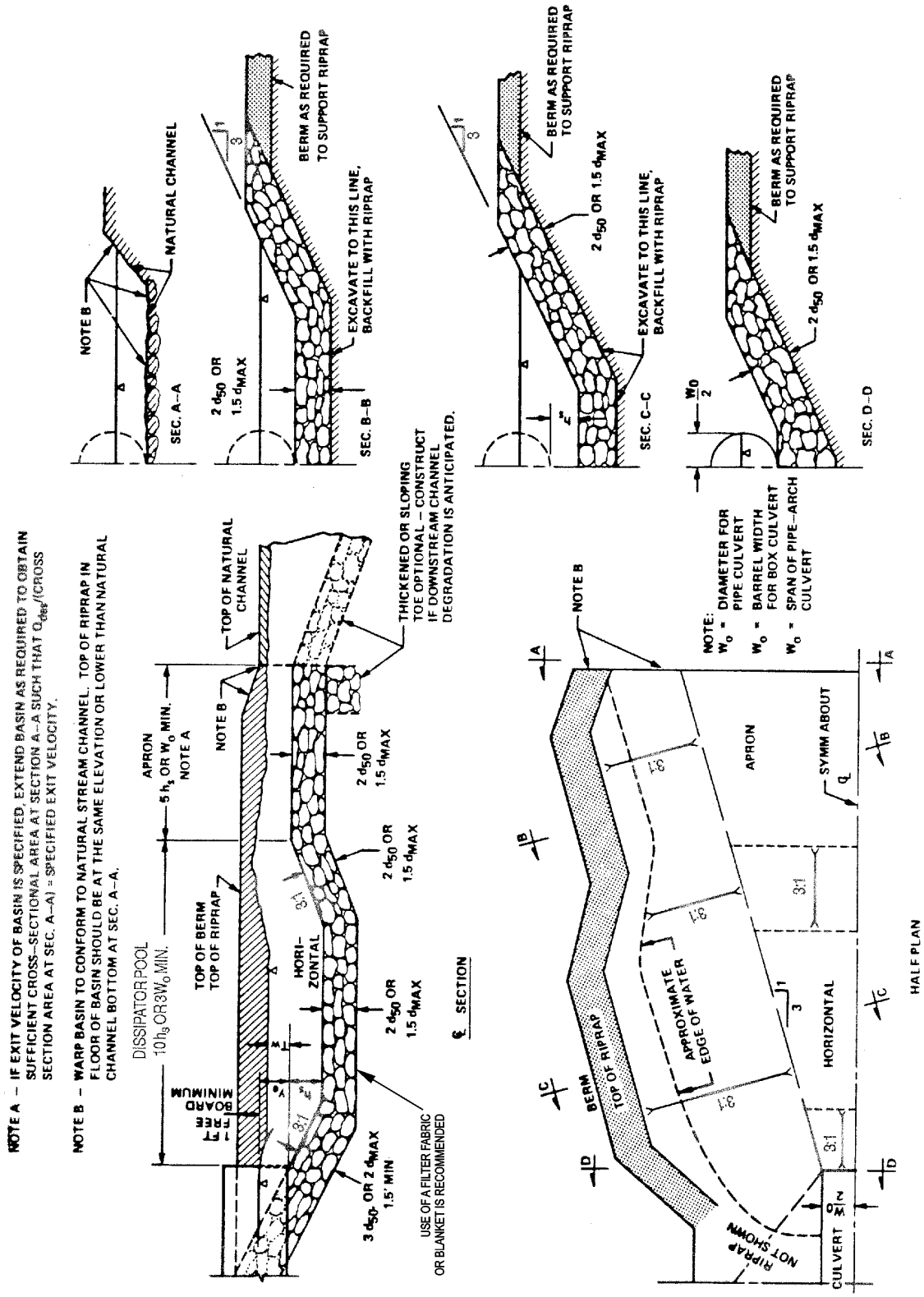
General details of the basin are shown in Figure 7.21, and the principal features are:

- The basin is preshaped and lined with riprap of median size d_{50}.

- The surface of the riprapped floor of the energy dissipating pool is constructed at an elevation, h_s, below the culvert invert. Elevation h_s is the approximate depth of scour that would occur in a thick pad of riprap of size d_{50} constructed at the outfall of the culvert if subjected to design discharge. The ratio of h_s to d_{50} of the material should be between 2 and 4.

- The length of the energy dissipating pool, L_s, is $10h_s$ or $3W_o$ whichever is larger. The overall length of the basin, L_b, is $15h_s$ or $4W_o$ whichever is larger.

FIGURE 7.21
DETAILS OF RIPRAPPED CULVERT ENERGY BASIN
(USDOT, FHWA, HEC-14, 1983)



General Hydraulic Design Procedure:

1. Estimate the flow properties at the brink of the culvert. Establish the brink invert elevation such that $TW/Y_o < 0.75$ for design discharge.

2. For subcritical flow conditions (culvert set on mild or horizontal slope), use Figure 7.22 or Figure 7.23 to obtain Y_o/D, then obtain V_o by dividing Q by the wetted area associated with Y_o. D is the height of a box culvert. If the culvert is on a steep slope, V_o will be the normal velocity obtained by using the Manning equation for appropriate slope, section, and discharge.

3. From site inspection and from field experience in the area, determine whether or not channel protection is required at the culvert outlet.

4. If the channel protection is required, compute the Froude number for brink conditions ($y_e = (A/2)^{1/2}$ for non-rectangular culverts). Select d_{50}/y_e appropriate for locally available riprap (usually the most satisfactory results will be obtained if $0.25 < d_{50}/y_e < 0.45$). Obtain h_s/y_e from Figure 7.24, and check to see that $2 < h_s/d_{50} < 4$. Recycle computations if h_s/d_{50} falls out of this range.

5. Size basin as shown in Figure 7.21.

6. Design procedures where allowable dissipator exit velocity is specified:

   - Determine the average normal flow depth in the natural channel for the design discharge.

   - Extend the length of the energy basin (if necessary) so that the width of the energy basin (at Section A-A, Figure 7.21), times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

7. In the exit region of the basin, the walls and apron of the basin should be warped (or transitioned) so that the cross section of the basin at the exit conforms to the cross section of the natural channel. Abrupt transition of surfaces should be avoided to minimize separation zones and resultant eddies.

8. If high tailwater is a possibility and erosion protection is necessary for the downstream channel, it is recommended to design a conventional basin for low tailwater conditions in accordance with the instructions above. Estimate centerline velocity at a series of downstream cross sections using the information shown in Figure 7.25. Shape the downstream channel and size riprap using guidelines presented in Chapter 5 and the stream velocities obtained above.

FIGURE 7.22
DIMENSIONLESS RATING CURVES FOR THE OUTLETS OF RECTANGULAR CULVERTS ON HORIZONTAL AND MILD SLOPES
(Simons, et al, 1970)

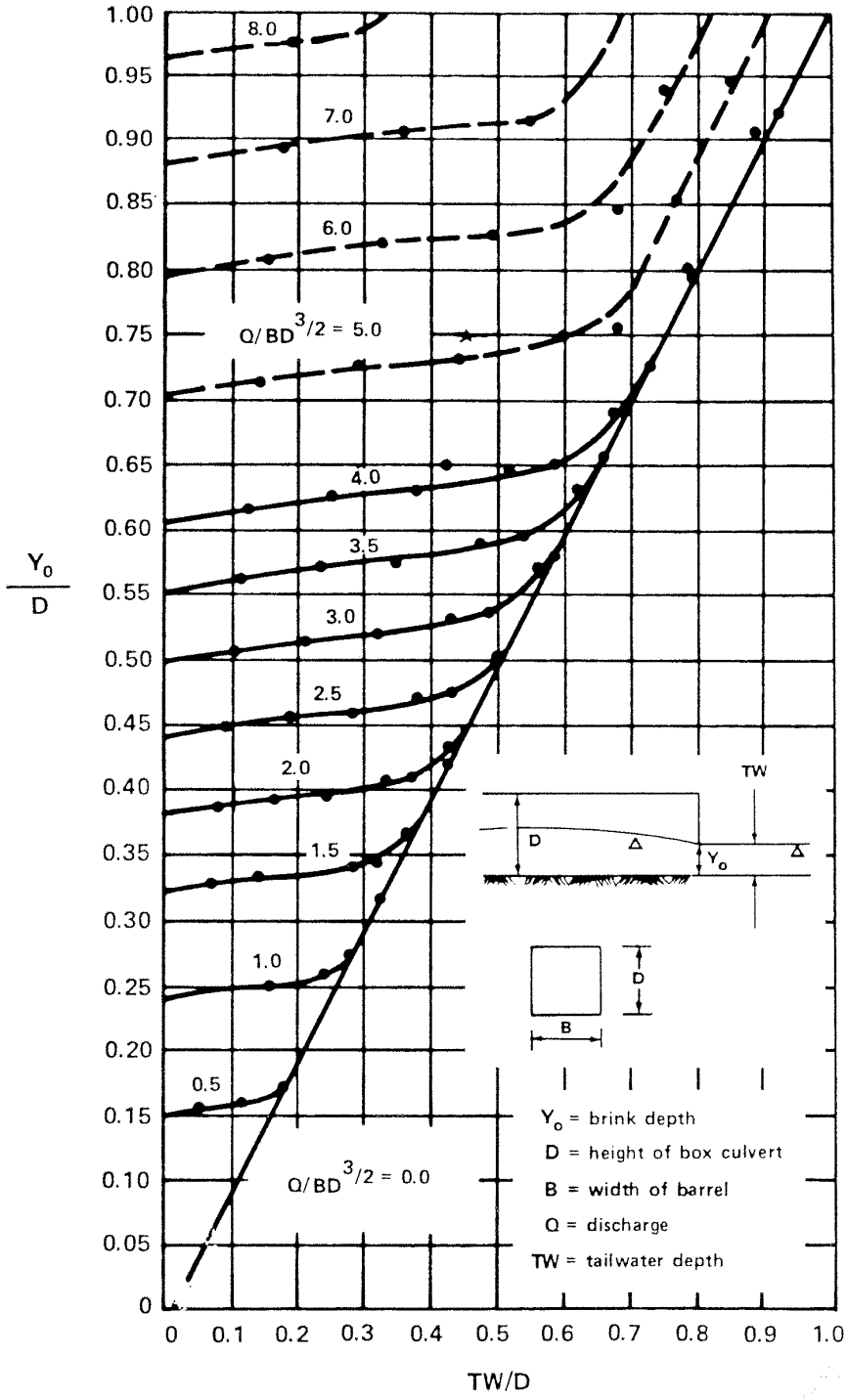


FIGURE 7.23
DIMENSIONLESS RATING CURVES FOR THE OUTLETS OF CIRCULAR CULVERTS ON HORIZONTAL AND MILD SLOPES

(Simons, et al, 1970)

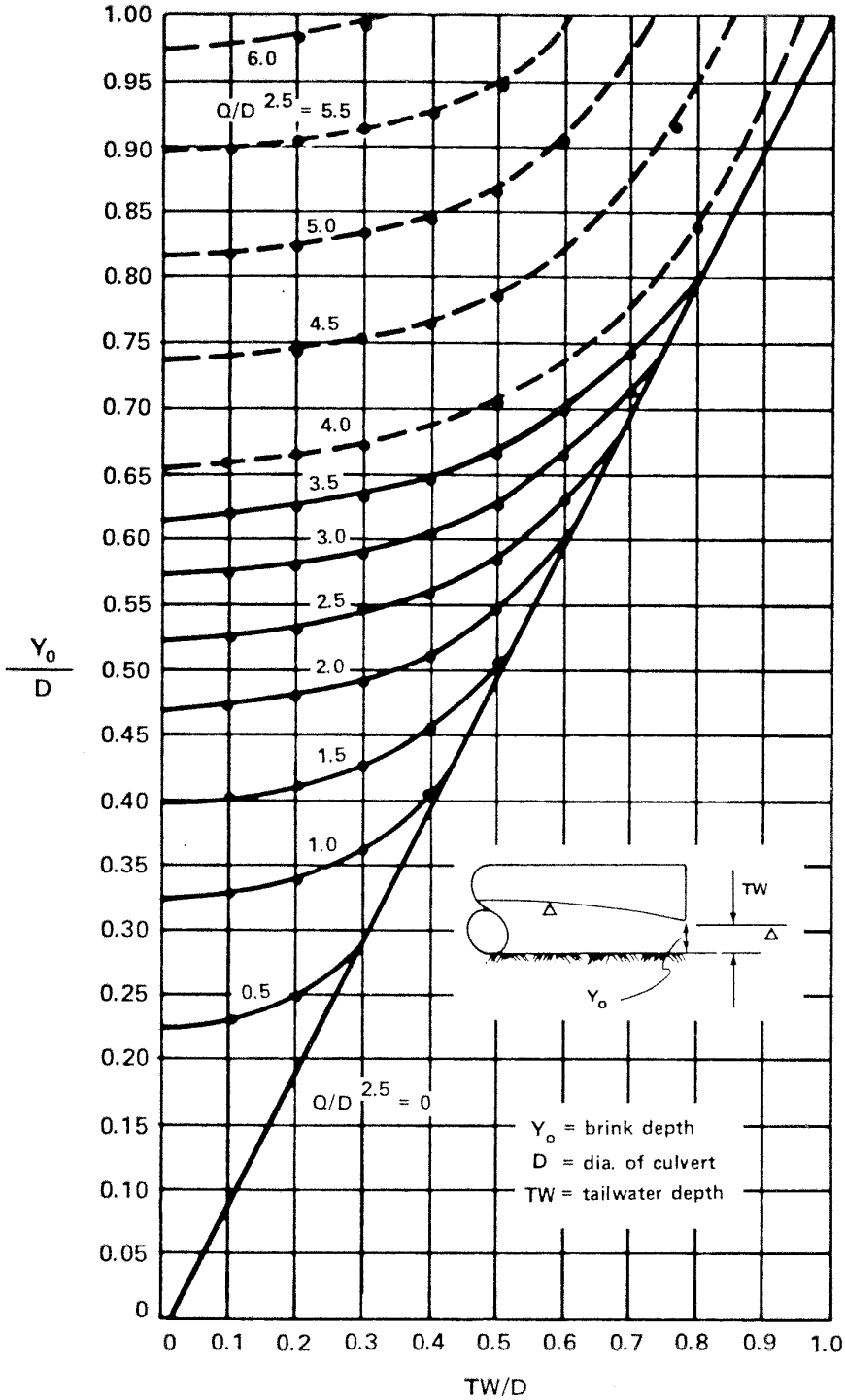


FIGURE 7.24
RELATIVE DEPTH OF SCOUR HOLE VERSUS FROUDE NUMBER
AT BRINK OF CULVERT WITH RELATIVE SIZE OF RIPRAP AS A THIRD VARIABLE

(USDOT, FHWA, HEC-14, 1983)

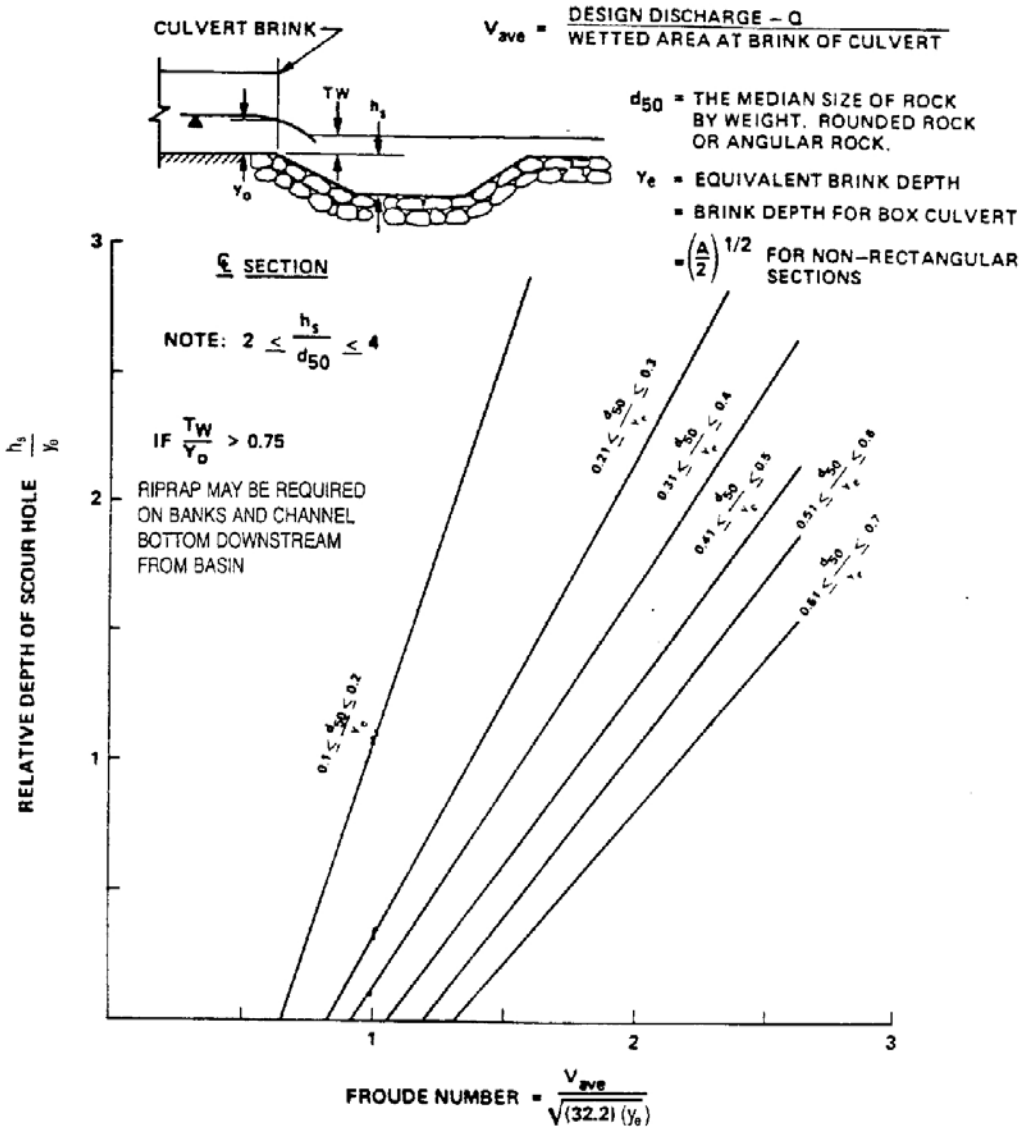
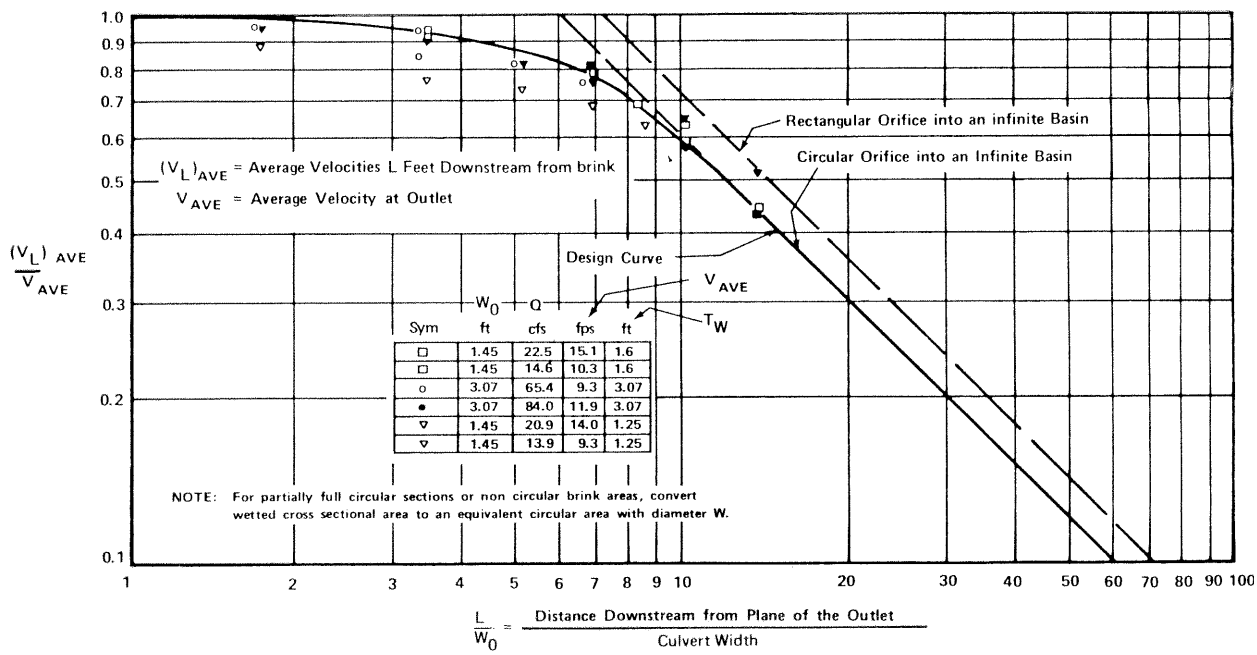


FIGURE 7.25
DISTRIBUTION OF CENTERLINE VELOCITY FOR FLOW FROM SUBMERGED OUTLETS
(TO BE USED FOR PREDICTING CHANNEL VELOCITIES DOWNSTREAM FROM CULVERT OUTLETS WHERE HIGH TAILWATER PREVAILS)
(Simons, et al, 1970; and USDOT, FHWA, HEC-14, 1983)



Sym	W_0 ft	Q cfs	fps	ft
□	1.45	22.5	15.1	1.6
□	1.45	14.6	10.3	1.6
o	3.07	65.4	9.3	3.07
•	3.07	84.0	11.9	3.07
▽	1.45	20.9	14.0	1.25
▽	1.45	13.9	9.3	1.25

Additional information regarding design of riprap basins for conduit outlets may be found in *Hydraulic Design of Energy Dissipators for Culverts and Channels* (USDOT, 1983).

7.4.3 Concrete Outlet Structures

This section provides hydraulic concepts and design criteria for an impact stilling basin and adaptation of a baffled apron to conduit outlets. Initial design selection should include at least the following aspects concerning conduit outlet structures.

1. High energy dissipation efficiency is required - hydraulic conditions exceed the limits for alternate designs (such as riprap outlet protection).

2. Low tailwater control is anticipated. For example, at outfalls to detention/retention facilities that are empty or have low water levels.

3. Use of concrete is more economical due to structure size or local availability of materials.

4. Site conditions direct the use of an outlet structure such as public use areas where plunge pools and standing water are unacceptable or locations with severe space limitations.

Impact Stilling Basin

Design standards are based on the USBR Type VI Basin, commonly referred to as an impact dissipator or conduit outlet stilling basin. The Type VI Basin is a relatively small structure which produces highly efficient energy dissipation characteristics without tailwater control. The original hydraulic design reference is *Hydraulic Design of Stilling Basins for Pipe or Channel Outlets* (Peterka, 1984). Additional structural details are provided in *Design of Small Canal Structures* (USBR, 1974).

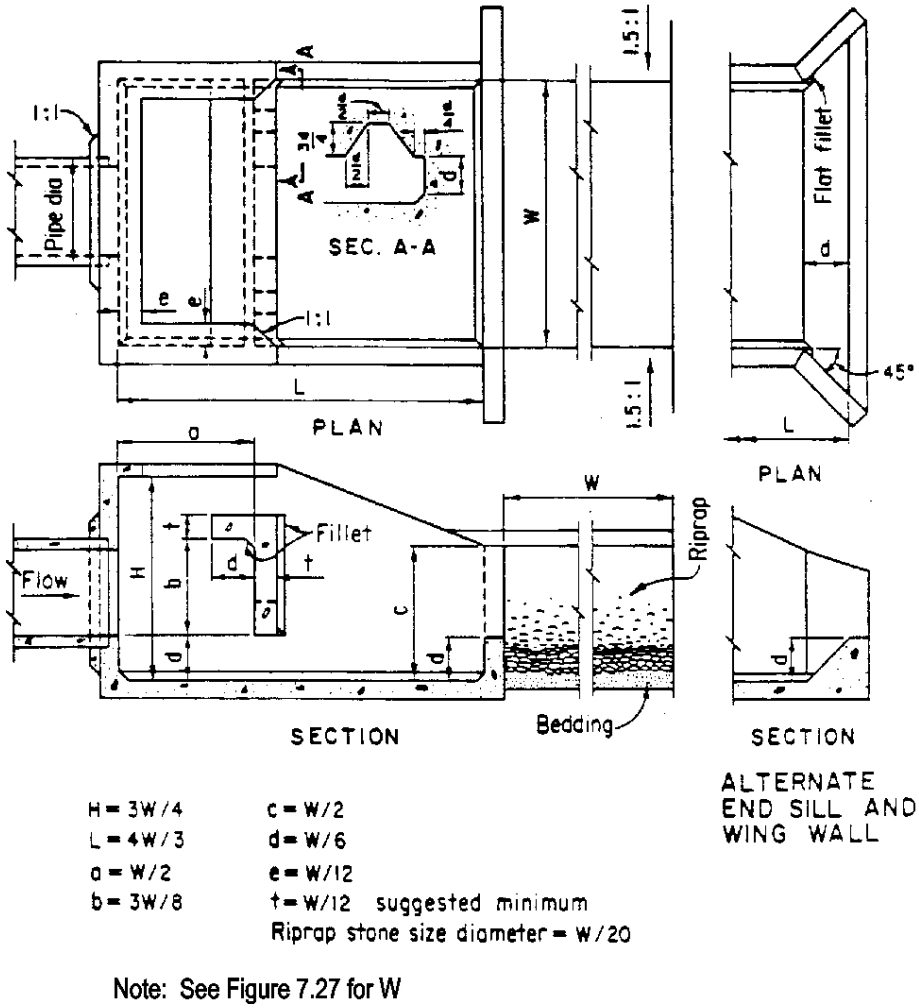
The structure is designed to operate continuously at the design flow rate. Maximum entrance conditions are up to 50 feet per second velocity and Froude number less than 9.0. Conditions exceeding this criteria would be extremely rare in typical urban drainage applications. As a result, the use of this outlet basin is limited only by structural and economic considerations.

Energy dissipation is accomplished through momentum transfer as flow entering the basin impacts a large overhanging baffle. At high flow, further dissipation is produced as water builds up behind the baffle to form a highly turbulent backwater zone. Flow is then redirected under the baffle to the open basin and out to the receiving channel. A sill at the basin end reduces exit velocities by breaking up the flow across the basin floor and improves the stilling action at low to moderate flow rates. A notch is recommended in end sills to provide for low flow drainage.

The generalized design configuration (Figure 7.26) consists of an open concrete box attached directly to the conduit outlet. The side walls are high enough to contain most of the splashing during high flows and slope down to form a transition to the receiving channel. The inlet pipe is vertically aligned with an overhanging L-shaped baffle such that the pipe invert is not lower than the bottom of the baffle. The end sill height is equal to the height under the baffle to produce tailwater in the basin. The alternate and transition (at 45 degrees) is recommended for grass lined channels to reduce the overall scour potential just downstream of the sill.

The standard USBR design has been modified for urban applications to allow drainage of the basin bottom during dry periods. The impact basin can also be adapted to multiple pipe installations. These modifications are discussed following the basic criteria. It should be noted that modifications to the design may affect the hydraulic performance of the structure. Model testing is advised for significant changes to the design.

FIGURE 7.26
GENERAL DESIGN OF THE USBR TYPE VI IMPACT STILLING BASIN
(Adapted from: Peterka 1984)



General Hydraulic Design Procedure for Stilling Basins:

1. Determine the design pipe flow rate Q and the effective flow area A at the outlet. For partial flow conditions, refer to the partial flow diagram in Section 7.3. Using the relationship $Q = AV$, determine the flow velocity V at the pipe outlet. Assume depth $D = A^{0.5}$ and compute the Froude number = $V/(gD)^{0.5}$.

2. The entrance pipe should be turned horizontal at least one pipe diameter equivalent length upstream from the outlet. For pipe slopes greater than 15 degrees, the horizontal length should be a minimum of two pipe diameters.

3. Do not use this type of outlet energy dissipator when exit velocities exceed 50 feet per second or Froude numbers exceed 9.0. These conditions would be extreme and must be

considered as special cases. Performance is achieved with a tailwater depth equal to half full flow level in the pipe outlet.

4. Determine the basin width (W) by entering the appropriate Froude number and effective flow depth on Figure 7.27. The remaining dimensions are proportional to the basin width according to the legend in Figure 7.26. Note that the baffle thickness, t, is a suggested minimum. It is not a hydraulic parameter and is not a substitute for structural analysis.

   The basin width should not be increased since the basin is inherently oversized for less than design flows. Larger basins become less effective as the inflow can pass under the baffle.

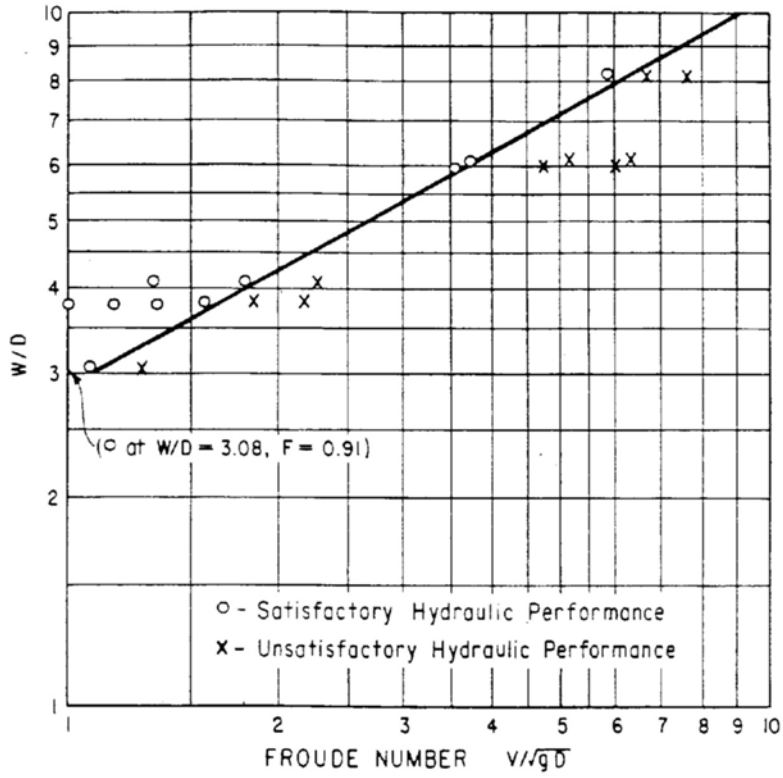
5. Structure wall thickness, steel reinforcement, and anchor walls (underneath the floor) should be designed using accepted structural engineering methods. Hydraulic forces on the overhanging baffle may be approximated by determination of the jet momentum force:

$$F_m = \rho VQ = 1.94\ VQ \tag{7.18}$$

6. Riprap with a minimum d_{50} of 18 inches should be provided in the receiving channel from the end sill to a minimum distance equal to the basin width. The depth of rock should be equal to the sill height or at least 2.5 feet. Rock may be buried below finished grades and the area vegetated as desired to match the site.

7. The alternate end sill and wingwall shown in Figure 7.26 is recommended for all grass lined channels to reduce the scour potential below the sill wall.

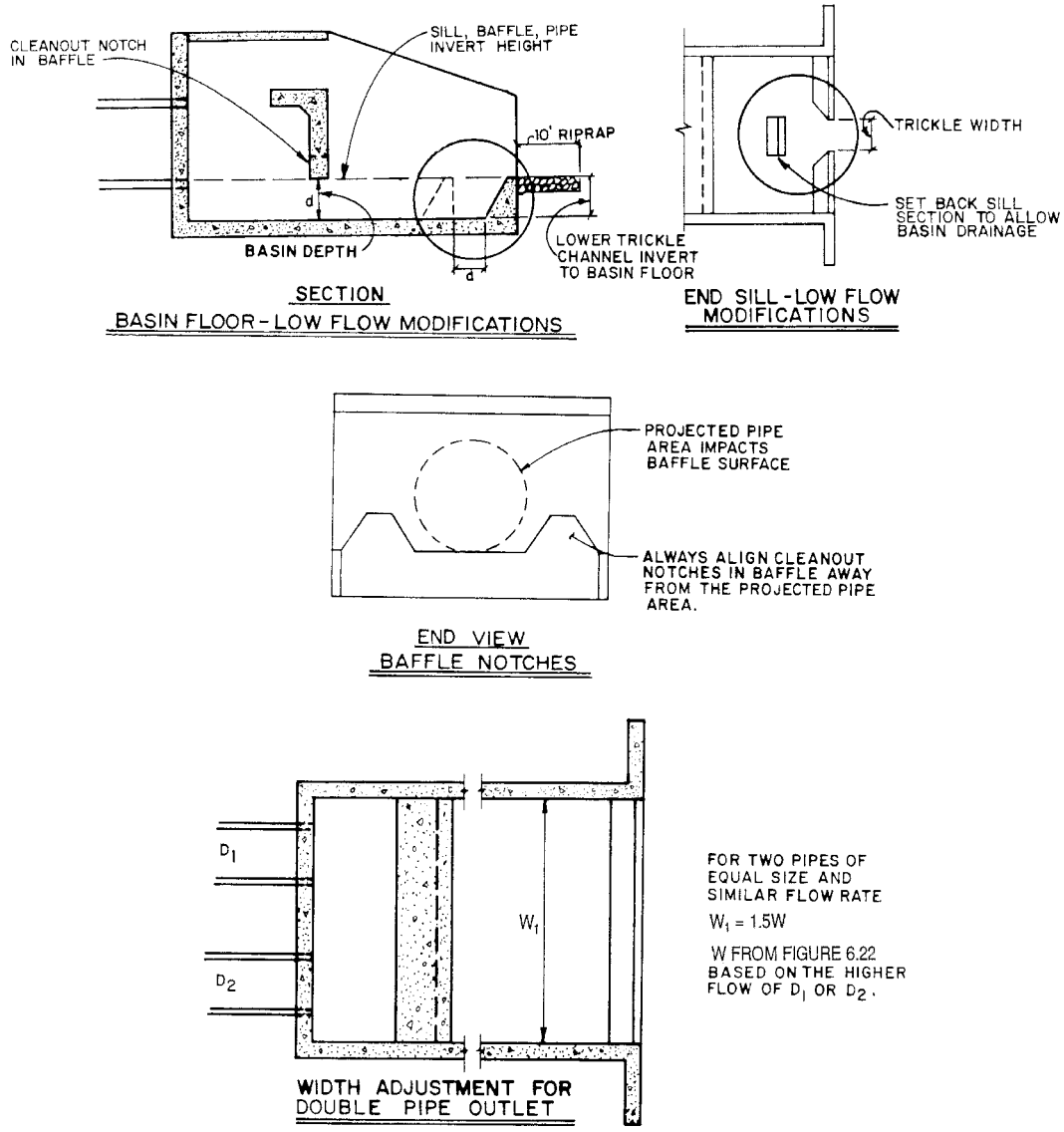
FIGURE 7.27
DESIGN WIDTH OF THE USBR TYPE VI BASIN
(Adapted from: Peterka, 1984)



"w" Is the inside width of the basin
"D" Represents the depth of flow entering the basin and is the square root of the area at the conduit outlet
"v" Is the velocity of the incoming flow
The tailwater depth is uncontrolled

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 7.28.

FIGURE 7.28
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 7.26) 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 7.28.

Multiple Conduit Installations

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 7.28.

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.

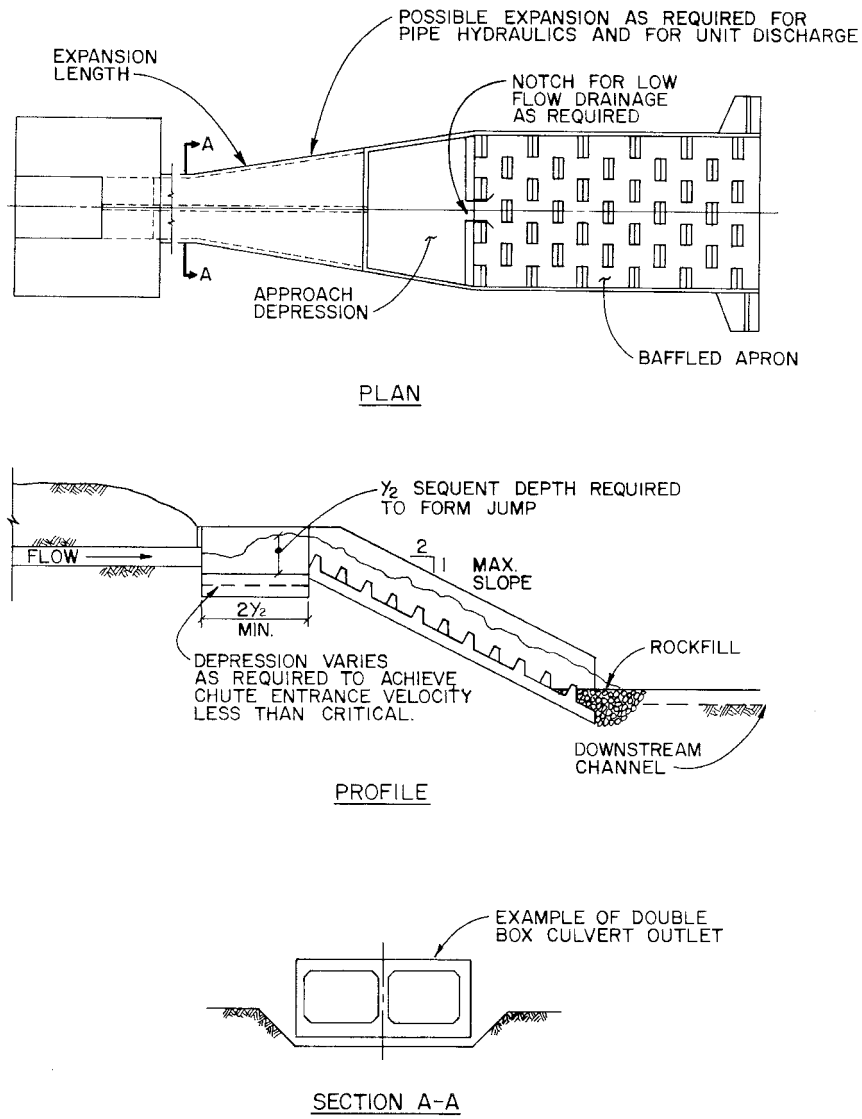
Remaining structure dimensions are based on the design width of a separate basin W. If the two pipes have different flows, the combined structure should be based on the higher Froude number flows.

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 sub-

sidence. 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 7.29 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 7.3.4. 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 7.29
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, 1983).

7.5 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 (7.19)) and the orifice equation (Equation (7.20)) are the commonly used analytic methods.

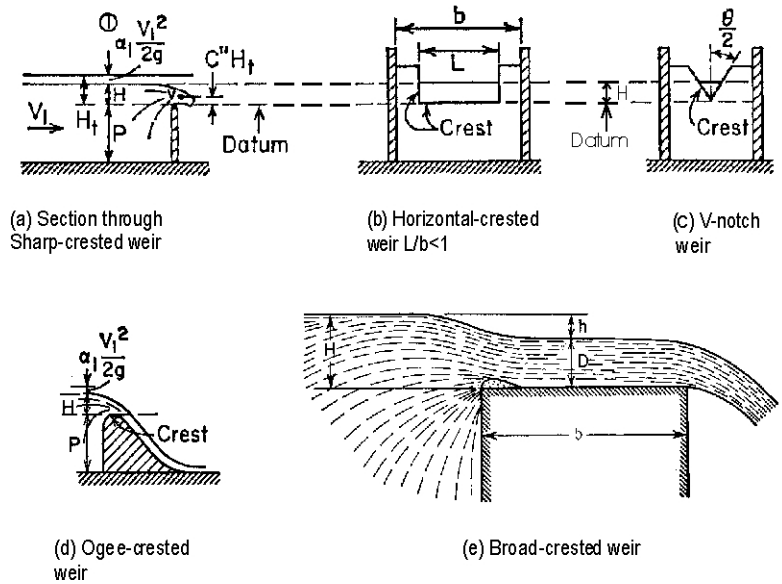
$$Q = CLH^{3/2} \tag{7.19}$$

$$Q = C_o A(2gH)^{1/2} \tag{7.20}$$

Weir-Type Spillways

Weir-type spillways can be generally classified as sharp crested, broad crested or compound curve (ogee) shaped (Figure 7.30). 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 7.30
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 (7.21)) (Chow, 1959, pg. 362) is often used to estimate C;

$$C = 3.27 + 0.40\ H/h_w \tag{7.21}$$

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 Chapter 5, 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).

It is important to note that downstream water surface elevation (tailwater) must be analyzed by appropriate methods (see the Open Channel Chapter) 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) 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.

Compound Rating Curves

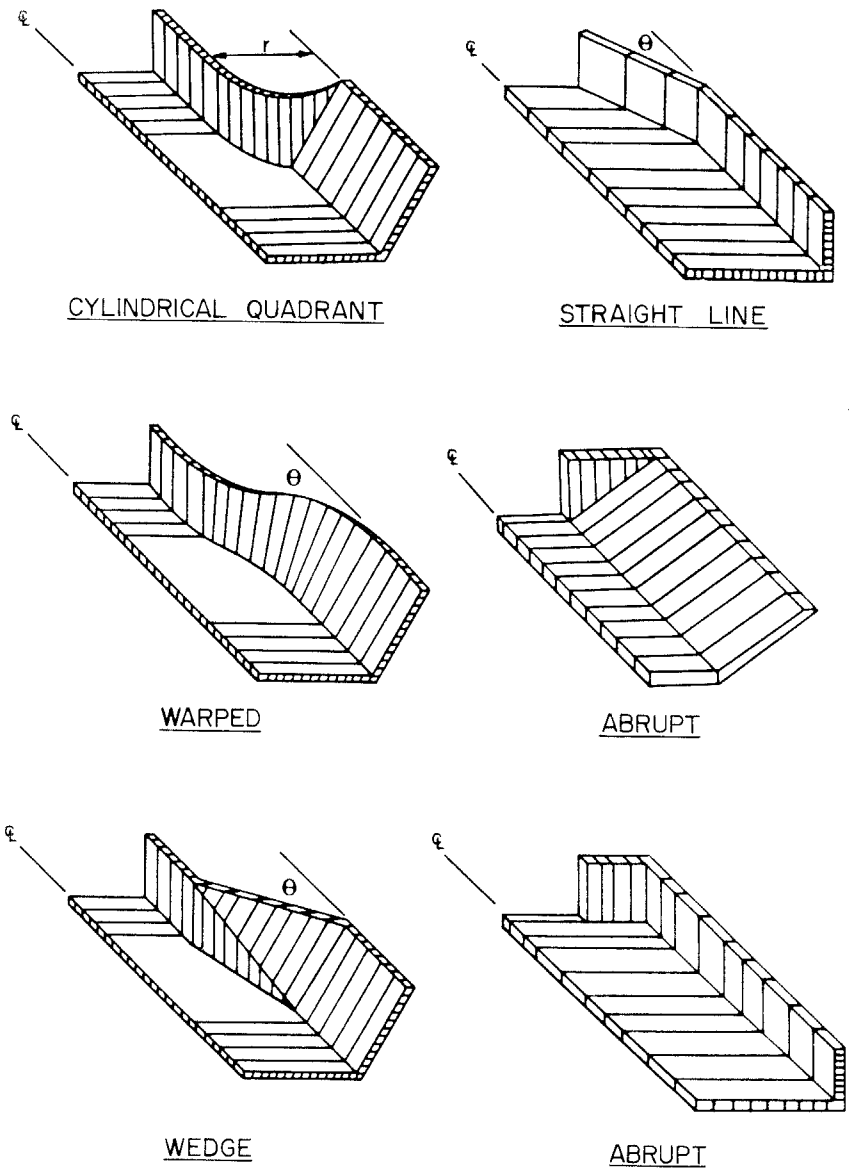
When an impoundment incorporates more than one spillway, a compound rating curve is developed for use in storage routing. Coupled with stage-storage data, an inflow hydrograph can be routed through a basin, thereby estimating ponded water surface elevation and outflow discharge (See Chapter 8). The principal 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.

7.6 SPECIAL CHANNEL STRUCTURES

7.6.1 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 7.31. Of these, the abrupt (headwall) and the straight line (wingwall) are the most common.

FIGURE 7.31
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, R/W, 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 Spillways

Side channel spillways offer a unique design consideration since the channel energy grade varies parallel to the spillway. Thus, weir equations are not always applicable. The FCDMC reviews side channel spillways on a case by case basis. The hydraulic analysis of side channel spillways should be pursued with consultation with FCDMC staff.

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, 1991) 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.

7.6.2 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* (USACOE, 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.

$$F_b = M\Delta V \tag{7.22}$$

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.

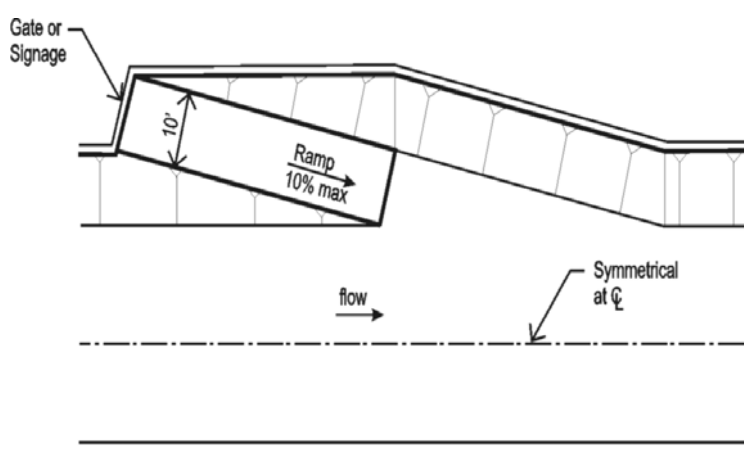
$$\Delta P = \frac{P}{2}(\Delta V^2) \tag{7.23}$$

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.

7.6.3 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 7.32 illustrates a typical ramp design and a typical flared sideslope design.

FIGURE 7.32
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.

7.6.4 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 (7.24) should be multiplied by the appropriate value from Table 7.6, when the approach channel and trashrack are at an angle to each other. Equation (7.24) 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.

$$H_g = 1.5 \frac{[V_g^2 - V_a^2]}{2g} \tag{7.24}$$

TABLE 7.6
LOSS FACTORS FOR APPROACH ANGLE SKEWED TO TRASHRACK

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 (7.25):

$$H_g = K_t \frac{V^2 n}{2g} \tag{7.25}$$

where: K_t is given by Equation (7.26):

$$K_t = 1.45 - 0.45 \frac{a_n}{a_g} - \left(\frac{a_n}{a_g}\right)^2 \tag{7.26}$$

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.

7.6.5 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 10 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).

7.7 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).

7.8 OPERATION, MAINTENANCE & AESTHETIC CONSIDERATIONS

7.8.1 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 pre-

sented 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.

7.8.2 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.

7.9 DESIGN EXAMPLES FOR RIPRAP BASINS

7.9.1 Design Example Number 1

GIVEN:

8 ft by 6 ft box culvert

Q = 800 cfs

supercritical flow in culvert

normal flow depth = brink depth

Y_o = 4 ft

Tailwater depth (TW) = 2.8 ft

FIND:

Riprap basin dimensions for these conditions.

SOLUTION:

1. $Y_o = y_e$ for rectangular section, y_e = 4 ft

2. $V_o = Q/A$ = 800/(4)(8) = 25 ft/sec

3. $F_r = V_o/ [(32.2)(y_e)]^{0.5}$ = 25/[(32.2)(4)]$^{0.5}$ = 2.20

4. TW/y_e = 2.8/4.0 = 0.7; TW/y_e < 0.75

5. Try d_{50}/y_e = 0.45, d_{50} = (0.45)(4) = 1.80 ft

From Figure 7.24, h_s/y_e = 1.6

h_s = (4)(1.6) = 6.4 ft

h_s/d_{50} = 6.4/1.8 = 3.6 ft, $2 < h_s/d_{50} < 4$

6. From Figure 7.21:

L_s = (10)(6.4) = 64 ft

L_s min. = (3)(W_o) = (3)(8) = 24 ft, use L_s = 64 ft

L_B = (15)(6.4) = 96 ft

L_B min. = (4)(W_o) = (4)(8) = 32 ft, use L_B = 96 ft

Other basin dimensions designed in accordance with details are shown in Figure 7.21.

Use of a filter fabric or engineered blanket is recommended.

7.9.2 Design Example Number 2

GIVEN:

8 ft by 6 ft box culvert

Q = 800 cfs

supercritical flow in culvert

normal flow depth = brink depth

Y_o = 4 ft

Tailwater depth (TW) = 4.2 ft

Downstream channel can tolerate 7 ft/sec for design discharge

FIND:

Riprap basin dimensions for these conditions.

SOLUTION:

Note: High tailwater depth, TW/Y_o = 1.05 > 0.75.

1. Design riprap basin using steps 1 through 6 of Design Example 1.

d_{50} = 1.8 ft; h_s = 6.4 ft; L_s = 64 ft; L_B = 96 ft

2. Design riprap for downstream channel. Use Figure 7.25 to estimate the average velocity along the channel. Compute the equivalent circular diameter, D_e, for the brink area, A, from:

$A = \pi(D_e^2 / 4) = Y_o W_o$

$\pi\ (D_e^2 / 4) = 4(8) = 32$ sq ft

$D_e = [4(4)(8/\pi)]^{0.5} = 6.4$ ft

V_o = 25 ft/sec (Design Example Number 1)

L/D_e	L (compute)	V_L/V_o (Figure 7.25)	V_L
10	64	0.59	14.7
15	96	0.36	9.0
20	128	0.30	7.5
21	135	0.28	7.0

The channel should be lined with the same size rock used for the basin. Protection must extend at least 135 feet downstream from the culvert brink.

7.9.3 Design Example Number 3

GIVEN:

6 feet diameter cmp

Q = 135 cfs

S_o = 0.004 ft/ft

Manning's n = 0.024

Normal depth in pipe for Q = 135 cfs is 4.5 feet

Normal velocity is 5.9 ft/sec

Flow is subcritical

Tailwater depth (TW) is 2.0 ft

FIND:

Riprap basin dimensions for these conditions.

SOLUTION:

1. Determine Y_o and V_o:

$Q/D^{2.5} = 135/(6)^{2.5} = 1.53$

TW/D = 2.0/6 = 0.33

From Figure 7.23, Y_o/D = 0.45

Y_o = (0.45)(6) = 2.7 ft

TW/Y_o = 2.0/2.70 = 0.74, TW/Y_o < 0.75 OK

Brink Area (A) for Y_o/D = 0.45 is:

A = (0.343)(36) = 12.3 sq ft [0.343 is from Table 7.2]

$V_o = Q/A$ = 135/12.3 = 11.0 ft/sec

2. $y_e = (A/2)^{0.5} = (12.3/2)^{0.5} = 2.48$ ft

3. $F_r = V_o/[(32.2)(y_e)]^{0.5} = 11/[(32.2)(2.48)]^{0.5} = 1.23$

4. Try $d_{50}/y_e = 0.25$, $d_{50} = (0.25)(2.48) = 0.62$ ft

From Figure 7.23, $h_s/y_e = 0.75$, $h_s = (0.75)(2.48) = 1.86$ ft

check: $h_s/d_{50} = 1.86/0.62 = 3$, $2 < h_s/d_{50} < 4$OK

5. $L_s = (10)(h_s) = (10)(1.86) = 18.6$

or

$L_s(3)(W_o) = (3)(6) = 18$ ft, Use $L_s = 18.6$ ft

$L_B = (15)(h_s) = (15)(1.86) = 27.9$ ft

or

$L_B = (4)(W_o) = (4)(6) = 24$ ft, Use $L_B = 27.9$ ft

$d_{50} = 0.62$ ft, Use $d_{50} = 8$ inches

Other basin dimensions designed in accordance with details are shown on Figure 7.21.

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8 STORMWATER STORAGE

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8.1 SYMBOLS

The following symbols will be used in equations throughout Chapter 8.

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

8.2 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.

8.2.1 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.

8.2.2 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.

8.3 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.

8.3.1 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

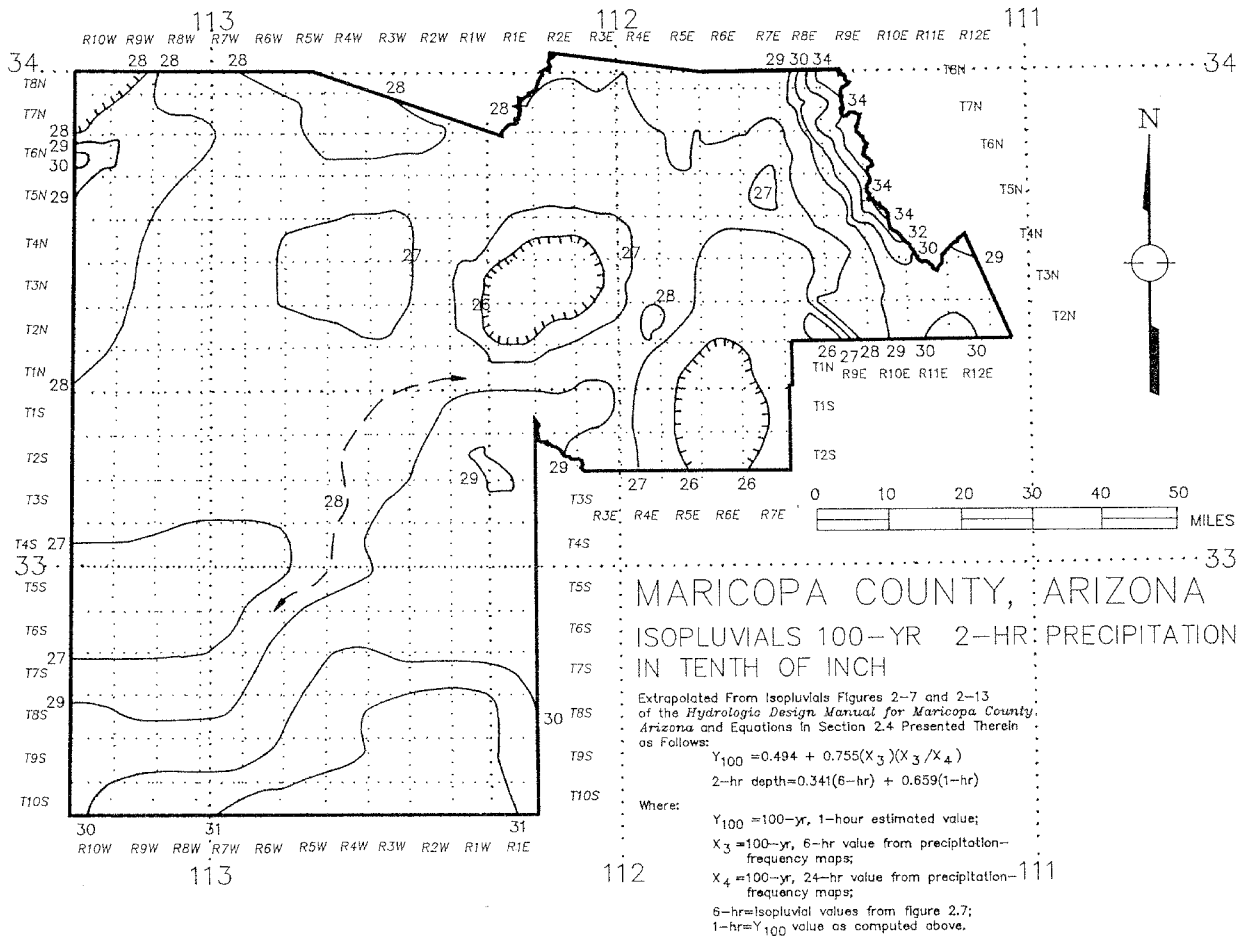
Procedures and criteria for development of inflow hydrographs for stormwater storage facilities are described in the *Drainage Design Manual for Maricopa County, Hydrology*.

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:

$$V = C\left(\frac{P}{12}\right) A \tag{8.1}$$

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 8.1. The amount of rainfall for other frequencies and durations can be determined by using Section 2.2 of the Hydrology Manual.

FIGURE 8.1
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 8.2
FLOOD ROUTING (INFLOW AND OUTFLOW HYDROGRAPH)

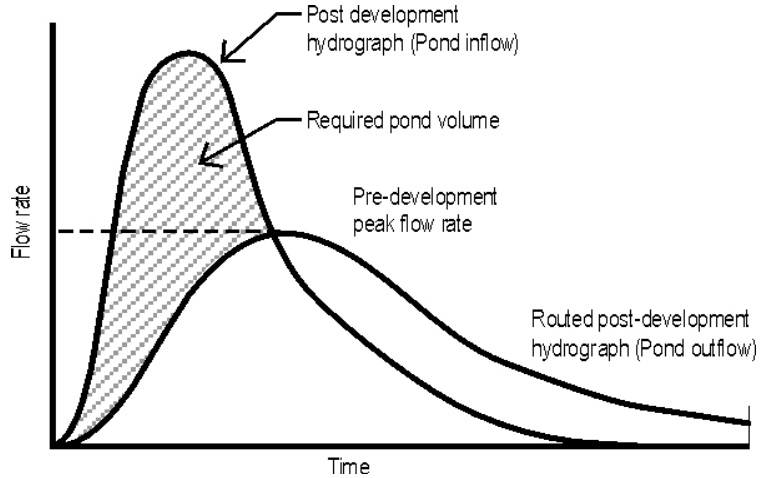


Figure 8.2 illustrates the general relationship among the three variables that must be considered for flood routing through a structure.

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 10 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 8.4, Section 8.5, and Section 8.6 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 8.5.1 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:

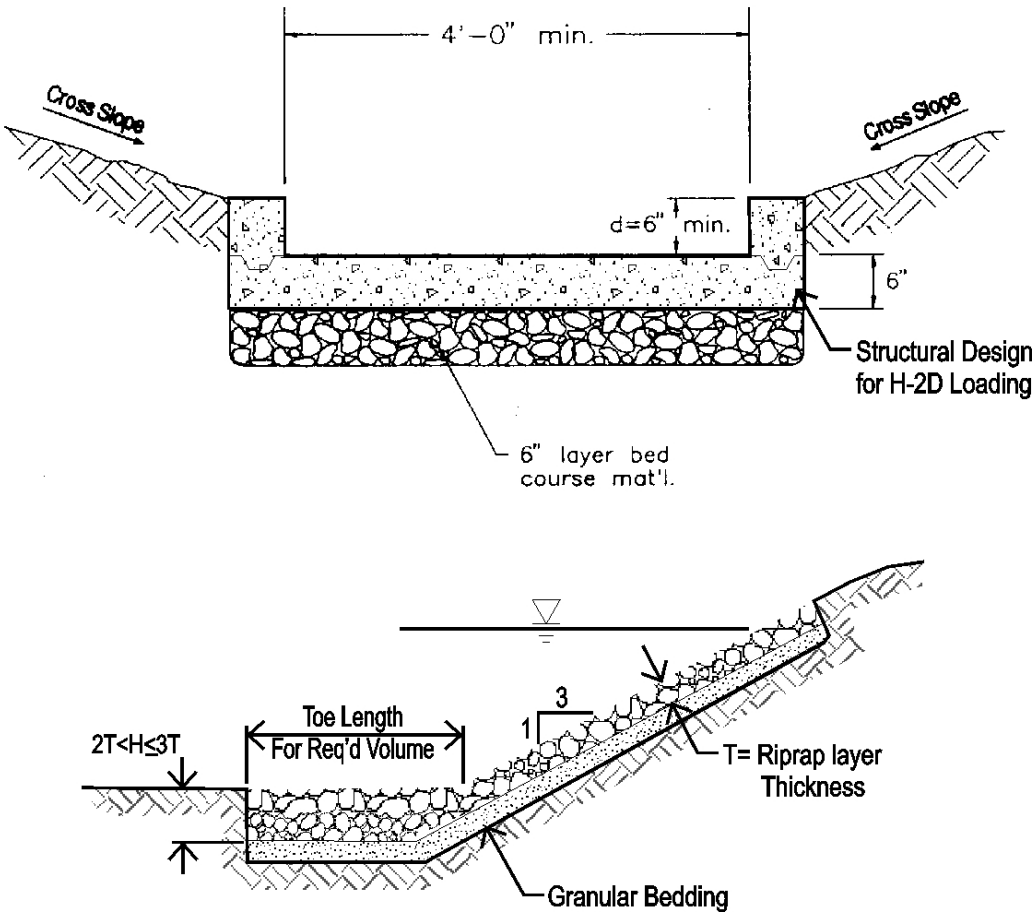
- Loose rock riprap with a specific, engineered gradation
- Loose or grouted boulders (minimum dimension 18 inches and larger)
- River stone
- Gabions
- Soil cement and concrete

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 8.3. 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 8.3
RECTANGULAR CONCRETE CHANNEL SECTION
(Adapted from: WRC Engineering, Inc. 1985)



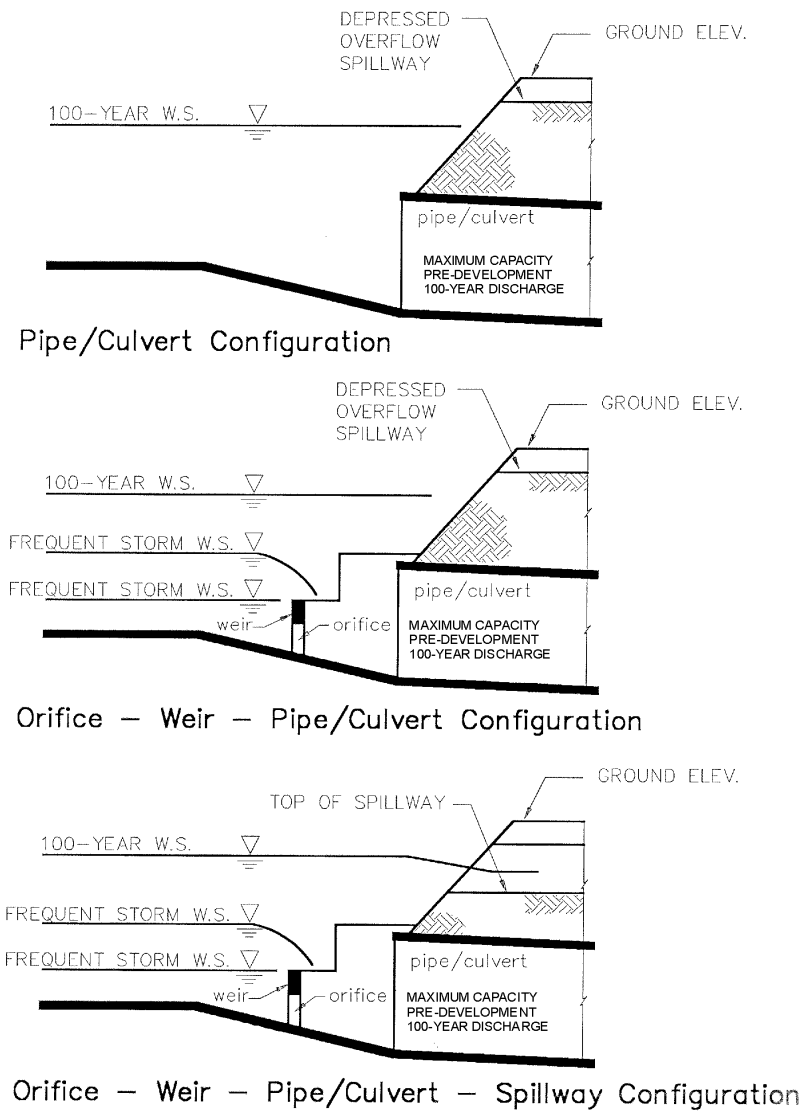
Stormwater Storage Facility Inlet and Outlet Structures

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 8.4 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 8.4
EXAMPLES OF PRIMARY OUTLET STRUCTURES
(Pima County Department of Transportation and Flood Control District, 1986)

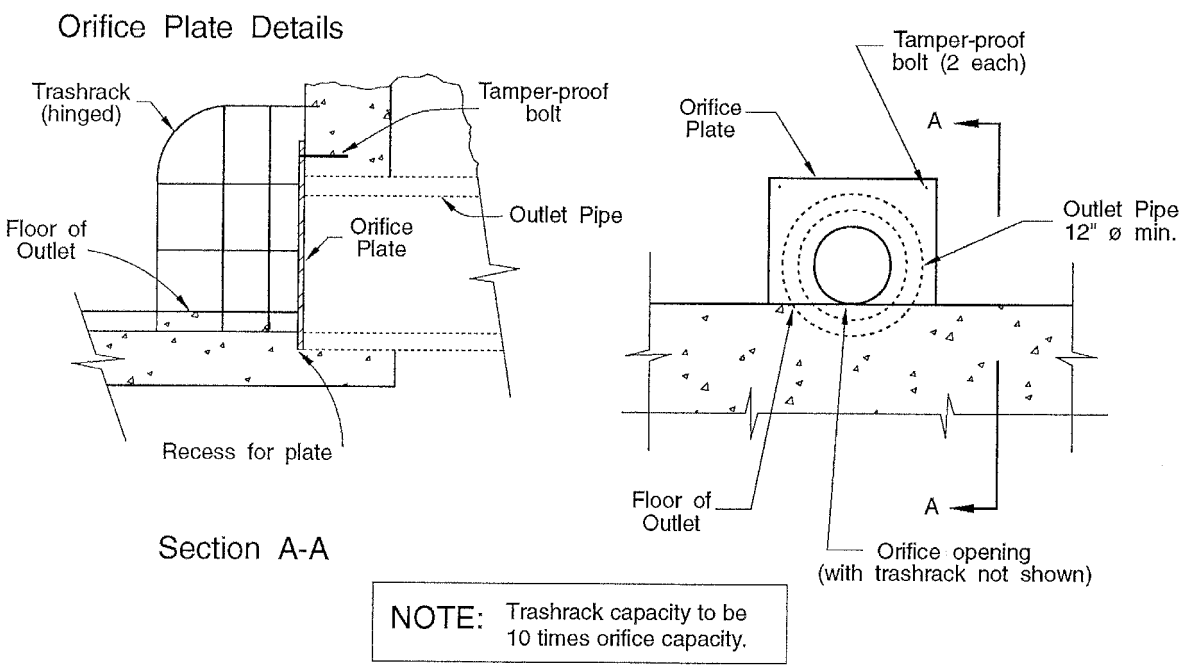


If the flow capacity of an outlet pipe must be further reduced, an orifice plate may be attached, as shown on Figure 8.5(a). The orifice plate must be constructed of heavy, galvanized steel and attached by tamper-proof bolts. Other outlet configurations may be allowed provided they meet

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 8.5
ORIFICE PLATE DETAIL
(Adapted from WRC Engineering, 1987)



Trashracks - Trashracks shall be provided to inlets of pipe and orifice outlet structures. See Chapter 7 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 8.3.3.

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 8.7 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 8.6 shows a typical drywell installation, while Figure 8.7 shows examples of surface treatments.

All drywells must be registered with the Arizona Department of Environmental Quality (ADEQ).

FIGURE 8.6
TYPICAL DRYWELL INSTALLATION
(McGuckin Drilling Inc., 1987)

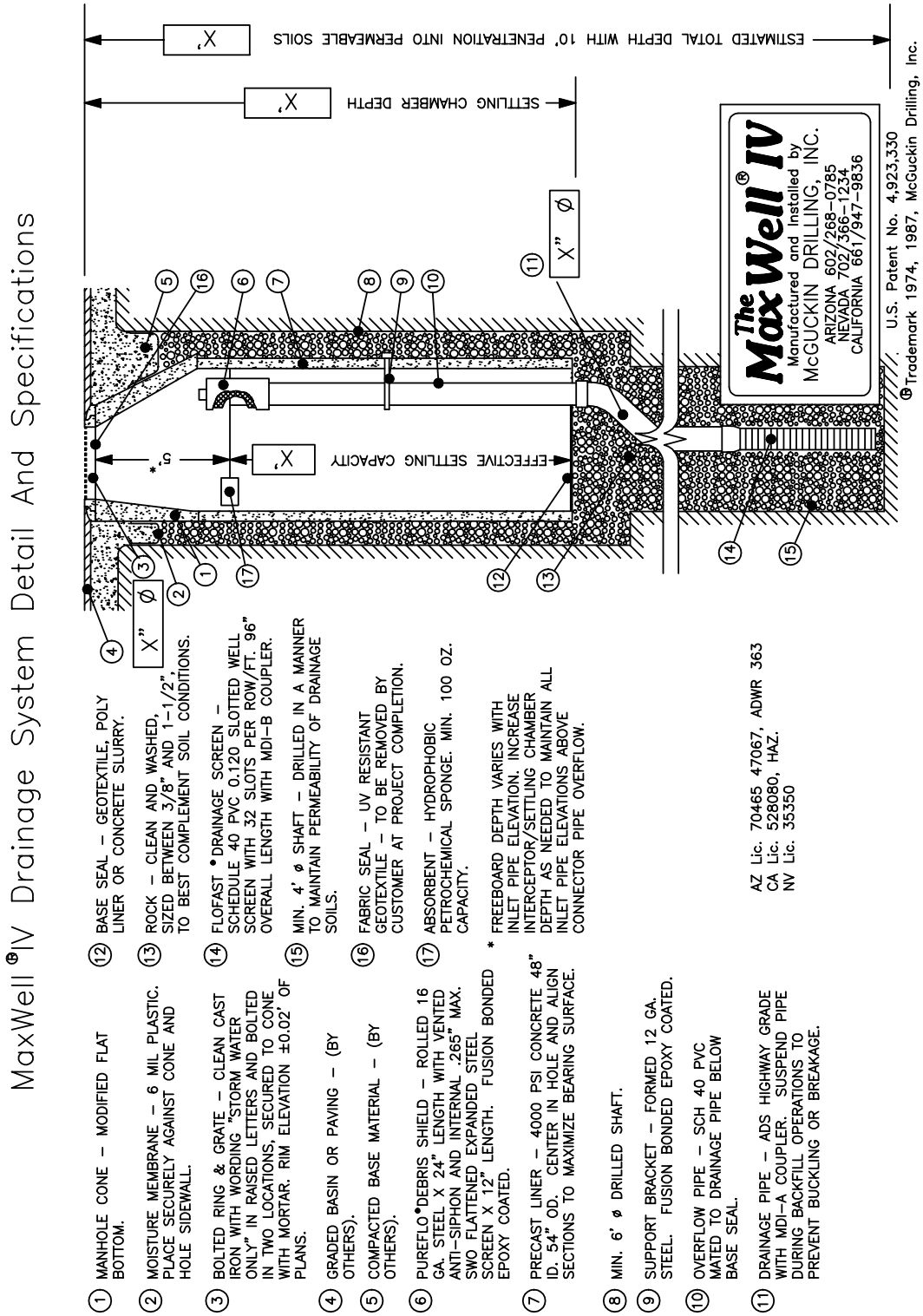
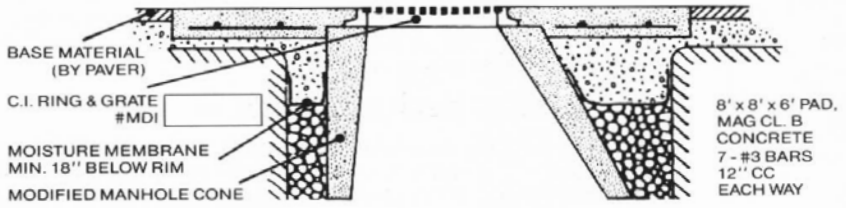


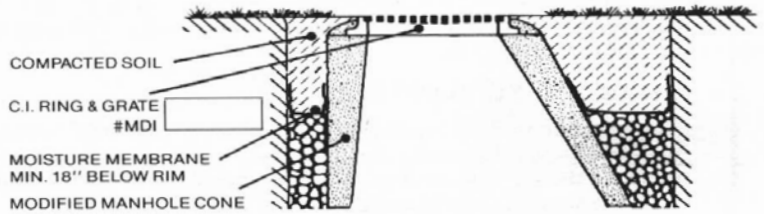
FIGURE 8.7
TYPICAL DRYWELL SURFACE TREATMENTS
(McGuckin Drilling Inc., 1987)

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.

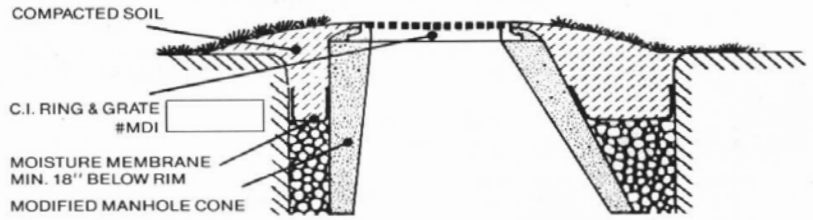
A. Adds a concrete pad for heavy traffic areas.



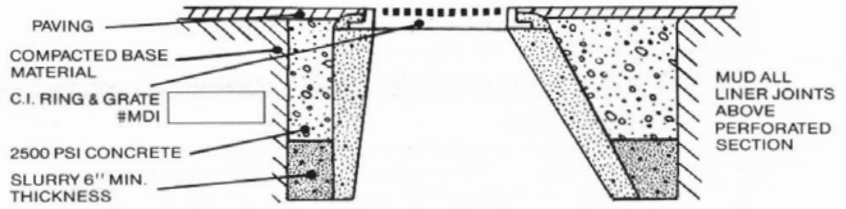
B. For landscaped retention ponds and planters. No paving or pad. In areas where silt might flow to drywell, use C.



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 of the applicable jurisdictional agency.

- The feasibility of subsurface disposal of stormwater at a site by an engineered basin must be documented by field investigations and testing in accordance with ASTM Standard D-3385, Double-Ring Infiltrometer. The field investigation shall include borings at least 10-feet deep to assure that the soils underlying the basin will not impede flow, thereby resulting in water mounding. One test boring shall be provided for each 2500 square feet of basin bottom. The test results shall be de-rated by a factor of 10 to allow for future working conditions in the completed retention facility. This compensation is necessary because actual functional rates will diminish over time due to basin soil compaction, progressive silt loading, and actual stormwater constituents that clog soil pores. A report shall be prepared by a geotechnical or civil engineer, licensed to practice in Arizona, that documents the field investigations, testing, adjusted design rates and the final proposed design. Shallow-pit percolation tests for obtaining permeability rates are not recommended. Refer to the jurisdiction's Policy and Standards Manual for specific requirements in this regard. If shallow-pit percolation tests are used in-lieu of the double-ring infiltrometer test, a de-rating factor of 20 shall be applied.

- The accepted design disposal rate for a drywell is not to exceed 0.1 cfs per well unless a greater rate can be supported by a constant-head percolation test on a completed well at the site. Should this test reflect a higher value, the results shall be de-rated based on the in-situ soil conditions. A de-rating factor of 2 shall be applied for coarse-grained soils (cobbles, gravels and sands). A de-rating factor of 3 shall be applied for fine grained soils (silts and loams). A de-rating factor of 5 shall be applied for clay soils. The maximum allowable rate shall not exceed 0.5 cfs per drywell in any case for design purposes. In accordance with ADEQ requirements, the installation of any subsurface drainage structure must be located 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 minmum 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 8.4).

- 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 8.7.4).

- 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 8.3.3).

- 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.

8.3.2 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 bot-

toms 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 8.3.3).

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 jurisdic-

tion. 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 8.4). Positive drainage to the outlet structures and trash/debris control must be provided so that the facility drains completely and efficiently.

8.3.3 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.

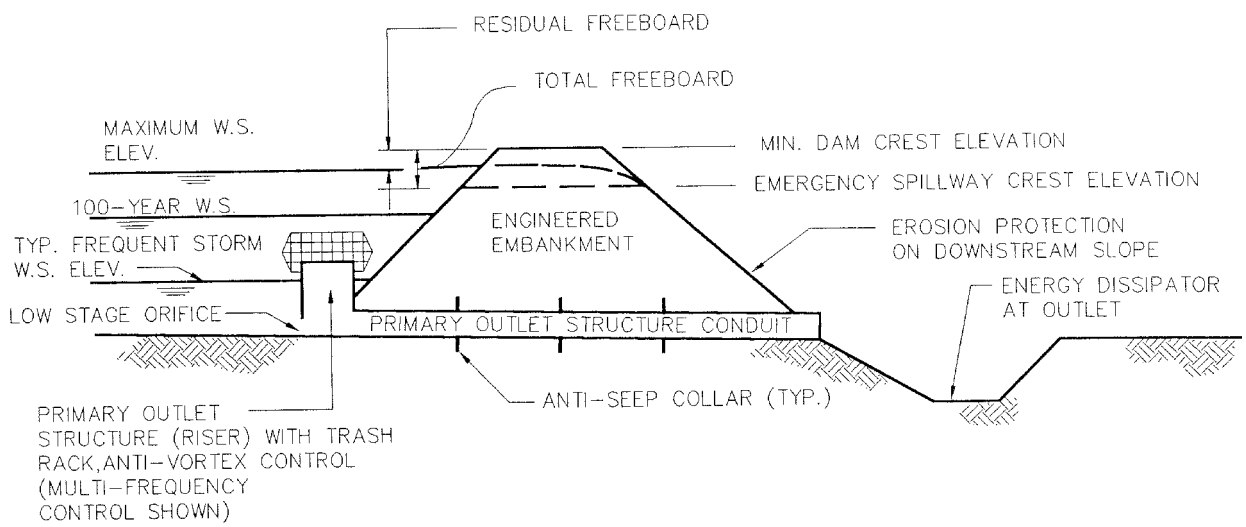
Design of Embankments Not Regulated by ADWR

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 8.8 shows a typical section of an embankment dam with common components applicable to a typical detention or retention facility.

FIGURE 8.8
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

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.

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 spillway 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 8.1. Spillway design standards for jurisdictional dams and total freeboard and residual freeboard dimensions shall conform to the applicable ADWR design requirements.

TABLE 8.1
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.

1. Generally, these are designed as a broad crested weir (see Chapter 7, Hydraulic Structures).

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, 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.

8.4 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.

8.5 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.

8.5.1 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.

8.5.2 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 8.7.4).
- Provide controlled vehicular access into the facility for trucks and front-end loaders.

8.5.3 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.

8.5.4 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.

8.5.5 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.

8.5.6 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.

8.5.7 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.

8.5.8 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.

8.6 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.

8.6.1 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 8.9 provides a generic site plan for recreation elements that might be incorporated into a stormwater storage facility. Figure 8.10 depicts a typical cross-section of the site plan.

FIGURE 8.9
STORMWATER STORAGE FACILITY RECREATION ELEMENTS

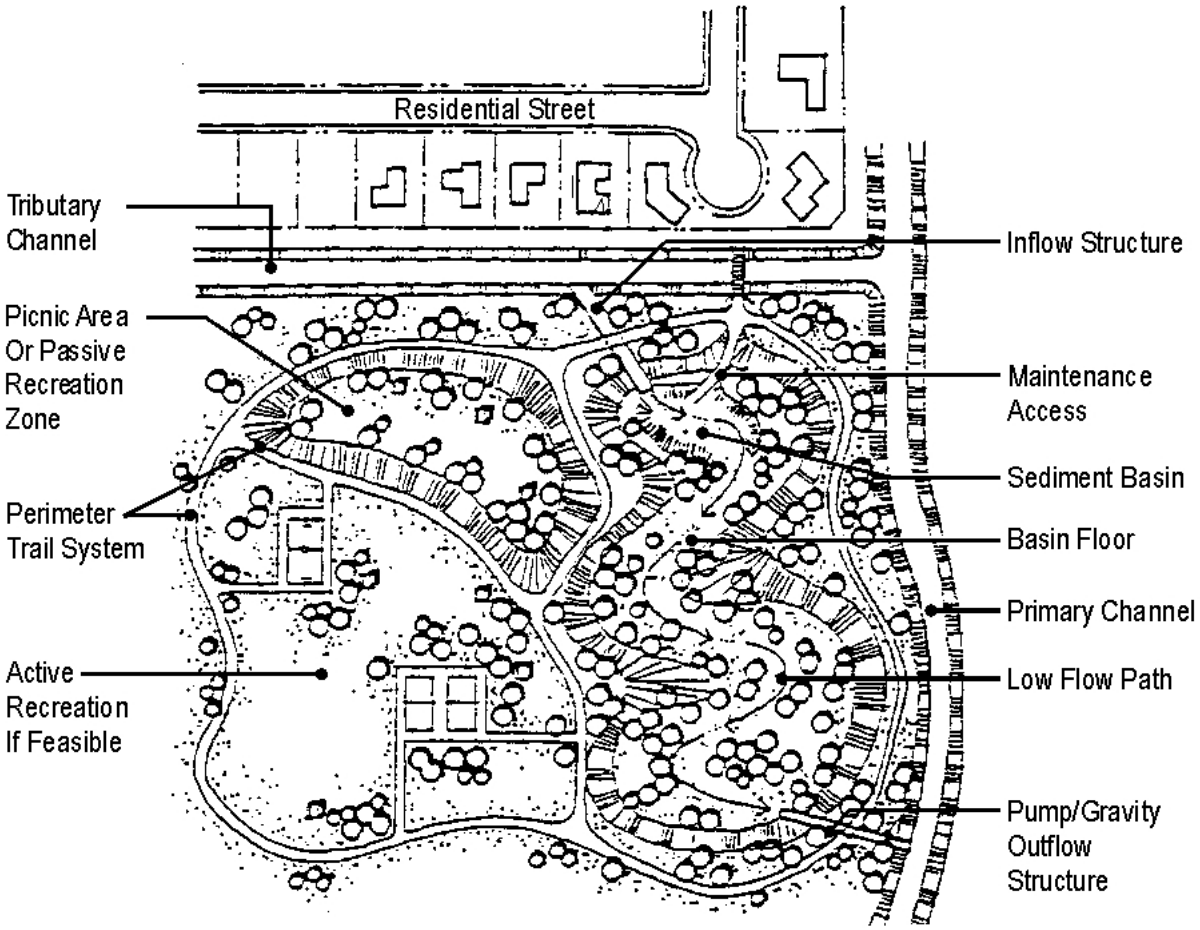
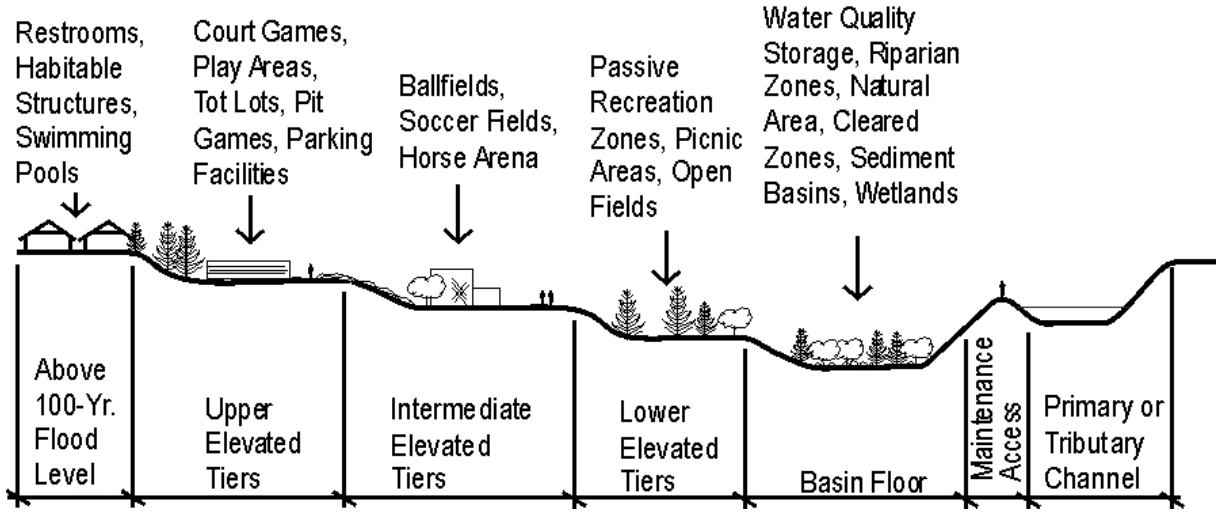


FIGURE 8.10
TYPICAL CROSS-SECTION OF STORMWATER STORAGE FACILITY W/RECREATION ELEMENTS



8.6.2 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.

8.6.3 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.

8.6.4 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.

8.7 WATER QUALITY

8.7.1 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.

8.7.2 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.

8.7.3 Role of Stormwater Storage Facilities in Water Quality Control

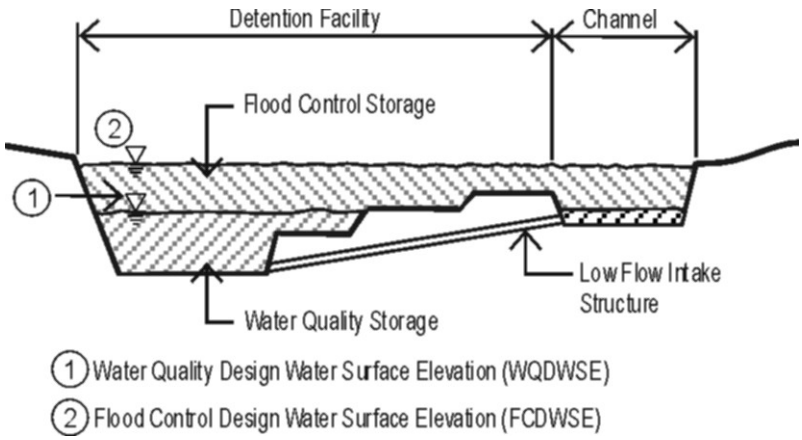
Most pollutants of concern have a high affinity for suspended solids in runoff and for soil particles. Thus, the most logical way to achieve pollutant removal is through sedimentation and infiltration. Consequently, stormwater storage facility design for water quality control should maximize settling 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 8.11.
- 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 8.11
STORAGE COMPONENTS OF A DUAL PURPOSE STORMWATER STORAGE FACILITY

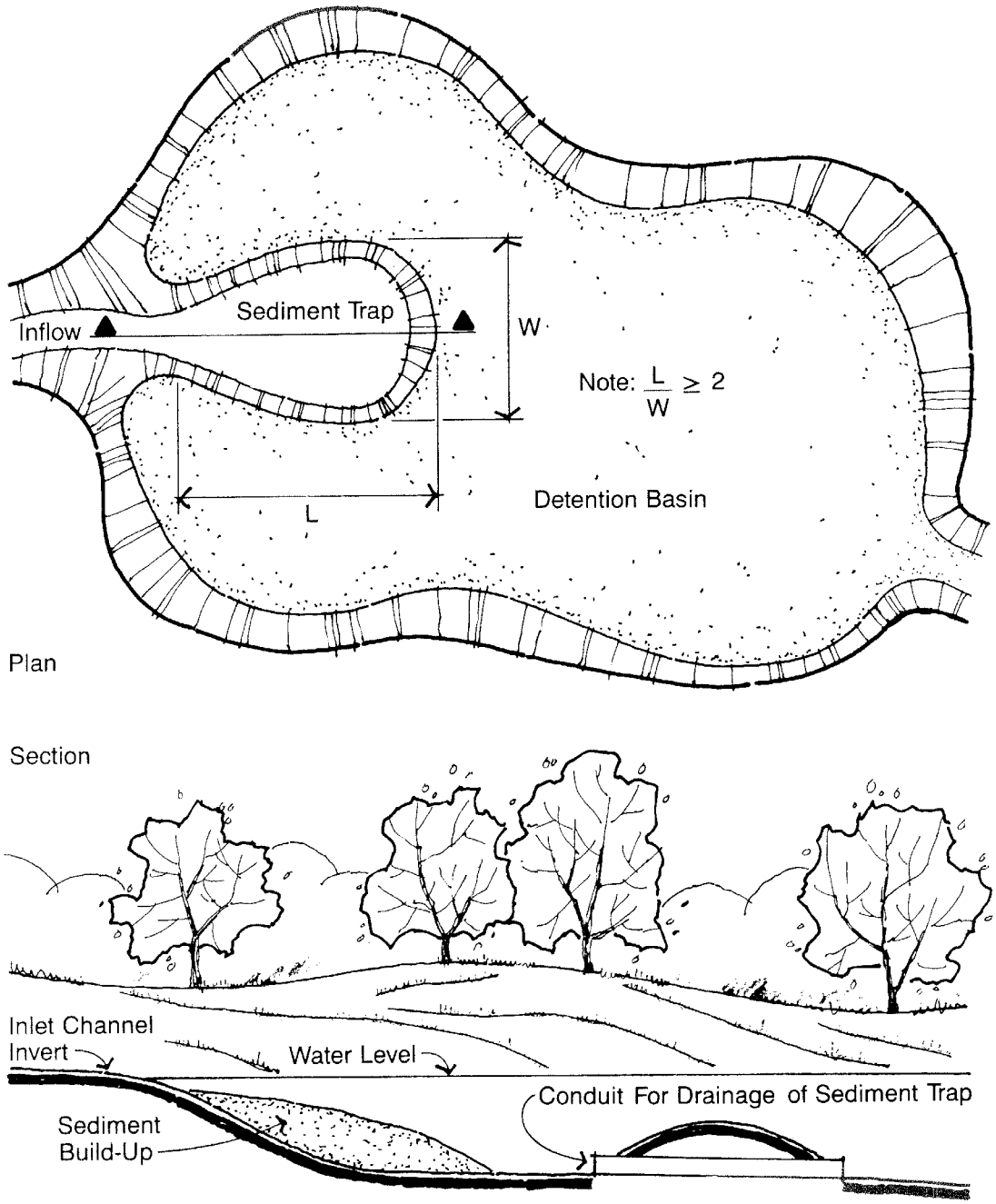


8.7.4 Method for Control of Sedimentation

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 8.12 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 8.12).
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 8.12
SEDIMENT TRAP CONCEPT
(Pima County Department of Transportation and Flood Control District, 1986)



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9 PUMP STATIONS

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9.1 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

9.2 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).

9.3 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.

9.3.1 Design Frequency

Pump stations must provide sufficient capacity to discharge the volume of storm runoff generated by the design storm.

9.3.2 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.

9.3.3 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 stormdrain 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 stormdrain system if pumped water can be discharged directly into a catch basin or other inlet.

9.3.4 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.

9.3.5 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.

9.4 DISCUSSION

9.4.1 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.

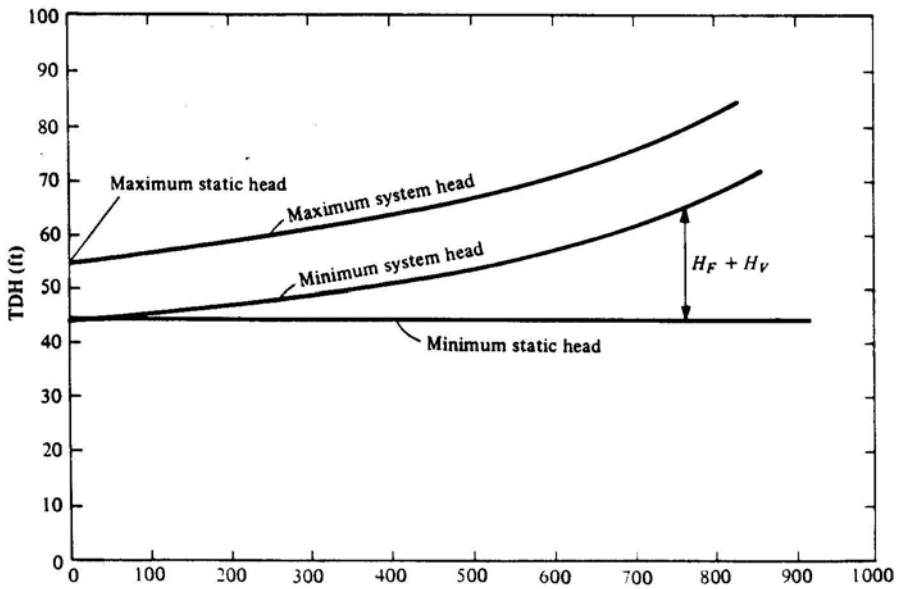
Possible requirements for future expansion must be considered in the planning. Expansion may be accomplished by modification of the existing units (such as increasing the pump impeller diameter), by adding pumps or by replacement of the original pumps. The last alternative is the least desirable one 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.

9.4.2 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 9.1.

FIGURE 9.1
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:

$$h = 2.87n^2 \frac{LV^2}{D^{4/3}} \tag{9.1}$$

where h is the head loss (ft), L is the pipe length (ft), D the pipe diameter (ft), V the flow velocity (ft/sec) and n 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.

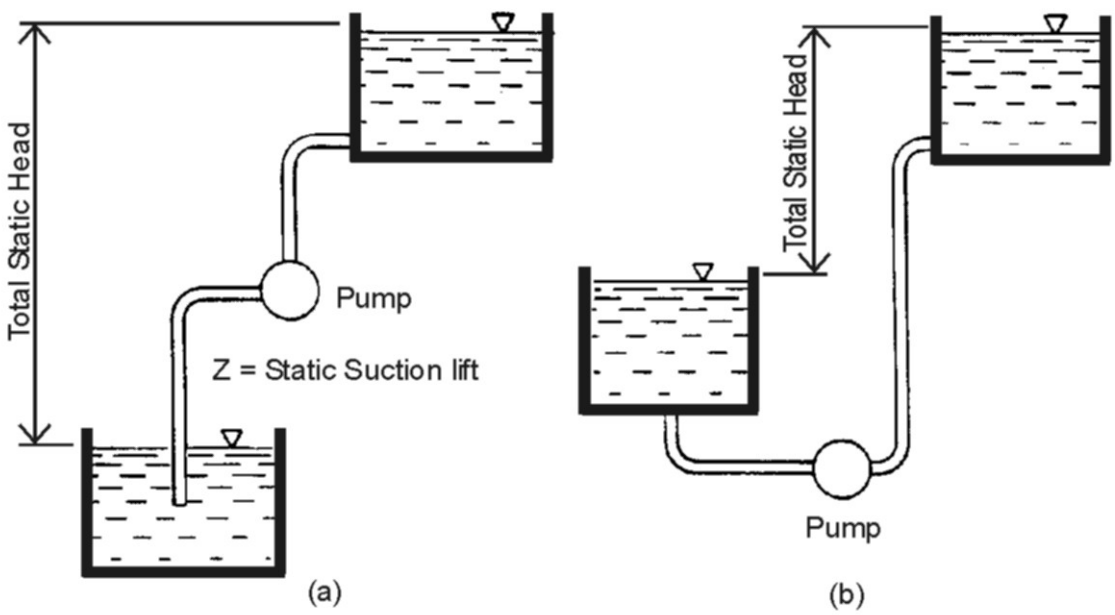
Minor losses are most often expressed as:

$$H_L = \frac{KV^2}{2g} \tag{9.2}$$

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."

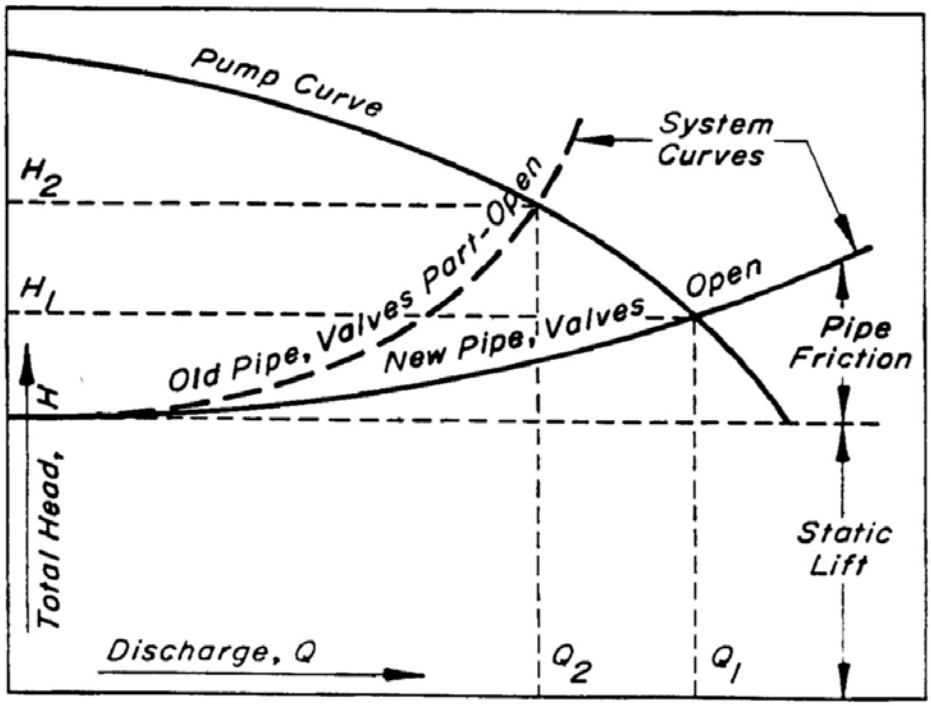
Static head is the difference in elevation between the water surface in the wet well and that at the discharge point. Storm water pumping systems are usually designed to discharge into a channel, conduit or receiving body at atmospheric pressure; in this case, the appropriate elevation to use is the centerline elevation of the effluent pipe (see Figure 9.2). Since the level in the wet well may vary, system head curves for maximum and minimum static heads are usually plotted.

FIGURE 9.2
TOTAL STATIC HEAD
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 9.3) 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 9.3
PUMP OPERATING POINT IN A GIVEN PIPING SYSTEM
(ASCE, 1975)



9.4.3 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.

9.4.4 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.

9.4.5 Piping and Valves

Suction inlets for dry pit pumps should be designed so as 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.

9.4.6 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 ground water levels, particularly maximum heights.
- Safety and environmental requirements of local codes should be met, including those concerning exhaust of internal combustion engines, if used.

9.4.7 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:

$$t = \frac{V_w}{P - Q} + \frac{V_w}{Q} \tag{9.3}$$

where t is the cycling time in minutes, P the pumping rate in gallons per minute, Q the inflow in gallons per minute and V 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:

$$V_w = \frac{t_m P}{4} \tag{9.4}$$

or for a 10 minute minimum

$$V_w = 2.5P \tag{9.5}$$

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.

9.4.8 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.

9.4.9 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.

9.4.10 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.

9.4.11 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.

9.4.12 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.

9.5 AIDS

TABLE 9.1
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 9.1
DESIGN CHECKLIST FOR PUMP STATIONS (CONTINUED)

Hydraulic Analysis
Mass Curve Routing
Outflow Hydrograph
Pump Characteristics
Pipe Losses
Miscellaneous Losses
Sediment Transport
Additional Considerations

9.6 REFERENCES

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Clark, John W., Viessman, Warren, and Hammer, Mark J., 1977, *Water Supply and Pollution Control*, Third Edition, Harper & Row.

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10 SEDIMENTATION

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10.1 SYMBOLS

γ	=	Water specific weight, lb/ft^3
γ_s	=	Sediment specific weight, lb/ft^3
σ_g	=	Geometric standard deviation of sediment particle size, mm
τ_o	=	Bed shear stress, lb/ft^2
B_f	=	Bulking factor
C	=	Sediment concentration, mg/l
d_5 d_{16} d_{50} d_{84}	=	Sediment particle size of which designated percentage is smaller than, mm
d_c	=	Sediment diameter at incipient motion, mm
d_g	=	Geometric mean sediment particle size, mm
d_h	=	Dune or antidune bedform height, ft
d_s	=	Characteristic bed material particle size, mm
F_r	=	Froude Number
F_*	=	Shields parameter
FS	=	Scour depth factor of safety
L_w	=	Watercourse length upstream of channel pivot point, ft
P_c	=	Fraction of bed material coarser than the armoring size
Q_s	=	Sediment bed material discharge, tons/day
Q_w	=	Water discharge, cfs
R	=	Hydraulic radius, ft
S	=	Watercourse slope, ft/ft

S_e	=	Energy slope of watercourse, ft/ft
v	=	Average velocity of flow, ft/sec
V_m	=	Total volume of water-sediment mixture
V_w	=	Volume of clear water
Y_a	=	Desired thickness of armor layer, ft
Y_s	=	Depth of scour to establish armor layer, ft
Y_h	=	Hydraulic depth, ft
Y_{max}	=	Maximum flow depth, ft
$Z_{bedform}$	=	Component of scour due to bedform, ft
Z_{bend}	=	Scour depth due to bend in watercourse, ft
$Z_{contraction}$	=	Scour depth due to contraction of flow, ft
$Z_{general}$	=	General scour component of scour depth, ft
Z_{local}	=	Local scour depth, ft
$Z_{long\text{-}term}$	=	Long-term degradation component of scour depth, ft
$Z_{low\text{-}flow}$	=	Incised low-flow channel component of scour depth, ft
Z_t	=	Total scour depth, ft

10.2 DEFINITIONS

Antidune	Alluvial channel bed form (see Section 10.11.2 for definition)
Avulsions	A sudden cutting off or separation of land by a flood or by an abrupt change in the direction of a watercourse whereby the watercourse deserts its old path for a new one.
Bar	Alluvial channel bed form (see Section 10.11.2 for definition)
Bed load	Sediment that moves by saltation, rolling, or sliding on or near the streambed.
Bed material	The sediment mixture of which the streambed is composed.
Bed material load	Sediment of particle size that is found in appreciable quantities in the shifting portions of the bed of the watercourse.
Bed or streambed	The bottom of a watercourse.
Bed load discharge	The quantity of bed load passing any cross section of a watercourse in a unit of time.
Bed material discharge	That part of the sediment discharge of a watercourse that is composed of particle sizes present in appreciable quantities in the shifting portions of a streambed.
Chute and Pool	Alluvial channel bed form (see Section 10.11.2 for definition)
Dune	Alluvial channel bed form (see Section 10.11.2 for definition)
Fluvial sediment	Fragmentary material that originates from weathering of rocks and is transported by, suspended in, or deposited from water.
Ripple	Alluvial channel bed form (see Section 10.11.2 for definition)
Sediment	Rock and mineral particles subject to movement by rainfall, runoff, streamflow and wind forces.
Sediment concentration	The volume weight of sediment discharge relative to the volume weight of the water and sediment discharge.
Sediment discharge	The quantity of sediment that is conveyed past any cross section of a watercourse in a 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 load	The sediment that is being moved by the flowing water of a watercourse. (Load refers to the material itself and not to the quantity being moved.)

Sediment yield	The total sediment outflow from a watershed or a drainage area at a point of reference and in a specified period of time. This is equal to the sediment discharge from the drainage area, assuming no sediment sinks upstream.
Sedimentation	Process of erosion, entrainment, transportation, deposition and compaction of sediment.
Stream discharge	The volume of water passing through a cross section of a watercourse in a unit of time. The water may contain both dissolved solids and sediment.
Suspended load	Sediment being transported in suspension in the flowing water of the watercourse.
Suspended sediment discharge	The quantity of suspended-sediment passing through a watercourse cross section in a unit of time.
Total load	Bed material load plus wash load.
Total sediment discharge	The total sediment discharge of a watercourse equals the suspended sediment discharge plus the bed load discharge.
Total sediment load	The total sediment load is the sediment moving as suspended load plus that moving as bed load.
Wash load	That part of the total sediment load that is composed of particle sizes not present in appreciable quantities in the bed sediment. Normally the wash load is considered to be particles that are finer than 0.062 mm.
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.

10.3 INTRODUCTION

Sedimentation and the fluvial processes associated with sediment transport play an important role in the long-term conveyance capacity of a drainage system as well as the on-going cost of maintenance. Sedimentation is a very complex subject and not all drainage designs need to consider it as a primary design criteria

Culvert sedimentation reduces conveyance capacity

This chapter provides the basic concepts of sedimentation and sediment transport. It provides some analytical methods, but perhaps more importantly, directs the interested reader to publications dealing with the processes involved. Specific references are provided throughout this chapter. Some useful general references in the topic of erosion and sedimentation include ASCE (1975), ADWR (1996), Richardson and others (2001), Simons and Senturk (1992), ADWR (1985), Guy (1970), Henderson (1987), Schumm (1977), SCS (1977), and U.S. Army Corps of Engineers (1994).

For those versed in this subject, this chapter identifies a checklist of issues for consideration during the design of drainage facilities, a three tier approach to analysis, and the specific requirements of the Flood Control District of Maricopa County (the lead agency for review of sediment transport analysis). Those readers should turn to Section 10.14.

10.4 NATURE OF EROSION AND SEDIMENTATION

10.4.1 General

Erosion and sedimentation is a geologic process that is altered, positively or negatively, by the impact of man. That impact is either from activities that disturb the land surface, either directly, such as by construction, or indirectly by the use of the land, such as intense recreational use. Erosion and sedimentation is a continual process of sediment production, transport and redistribution. Thus it is a process that involves erosion (scour), sediment transport, and deposition (fill). Water is the primary element in erosion and sedimentation. The planning, design, construction and maintenance of stormwater drainage projects must consider erosion and sedimentation.

10.4.2 Sediment Production

Erosion and sedimentation that occurs under natural conditions is called geologic erosion. Geologic erosion is a major factor in the shape and character of much of the earth's surface.

Geologic erosion will continue with increased or decreased rates depending upon weather, vegetation, soil conditions, land use, and geologic and topographic factors. In Maricopa County, where there are extensive deposits of geologically young rock, annual precipitation is low, vegetation is sparse and rainfall intensities can be extreme, the rate of geologic erosion is high. Watercourses in Maricopa County are typically heavily laden with sediment.

Although control of geologic erosion is limited to relatively small areas and can only be effective for a short time (within the context of geologic time), it can be undertaken with some degree of success. For example, erosion and sedimentation can be controlled, although it is better to think in terms of management rather than control, by sedimentation basins, watercourse stabilization, bank lining, grade control structures, revegetation, and other structural and nonstructural methods.

Erosion that occurs beyond the rate of geologic erosion is accelerated erosion. That increase over natural erosion is the result of some activity by man. Examples of activities that generally result in accelerated erosion are; land grading, vegetation removal, disruption of the soil surface, and increasing runoff in quantity or duration. Accelerated erosion also occurs where there is an excessive concentration of recreational land use. The cumulative impact of several individual activities may lead to vastly accelerated erosion rates.

Land development and urbanization is a major cause of localized accelerated erosion. That is often due to two factors; first, disruption of the land surface including vegetation removal, and second, increased and concentrated runoff. This is occurring in parts of Maricopa County with current land development and landscape practices. During construction, erosion rates are usually high, due to the removal of vegetation, excavation, and grading activities. Best management practices for erosion control are required during construction.

Agriculture is the leading factor, in terms of quantity of sediment produced, for accelerated erosion. Cultivation by mechanical equipment causes drastic soil disturbance and removal of protective vegetation cover. Livestock grazing often results in diminished vegetation cover and with poor management may result in denuding the land surface. Under severe grazing conditions the sediment production can be greatly increased for small areas, but conclusive evidence that well managed livestock grazing over large areas produces accelerated erosion is difficult to verify. In Maricopa County, much of the agricultural land is irrigated and uncontrolled runoff is often minimized by irrigation berms, canals, drains, and so forth. However, abandoned agriculture land that is no longer maintained may produce high erosion rates locally.

Runoff from roadway surfaces and the concentration of runoff through cross-drainage structures can result in accelerated erosion. Local sediment problems can result during construction, but are typically minimized by use of best management practices. Roadway drainage needs to be carefully planned and designed to avoid drainage induced erosion and sediment problems.

10.4.3 Sheet and Rill Erosion

Sheet erosion is the removal of small amounts (per unit area) of soil by overland runoff, and includes rill erosion unless otherwise specified. Rill erosion is the removal of soil in small channels that can be smoothed completely by normal cultivation methods. Any of the following activities on the land surface will increase sheet and rill erosion:

- Increase in intensity of runoff.
- Diminished rainfall losses.
- Reduction in vegetation cover.
- Reduction of rock and litter ground cover.
- Tillage or other reduction in soil compaction.
- Increase in land surface slope.

Methods to estimate sheet and rill erosion are discussed in Section 10.10, Watershed Sediment Yield.

Rill Erosion

10.4.4 Gully Erosion

Gully erosion is the removal of soil by concentrations of surface runoff “sufficient to cause the formation of channels that can not be smoothed completely by normal cultivation methods” (ASCE, 1975). Gully erosion has the following characteristics:

- A normal landform that may contribute to geologic erosion in certain regions.
- Can be exacerbated by improper land use.
- Can be continuous or discontinuous.
- Can jeopardize structures and roadways.
- Can be a major source of sediment production during severe storms.

Gully Erosion

10.4.5 Channel Erosion

Channel erosion includes erosion from the bed and the banks of the watercourse. Under certain conditions, this erosion can be significant. For example, large and sustained peak flows can appreciably accelerate erosion. Erosion potential is also accelerated if the flow is contracted such as by watercourse encroachment. Erosion is also increased due to local scour. Bank scour can lead to lateral migration of the watercourse or be contributary to avulsions during floods.

10.5 SEDIMENTATION ISSUES

10.5.1 Sand and Gravel Mining

Sand and gravel mining located within floodplains may also represent a source of large amounts of sediment. Mine excavations may also act as a sediment sink depending upon the geometry of the mine pit and quantity of flow. The fluvial processes in and around sand and gravel mining facilities during times of flood are complex and beyond the scope of this chapter. Technical guide-

lines and procedures for analyzing sand and gravel mining and its potential impact on watercourses are provided by U.S. Army Corps of Engineers (1980 and 1987), Li and others (1989), and Urban Drainage and Flood Control District (1987). Erosion and sediment control methods for surface mining are available in EPA (1976).

10.5.2 Watercourse Stabilization

Any watercourse modifications affecting the flow direction, depth, velocity or duration of discharge may result in erosion and sedimentation. This applies not only to flood control and drainage facilities, but any structural works to a watercourse may have discernable and potentially deleterious impacts regarding erosion and sedimentation to the watercourse. The following is a partial list of watercourse modifications and potential impacts:

- Channel straightening, will generally increase channel gradient and flow velocity, and may initiate channel erosion.
- Channel constriction increases flow velocities and often flow depth, thus increasing sediment transport capacity and may initiate channel erosion.
- Lowering the bed elevation of a watercourse may prompt degradation in the mainstem of the watercourse and its tributaries.
- Raising the bed elevation or reducing the slope of the energy grade line may result in sediment deposition upstream due to reduced transport capacity.
- Bank lining may increase flow velocity and increase erosion and/or bank attack where banks are left unprotected.

Local scour caused by concentrated flows and more frequent high flows

Failed riprap armoring at end of concrete apron of box culvert

Alluvial channels are often in a balanced state of dynamic equilibrium (see Section 10.8) and even subtle changes in water discharge, watercourse hydraulic characteristics or sediment properties may result in erosion or sedimentation that is initiated promptly and proceeds rapidly. Accelerated erosion and sedimentation results in the destruction of natural conditions in the watercourse, may seriously depreciate land values by unsightly erosion and lost land, may decrease flood conveyance capacity by sediment deposition, and often requires expensive structural mitigation measures to restore the aesthetics and function of the watercourse.

10.5.3 Stormwater Storage

Stormwater storage facilities must be planned and analyzed in regard to their impact on watercourses. Those impacts are generally viewed within the context of the following:

- Sediment deposition in the impoundment and upstream backwater of the impoundment. This may result in decreased upstream conveyance capacity, the potential for breakout flows due to sediment deposits, and maintenance requirements in regard to sediment deposits.

- Release of “clear water” downstream of the impoundment. This may result in local scour and/or degradation of the downstream watercourse. Reduction of the continual replenish-

ment of finer sediments may result in a changed character of the watercourse including the bed becoming more “cobbly” and loss of riparian vegetation.

- Although the peak discharges are usually reduced downstream of the storage facility, the duration of high flows often increases. This may increase the opportunity for scour and the flushing of finer sediments through the system.

10.5.4 Water Quality Issues

Sediment in water is often viewed as an undesired element for municipal and industrial water. The undesirable characteristics are often due to its quantity (volumetric) and its abrasive nature (size gradation and hardness).

Chemicals such as pesticides, herbicides, organic compounds such as nitrogen and phosphorus, pathogens, and waste products can become attached to sediment particles and thereby be transported and stored along with sediment. Sediment can also be an asset such as for irrigated agriculture and for certain riparian habitats.

10.6 CHANNEL PROCESSES

10.6.1 General

Watercourses are either erodible or nonerodible, and either natural or artificial. In general, this section considers erodible channels, although sedimentation of nonerodible channels must also be considered as discussed in the next paragraph. Natural channels are those for which the form and dimensions are the result of a natural process. Artificial channels are constructed to predetermined dimensions and alignment. Natural channels are often analyzed for erosion and sedimentation under changed conditions, such as increased future flood flows. Artificial channels are typically designed based on stable dimensions and balance of sediment transport. Occasionally the question arises as to the design of modifications to natural channels such as local scour protection for structures built within the watercourse. Both natural and artificial channels must be considered in regard to sedimentation.

Although sedimentation, as defined herein, is limited to erodible channels, constructed channels of nonerodible material must be analyzed under reasonable conditions of sedimentation and sediment transport. For example, a fully lined concrete channel may have uncontrolled local runoff that can introduce a large amount of sediment load into the channel. Deposition of sediment in the channel can diminish the flow area, and also increase the resistance to flow. Under such conditions, the conveyance capacity of a channel can be reduced significantly. Therefore, constructed, nonerodible channels in Maricopa County often must be analyzed and designed under conditions of sediment transport.

10.6.2 Regime

Many natural watercourses are in equilibrium with the major hydraulic parameters of width, depth, velocity and slope remaining constant even though erosion and sedimentation is occurring. Those watercourses are in dynamic equilibrium with realignments and bank erosion maintaining an overall system balance. Sediment inflow to a reach equals, on the average, sediment outflow. Such watercourses are said to be “in regime.” Any factors or activities that would tend to upset these balance relations may result in the watercourse seeking a new regime.

The “in regime” approach is a means to describe watercourse form in terms of time-average magnitudes. A historic perspective of regime equations is provided by Schumm (1971), and practical guidance for selected regime equations are provided by Leopold and Maddock (1953) and Mahmood and Shen (1971). The regime concept is based on the premise that water discharge is the controlling factor in watercourse form. However, other factors such as sediment yield, upstream channelization, land use, vegetation and climate changes affect watercourse form.

Furthermore, the applicability of regime equations must be made with care since they are empirical. Regime equations should be applied only to watercourses that are “in regime.” Stevens and others (1975) suggest that watercourses can be classified as “in regime” or not based on flood hydrology; watercourses with a low ratio of peak-flood discharge to the average annual peak-flood discharge can be in regime and regime-type equations applied. However, if the ratio is large, then the watercourse should exhibit nonequilibrium form for which regime equations do not apply. Stevens and others (1975) illustrate major watercourse widening and narrowing of the Gila River in the Safford Valley, Arizona, for a 125 year period, and during that time the ratio of peak-flood discharge to average annual peak-flood discharge was as large as 10. Based on stream-form data, they conclude that the Gila River, where such conditions occur, is not in quasi or dynamic equilibrium and is not “in regime.” Most watercourses in Maricopa County, except possibly those for which flood peaks are controlled by upstream regulation, are expected to have large ratios of peak-flood discharges to average annual peak-flood discharge. Therefore, regime equations should be applied carefully, if at all.

10.6.3 Aggrading and Degrading Watercourses

Aggradation is the raising (filling) of the bed of a watercourse over some reach length as a result of incoming volume of sediment to the reach exceeding the outgoing volume of sediment. Degradation is the lowering (cutting) of the bed due to outgoing volume of sediment exceeding the incoming volume of sediment. The aggradation or degradation process will continue until a new dynamic equilibrium is established and the watercourse is back “in regime.”

Long term bed elevation changes (aggradation or degradation) may be the natural trend of the watercourse or may be the result of some modification to the watercourse or watershed condition. Factors that affect long term bed elevation changes are; dams and reservoirs (upstream or downstream), changes in watershed land use (urbanization, deforestation, etc.), channelization, cutoff of a meander bend (natural or man-made), changes in the downstream base level (control), gravel mining, diversion of water into or out of the watercourse, natural lowering of the total system, and lateral watercourse movement such as bank erosion or migration.

Watercourse aggradation

10.6.4 Stream Forms

Watercourses are classified as straight, braided or meandering. Descriptions of these stream forms and quantitative criteria are presented for each in ASCE (1975). Straight channels have the following characteristics:

- Essentially a straight alignment (low sinuosity).
- May have very flat slopes with nonerodible velocities.
- May have very steep slopes with high momentum that resists alignment changes.

Braided channels have the following characteristics:

- Formed by random interconnected channels separated by sand or gravel bars.
- Braided channels are often aggrading and may occur on flat, steep or moderate slopes.
- Often have high bed material transport during floods.
- Deposits form bars that are often vegetated.
- Distributary networks are types of braided stream form that occur on alluvial fans.

Flooding, erosion and sedimentation on alluvial fans is beyond the scope of this manual. The reader should consult the District's *Piedmont Flood Hazard Assessment for Flood Plain Management for Maricopa County* (Hjalmarson, 1998) and the National Research Council (1996).

Meandering channels have the following characteristics:

- Follow a winding course.
- Alignment tends to shift continuously by local erosion and opposing bank building.

Typical piedmont watercourse depicting coarse sand bed material.

10.7 SEDIMENT PROPERTIES

10.7.1 General

Sedimentation is a function of the flow in the watercourse, that is, the discharge and hydraulics of the conveyance system, and also of the properties of the sediment itself. The sediment properties of interest to typical fluvial sedimentation are; particle size, particle size distribution, fall velocity, specific gravity of sediment particles and specific weight of sediment deposits.

10.7.2 Sediment Particle Size

The commonly used sediment particle size scale and sieve number for sands and smaller gravel are shown in Table 10.1.

TABLE 10.1
SEDIMENT GRADE SCALE
(Lane, 1947; ASCE 1975)

Class name (1)	Size Range: Millimeters (2)	Size Range: Inches (3)	Approximate Sieve Mesh Openings per inch: Tyler (4)	Approximate Sieve Mesh Openings per inch: 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		
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			

Sediment size is measured by a length scale or “diameter.” Two commonly used sediment size diameters as defined by ASCE (1975) are:

Sieve diameter - The length of the side of a square sieve opening through which the given particle will just pass, and

Sedimentation diameter - The diameter of a sphere of the same specific weight and the same terminal fall velocity as the given particle in the same sedimentation fluid.

The size of sand and larger sediment particles is usually expressed as sieve diameter. The size of silts and clays is generally expressed as a sedimentation diameter.

10.7.3 Size-Frequency Distributions

Natural sediments are made up of grains with wide ranges of size. Statistical methods are used to describe the size distribution. The size distribution of sand and larger particles is obtained by mechanical sieve analyses. For silt and smaller particle sizes a fall velocity method is typically used. Size gradation is usually presented as cumulative size-frequency curves, where the fraction or percentage by weight of sediment that is smaller or larger than a given size is plotted against the size. Figure 10.1 is a typical size distribution graph. The data are plotted on a semi-logarithmic graph. The median size, d_{50}, that is, the size for which 50 percent of the material is finer, can be read from the curve. Other values of interest are d_{16}, d_{84}, d_5 and d_{95}, defined similarly as d_{50}. The geometric mean size, d_g, is estimated as:

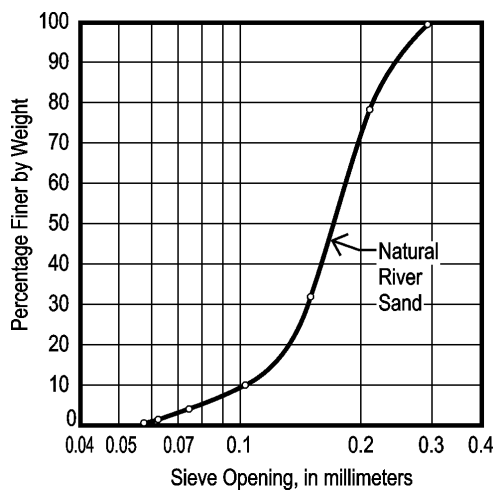
$$d_g = (d_{84}d_{16})^{1/2} \tag{10.1}$$

and the geometric standard deviation σ_g as:

$$\sigma_g = (d_{84}/d_{16})^{1/2} \tag{10.2}$$

These statistics and others (see ASCE, 1975) are commonly used to describe the size-frequency distribution of sediment.

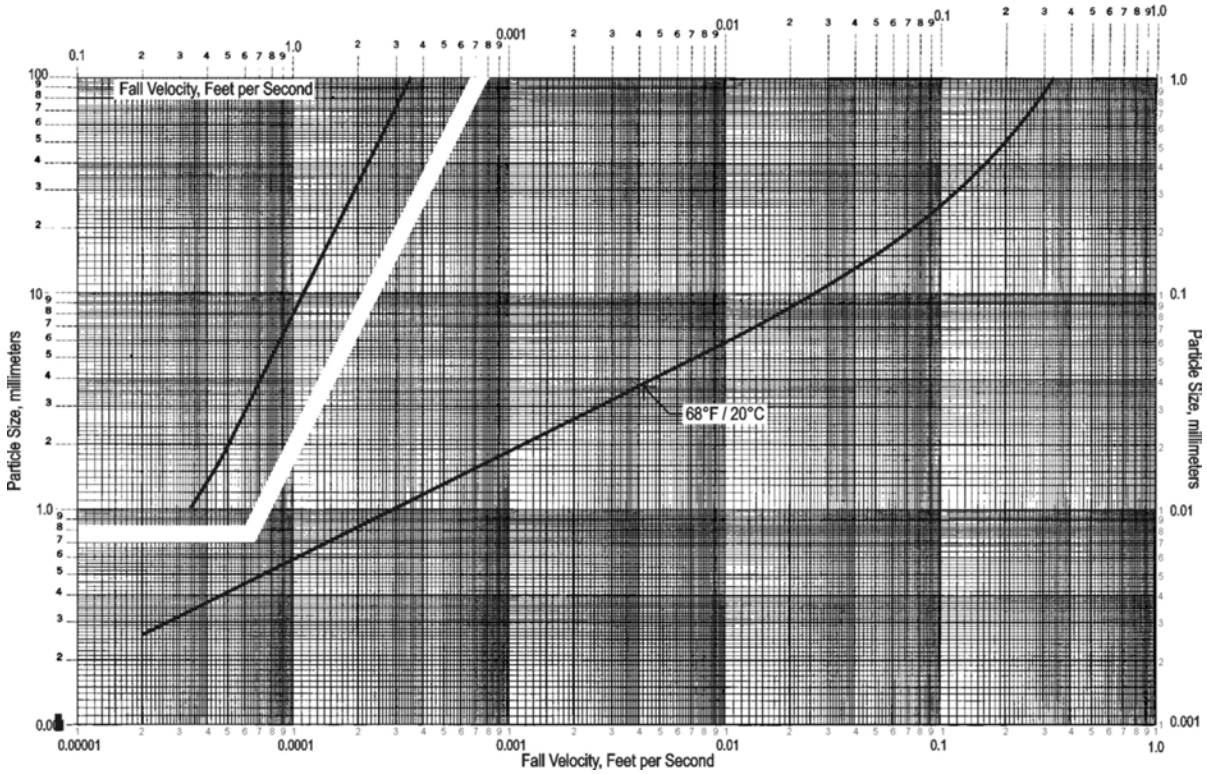
FIGURE 10.1
CUMULATIVE SEMILOGARITHMIC SIZE-FREQUENCY CURVE



10.7.4 Fall Velocity

The fall velocity of sediment is a function of particle size and shape, specific gravity, water temperature, and concentration of sediment in the water. For typical engineering sedimentation studies, the fall velocity can be estimated by Figure 10.2. It is noted that fall velocity is usually expressed for quiescent fluid conditions and there is a tendency for the fall velocity to decrease in turbulent flows.

FIGURE 10.2
SEDIMENT FALL VELOCITY CURVE FOR TYPICAL SEDIMENTS IN WATER AT 68°F
(Pemberton and Lara, 1971)



10.7.5 Specific Gravity of Sediment Particles

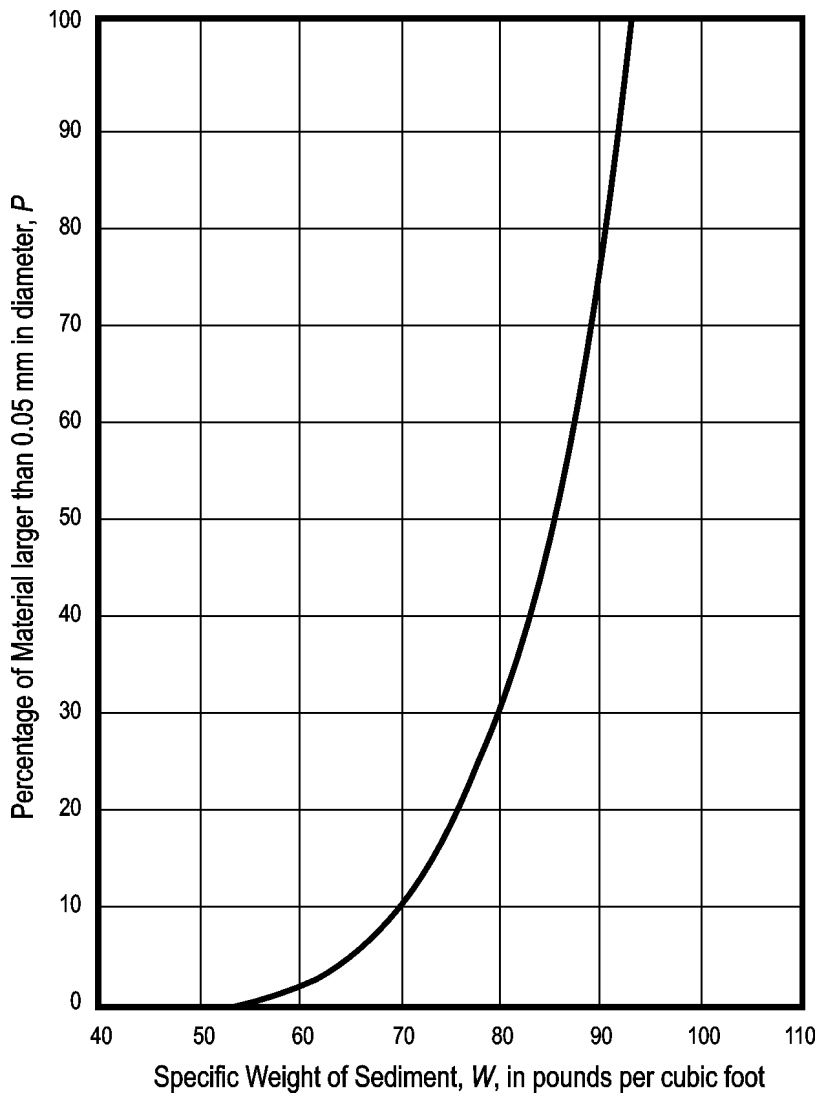
Sediment is rock material that is produced by weathering and abrasion. Less competent material in the rock decomposes more readily into smaller sizes. Coarser sediments decompose more slowly and may be pieces of the parent rock. Sand is most often composed of small quartz particles. Quartz has a specific gravity of 2.65 and that value is often used for sediment. 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. Specific gravity is important to the sedimentation process and should be measured if an untypical sediment source is suspected.

10.7.6 Specific Weight of Sediment Deposits

Specific weight of a sediment deposit is the dry weight of the sediment within a unit volume (including pore space of the sediment mass), in pounds per cubic foot. Specific weight is of particular interest when estimating depletion of storage volume in stormwater storage facilities. Specific weight varies over a large range due to sediment properties and hydraulic factors. The age of the sediment deposit can also be a factor for deposits of fine material, but generally is unaf-

fected by time for deposits of coarse sand and larger particles. A relation for specific weight of sediment deposited to percentage of sand is provided in Figure 10.3.

FIGURE 10.3
RELATION OF SPECIFIC WEIGHT OF SEDIMENT DEPOSITS TO PERCENTAGE OF SAND
(Modified from Lane and Koelzer, 1953; ASCE, 1975)



10.8 EQUILIBRIUM CONCEPT

Watercourses tend to adjust their physical characteristics toward a state of dynamic equilibrium such that their ability to transport sediment is in balance with the amount of water and sediment that is delivered to the watercourse. Dynamic equilibrium is achieved when the amount of sediment entering a reach is equal to the amount leaving. The concept of dynamic equilibrium must be understood within the context of long-term trends. That is, any particular discharge event

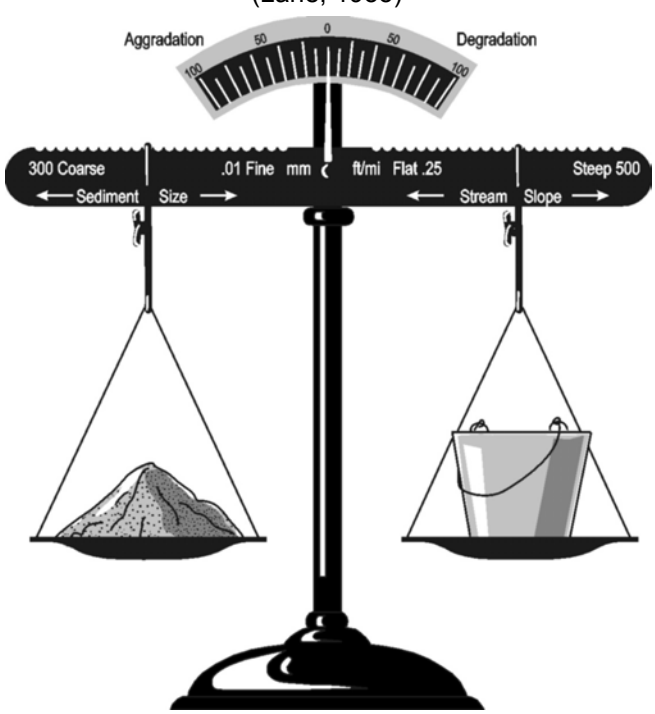
within a watercourse may not result in balanced sediment inflow and outflow; however, if the watercourse is in dynamic equilibrium, that balance is achieved during a reasonable time span of discharge events. Adjustments to the watercourse to achieve dynamic equilibrium are achieved in several ways, including bank erosion or fill, change in bed material size gradation and/or changes in bed slope (aggradation or degradation).

This concept is illustrated by Lane's balance (Lane, 1955) as shown in Figure 10.4, and the qualitative equation:

$$Q_w S \propto Q_s d_s \tag{10.3}$$

where Q_w is water discharge, S is longitudinal slope of the watercourse, Q_s is bed material discharge, and d_s is a characteristic bed material particle size. Alteration of one variable, as illustrated by the "balance" of Figure 10.4, 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 of one variable by a change in the fourth variable.

FIGURE 10.4
STABLE CHANNEL BALANCE
(Lane, 1955)

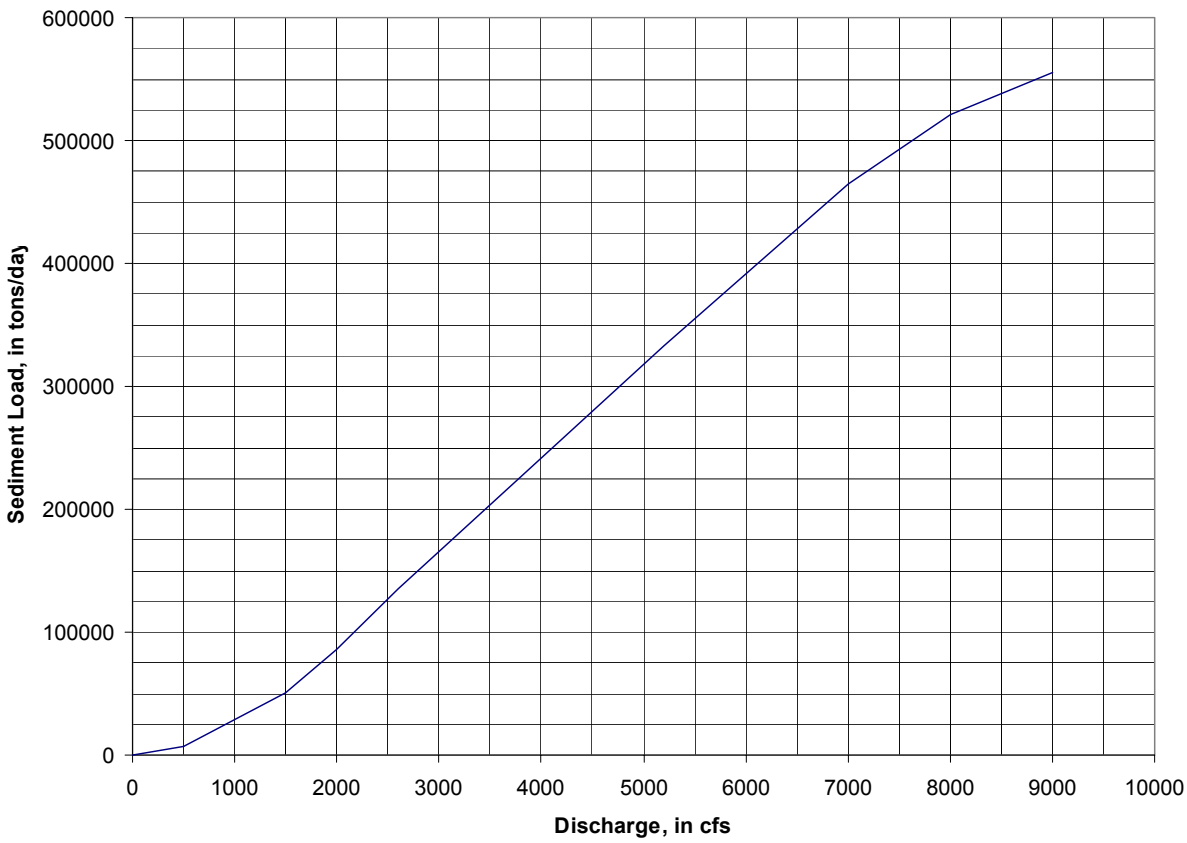


10.9 SEDIMENT DISCHARGE

10.9.1 Sediment Discharge Rating Curves

For Maricopa County, there is a scarcity or complete lack of adequate sediment discharge and corresponding storm runoff data. Therefore, it is often necessary to estimate sediment yield from the watershed and to investigate the sedimentation aspects of the drainage design or study by analytic or indirect empirical methods. Techniques of sediment transport modeling, such as with HEC-6 (USACE, 1991), are often used in such analyses. Sediment inflow relations are generally required for such analyses, 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). Such a rating curve is illustrated in Figure 10.5.

FIGURE 10.5
EXAMPLE OF A SEDIMENT DISCHARGE RATING CURVE



10.9.2 Sediment Concentration

Sediment discharge is a function of water discharge as expressed by

$$Q_s = 0.0027CQ_w \tag{10.4}$$

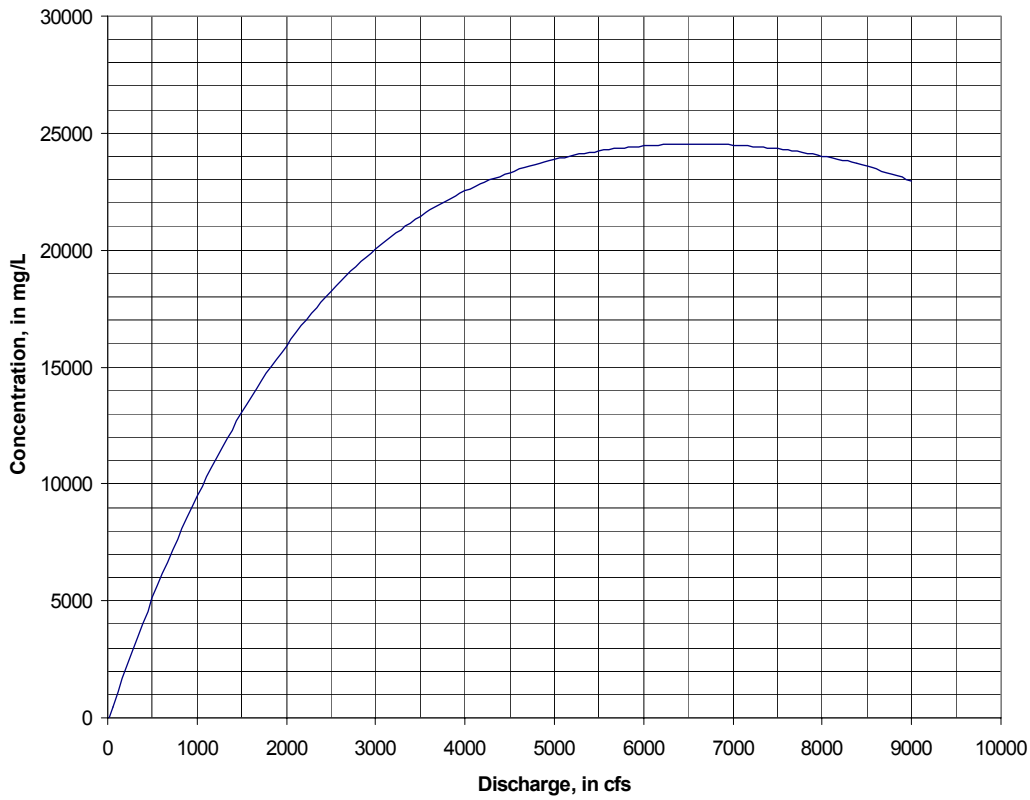
where Q_s is sediment discharge in tons per day, Q_w is water discharge in cfs, C is concentration of sediment in mg/l, and 0.0027 is a unit conversion factor (Porterfield, 1972). The sediment discharge rating curve should always be inspected by calculating and plotting C as a function of Q_w. The concentration of sediment will not always increase with increasing discharge. Generally the sediment discharge will be the greatest during the rising limb of the flood hydrograph as the watercourse is “flushed” of previous sediment deposits in the main channel and the overbank floodplain from lesser floods. Depending on the magnitude of such deposits and the shape of the flood hydrograph, the concentration of sediment discharge may decrease for water discharge above some level (see Figure 10.6). These phenomena can also be attributed to dilution of the sediment load for large floods. Inspection of the sediment concentration curve should always be performed. Qualitative inspection of the curve will provide some confidence that reasonable sediment discharges are being input to the analytic models and that reasonable results are being produced by the models. It is difficult to generalize these sediment concentration relations, however, concentrations in excess of 100,000 mg/l (ppm) of total load would be very high, but within reason under appropriate conditions. Concurrently, maximum concentrations of less than 10,000 mg/l for "typical” watercourses in Maricopa County during major floods would probably be low. It must be noted however, that many analytic techniques and sediment transport models are for only one component of the sediment load. For example, the sediment transport function may be limited to estimating the bed material load or to only the sand-sized fraction of the bed load. Often the bed material load is only a small fraction of the total sediment load, maybe only 10 percent. Therefore, two points must be carefully understood:

1. The limitations and applications of the sediment transport analysis or models used, and

2. A judgement based qualitative and quantitative estimation of the general sediment discharge relations. Regional experience and judgement based on past experience is necessary.

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 10.6
EXAMPLE OF A SEDIMENT CONCENTRATION RATING CURVE



10.9.3 Sediment Discharge Characteristics

Bed material sediment discharge is limited by either the sediment transport capacity of the watercourse (transport control), or by the amount of sediment available to the watercourse (supply control). Transport control is dictated by hydraulic conditions in the watercourse. Supply control is dictated by conditions of sediment yield to the watercourse.

The concentration of sediment discharge can be used to classify flows (O'Brien, 1986), as illustrated in Table 10.2.

TABLE 10.2
WATER AND SEDIMENT FLOW CLASSIFICATION:

Type of Flow (1)	Concentration Range, in mg/l (2)
Water flood	0 - 410,000
Mud flood	410,000 – 650,000
Mudflow	650,000 – 730,000
Landslide	730,000 – 880,000

In general, floods in Maricopa County will produce total sediment load concentrations well below 410,000 mg/l; however, steep hillslope processes could produce mud flood, mudflow and, conceivably, landslide conditions.

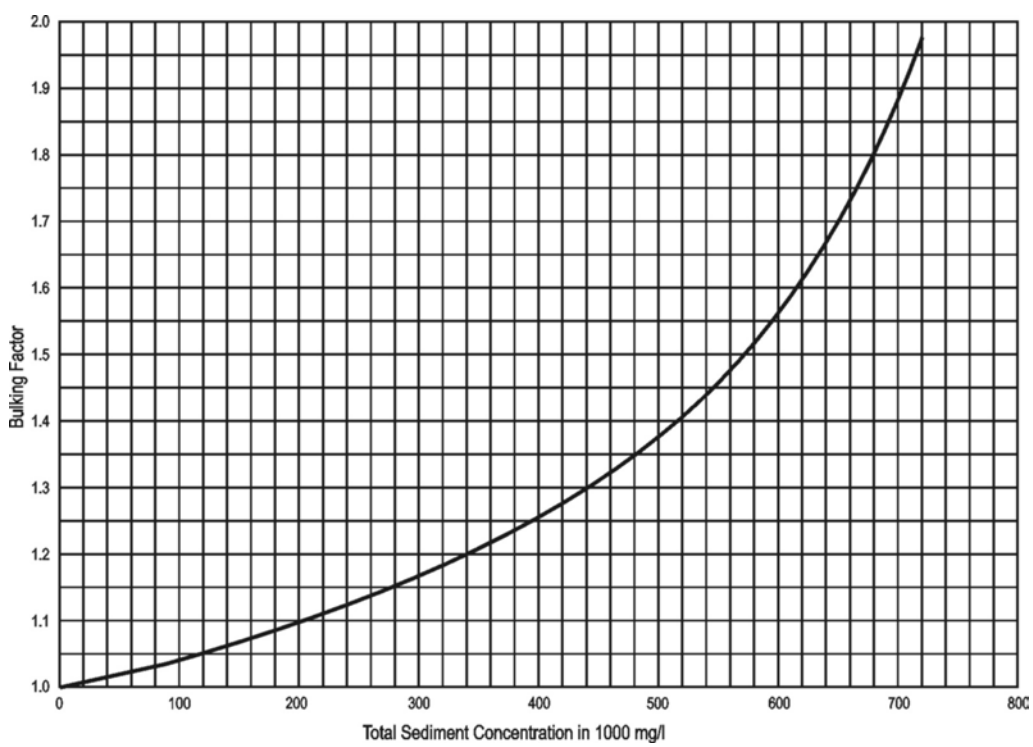
10.9.4 Sediment Bulking

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:

$$V_m = B_f V_w \tag{10.5}$$

where V_w is the clear-water volume and B_f is the bulking factor (Albuquerque Metropolitan Arroyo Flood Control Authority, 1994). The relation between total sediment concentration and the bulking factor is given by Figure 10.7. 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 must accommodate the bulked volumetric discharge.

FIGURE 10.7
RELATIONSHIP BETWEEN TOTAL SEDIMENT CONCENTRATION AND BULKING FACTOR
(Albuquerque Metropolitan Arroyo Flood Control Authority, 1994)



10.10 WATERSHED SEDIMENT YIELD

10.10.1 General

Sediment yield is a measure of the sediment production from a watershed exiting the 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). Since sediment yield increases with increasing time duration, the yield is usually expressed in terms of annual average or per specific flood event or flood duration. An aid to analyzing sediment yield data is to convert it to a value per unit of drainage area (acre-feet per square mile per year).

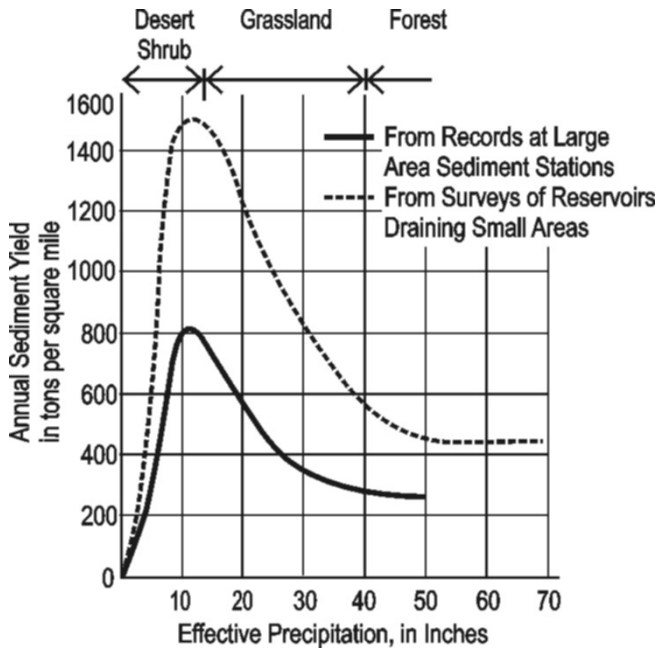
Sediment yield is dependent upon the rate of total erosion within the watershed and the efficiency of transport of those eroded sediments through the drainage network. Erosion and transport factors are widely variable, and therefore, measures of sediment yield are broadly generalized.

Deposition upstream of roadway embankment/culvert

Sediment yield is highly dependent upon vegetation cover and precipitation. Langbein, and Schumm (1958) illustrates in Figure 10.8 a trend of increasing sediment yield with increasing annual 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 10 to 15 inches of annual precipitation range. Notice in Figure 10.8 that the sedi-

ment 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 10.8
SEDIMENT YIELD, AS AFFECTED BY CLIMATE
(Langbein and Schumm, 1958)



10.10.2 Analytic Methods to Estimate Sediment Yield

Numerous methods are available for estimating sediment yield by analytic methods; see for example, Pacific Southwest Inter-Agency Committee (1974). A commonly used procedure 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 use of any of these methods is subjective to the selection of the input parameters. In practice, more than one analytic method may be used and the results compared. The use of empirical data or regional familiarity should be used in accepting results of these analytic methods.

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 and others (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 serve as a reminder 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 run-off from urban development crosses unprotected soil.

10.10.3 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 10.3.

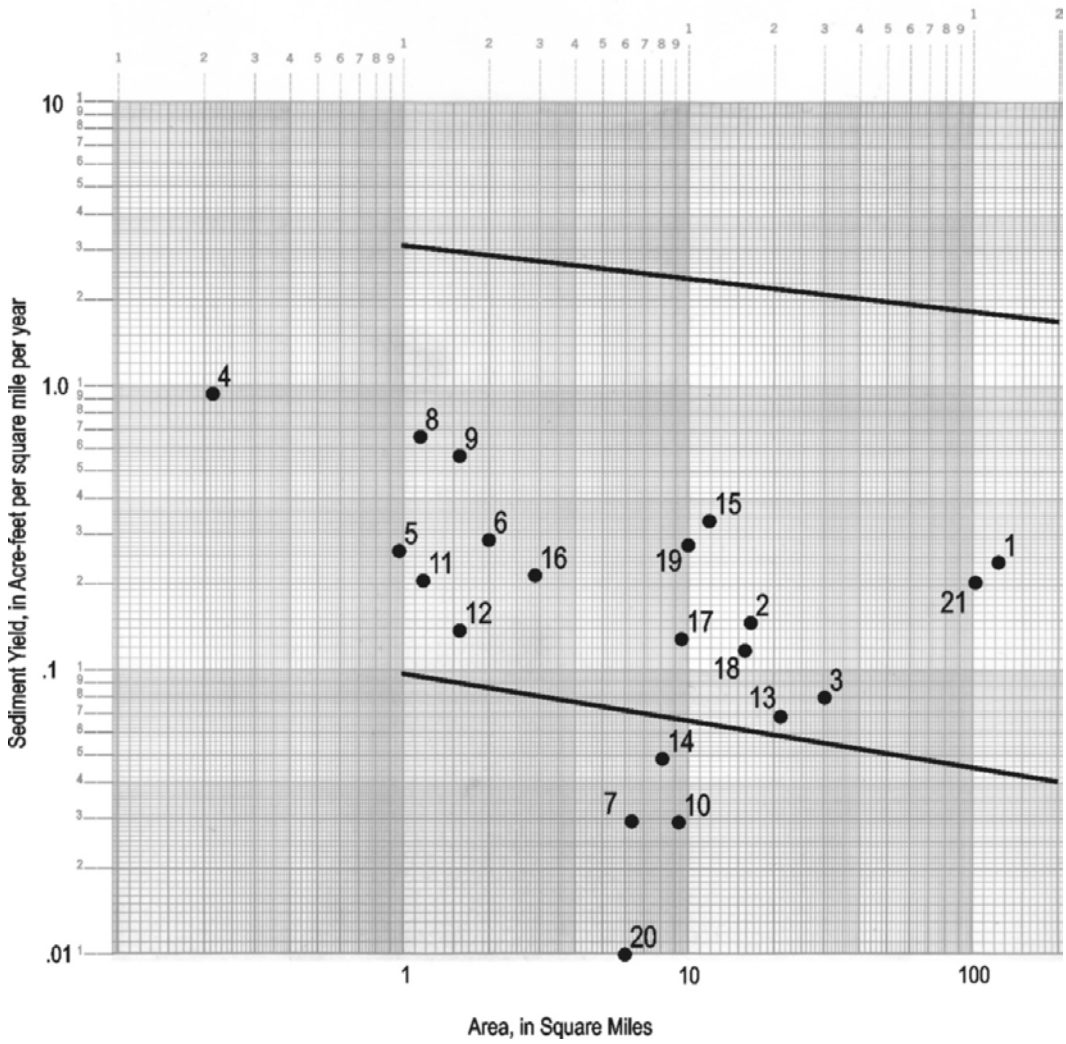
TABLE 10.3
MEASURED SEDIMENT YIELD FROM REPRESENTATIVE WATERSHEDS
(Representative of Conditions to be Expected in Maricopa County, Arizona).

No.	Location	Drainage Area sq. miles	Sediment Yield ac-ft/sq. mi./year	Reference
(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
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 Arroyo, 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	

A – U.S. Army Corps of Engineers, 1974
B – USDA, Natural Resources Conservation Service file data
C – Peterson, 1962
D – Langbein and others, 1951
E – Albuquerque Metropolitan Arroyo Flood Control Authority, 1994

The sediment yield data from Table 10.3 are plotted in Figure 10.9 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. Although at the time (1970), sediment yield for the immediately upstream Cave Creek Dam was only 0.24 acre-feet per square mile per year. The larger value (0.30) 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.

FIGURE 10.9
REGIONAL SEDIMENT YIELD AS A FUNCTION OF DRAINAGE AREA
LINES INDICATE ENVELOPE FOR 51 U.S. WATERSHEDS BY GLYMPH (1951)



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 explained by soil conditions, precipitation, and watercourse conditions among other things. 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 due to the well-developed soil covered with desert pavement. The differences of sediment yield are also related to climate differences. For example, certain watersheds in San Diego County, CA reflect yields of only 0.07 and 0.13 acre-feet per square mile per year due to the low annual precipitation of only 3 inches. Some sites with a large sediment yield such as Davis Tank, AZ are known to have watercourse bed and bank erosion. Lastly, other sites with relatively high yield such as Black Hills Tank, AZ may have experienced a large flood during a short period of data collection.

Runoff and sediment yield data were collected at the Black Hills Tank, near Cave Creek, Arizona, from 1945 to 1948 (Langbein and others, 1951, and 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. 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 and others (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 10.3 for Black Hills Tank for the 4-year period probably is too high because of the 1948 flood. However, that data does indicate the magnitude of sediment that can be produced from a single intense runoff event.

10.11 SEDIMENT TRANSPORT

10.11.1 General

The magnitude of sediment transport is dependent upon the ability of the flowing water to transport incoming sediment and/or to erode the material making up the bed and/or banks of the

watercourse. Watercourses composed predominately of sand-sized material will respond to virtually the entire range of flows to which it is subjected. However, watercourses composed of significant quantities of coarser (gravel, cobble and boulder) material will be limited to adjustments only during large flow events.

A basic understanding of sediment transport mechanics is fundamental in qualitative and quantitative sediment transport analyses. Inherent in that understanding are the concepts of incipient motion and armoring. Incipient motion analysis provides a means to estimate the largest size of sediment particle that can be transported during a given flow event. In cases where there is a sufficient quantity of coarse sediment, an armor layer may form that can act as a complete or partial control to sediment transport. The application and limitation of the numerous sediment transport equations must be understood and appreciated when performing sediment transport analyses and quantitative studies.

Shallow flow over roadway initially causes headcutting into road subgrade and pavement

10.11.2 Bed Form

Sediment transport is highly dependent upon the resistance to flow, and resistance to flow in an alluvial channel is strongly related to the physical shape of the bed. The physical elements that comprise the shape of the bed are called bed form. For a more thorough discussion of bed form and its impact on flow resistance see Simons and Senturk (1992). Those bed forms in common occurrence in alluvial channels are briefly described:

Plane bed - A flat or nearly-flat and smooth surface of the bed.

Ripples - Small bed forms that are typically less than a foot long and less than 1 1/2 inches high. They occur in lower regime flow.

Bars - Large bed forms that have lengths of the same order as channel width and heights about the same as flow depth. There are several kinds of bars, such as point bars, alternate bars, tributary bars and middle bars.

Dunes - Bed forms that are larger than ripples and smaller than bars. Size is a function of the geometry of the watercourse. It indicates higher transport rates than ripples.

Antidunes - Bed forms in upper regime flow that are often in trains. They are often called standing waves. They exhibit surface waves that are in phase with the antidunes.

Chutes and Pools - Bed forms of large elongated chutes of high slope and high velocity flow separated by low velocity pools. These represent very high sediment transport rates.

Watercourse exhibiting potential for large bed load discharge

Bed form is often associated with regime of flow. (Note: This is a different concept than the regime of Section 10.6.2.). Plane bed, ripples and dunes are typically in lower flow regime where the Froude number is usually less than 0.4. The transition to washed-out dunes and a return to plane bed (with high bed load transport) represent the transition regime where the Froude number is typically between about 0.4 to 0.7. Antidunes with standing waves or with violent breaking waves and chute and pool are in upper flow regime where the Froude number is typically greater than 0.7 (Guy, 1970).

10.11.3 Incipient Motion

Incipient motion occurs when the hydrodynamic forces acting on a grain of sediment of given size is equal to the forces resisting movement. Incipient motion is often analyzed using the Shields relation:

$$d_c = \frac{\tau_o}{F_*(\gamma_s - \gamma)} \tag{10.6}$$

where: d_c is the sediment diameter at incipient motion in feet,

τ_o is the bed shear stress in pounds per square foot,

γ_s is the sediment specific weight, typically 165 pounds per cubic foot,

γ is the water specific weight, 62.4 pounds per cubic foot,

and F_* is the dimensionless shear stress, often referred to as the Shields parameter. F_* ranges from 0.03 to 0.06 and a value of 0.047 is often used (AMAFCA, 1994).

The bed shear stress in pounds per square foot, is calculated by

$$\tau_o = \gamma R S \tag{10.7}$$

where R is the hydraulic radius, in feet, and S is the channel friction 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 AMAFCA (1994), ASCE (1975), Richardson and others (2001), Simons and Senturk (1992), Yang (1973), ADWR (1985), Chang (1988), and Shen (1971, 1972 and 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 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.

10.11.4 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. If the watercourse is in a degradational mode, this process can continue over a range of discharge events, each larger event removing the increasing 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 and data on bed material size gradation within the anticipated depth of scour. In application, the d_{95} particle size is considered to be the maximum size for armor formation. Therefore, armoring (for a given discharge) can be expected when the computed incipient motion size is equal to or smaller than the d_{95} size of the bed material.

The depth of scour (Y_s) necessary to establish an armor layer can be estimated by Pemberton and Lara (1984).

$$Y_s = Y_a(\frac{1}{P_c} - 1) \tag{10.8}$$

where Y_a is the desired thickness of the armor layer (normally assumed to be 2 to 3 times the critical particle size, d_c, and P_c is the decimal fraction of bed material coarser than the armoring size.

10.11.5 Sediment Transport Methods

The planning and design of drainage and flood control facilities often requires the analysis of sediment transport. Often those analyses are performed using sediment transport methods. Those methods may be mathematical or graphical and can be theoretically or empirically based. Often the method is some combination of all of the above. Some of the more popular sediment transport methods are the Einstein bed load function, the Meyer-Peter, Muller equation, the Yang unit stream power concept and the Colby relations. However, there are virtually dozens of sediment transport relations in the literature. A problem for the engineer is to select one or more of these relations for use in solving a particular problem. When selecting a sediment transport method, the data base (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 method is best and two or more methods may need to be used and the respective results evaluated. The results by different methods often differ drastically. It is absolutely imperative that the application and limitation of the various methods be understood when using those to estimate sediment transport. The engineer must use experience and judgement in both the selection of the sediment transport method and in the interpretation of the results. See AMAFCA (1994), ASCE (1975), Yang (1973), ADWR (1985), Chang (1988), Richardson and others (2001), Shen (1971, 1972, 1973), Sheppard (1960), and Simons and Senturk (1992) for further discussions of sediment transport methods.

10.12 BANK EROSION AND LATERAL MIGRATION

10.12.1 Bank Erosion

Bank erosion and widening of watercourses occurs from two primary mechanisms; grain-by-grain erosion and bank failure. Commonly, grain-by-grain erosion and bank failure act together; fluvial erosion scours the toe of the bank, and failure follows. Removal of the failed bank material occurs through fluvial erosion and the process is repeated.

The bank erosion process can result from watercourse incision (degradation), flow around bends, flow deflection due to local deposition or obstructions, aggradation, or a combination of the above. For the case of an incising watercourse, exceedence of the maximum stable bank height will lead to mass failure and bankline retreat. Flow around a bend can cause erosion at the toe of the bank and subsequent bank failure due to increased shear stress on the outside of the

bend. Both local deposition and aggradation over a longer reach create mid-channel bars that can deflect flow into the bank with essentially the same result as flow on the outside of a bend.

The specific failure mechanisms at a given location are related to the characteristics of the bank material. In general, bank material can be broadly classified as cohesive, noncohesive, and composite.

Noncohesive bank material tends to be removed grain by grain from the bank. The rate of particle removal and the rate of bank erosion, is affected by factors such as particle size, bank slope, the direction and magnitude of the velocity adjacent to the bank, turbulent velocity fluctuations, the magnitude and fluctuations in the shear stress exerted on the banks, seepage forces, piping, and wave forces.

Erosion of bank material at toe of slope caused bank failure.

Cohesive material is more resistant to surface erosion and has low permeability, which may reduce the effects of seepage and subsurface flow on the stability of the banks. However, the lower permeability may increase pore pressure due to saturation resulting in bank collapse when water levels drop in the watercourse. When undercut and/or saturated, such banks are more likely to fail due to mass wasting processes.

Composite or stratified banks consist of layers of materials of varying size, permeability, and cohesion. The layers of noncohesive material are subject to erosion, but may be partly protected by adjacent layers of cohesive material. This type of bank is vulnerable to erosion and sliding as a consequence of subsurface flows and interlayer piping.

Grain-by-grain erosion can be a significant process in areas of concentrated flow and high shear stress, that is, on the outside of bends. However, studies of bank erosion processes indicate that mass failure and subsequent fluvial transport of the failed material is the primary mechanism by which the lateral adjustments occur.

10.12.2 Lateral Migration

Estimating the rate and magnitude of lateral migration depends on the sediment balance in the reach being analyzed. If analysis indicates that the reach is degradational (that is, bed material transport capacity exceeds supply) either on an average annual basis or in response to a flood, lateral migration will be primarily the result of failure due to undercutting at the toe of the bank.

When the bed material transport capacity and the supply are approximately equal, no net erosion or deposition of sediment is expected within the reach. For this case, lateral migration is the result of localized fluvial entrainment of the bank material as the watercourse seeks equilibrium. The preferred method for estimating the rate and magnitude of migration involves the use of historical data and empirically based migration rates. When historic data are not available or when conditions are significantly different than empirically based migration rates, lateral migration can be estimated using geomorphic techniques. The volume of material fluvially entrained from a failed bank (due to undercutting of the toe) within a given bend is approximately proportional to the transport capacity of the watercourse based on the sharpness of the bend.

For degradation reaches, bank failure occurs due to a combination of undercutting of the toe and/or exceedence of the stable bank height. The process leading to lateral migration of the bank is, therefore, related to changes in bank height and volume of material moved to create a given lateral migration distance.

When the reach is strongly aggradational and the overbank area is relatively flat, the channel alignment and extent of flooding is essentially a random phenomenon (that is, alluvial fan flooding) and prediction of the erosion boundaries by analytical means is limited. For this case, however, the flood limits are often wider than the erosion limits.

For more information on channel stability, lateral migration and methods to estimate limits erosion and lateral migration see Hedman and Osterkamp (1982), Lagasse and Schall (1988), Leopold and Maddock (1953), Leopold and Wolman (1957), Leopold and others (1964), MacB-

room (1981), Rosgen (1996), Schumm (1961, 1971 and 1977), Hjalmarson (1998), and Thorn (1998).

10.13 ESTIMATION OF SCOUR

10.13.1 General

Definition of Scour

Scour, for the intent of this discussion, is the lowering of the bed elevation of a watercourse, either locally or over some defined reach length of 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 scour at a specific location in a watercourse.

Purpose of Scour Estimates

Scour estimates are often needed for the following drainage and flood related purposes:

a. 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.

b. 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.

c. Estimation of depth of toe-down for structural bank lining.

d. Estimation of depth of scour immediately at or downstream of hydraulic structures.

e. Estimation of potential scour depth for buried utility crossings of watercourses.

Applications and Limitations

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. Therefore, the following are general guidelines for estimating scour along with currently used references that are considered applicable in Maricopa County.

10.13.2 Total Scour

Total scour, for a given application, should consider the following components of scour:

a. Long-term degradation of the bed of the watercourse.

b. General scour through a specific reach of the watercourse.

c. Local scour.

d. Scour induced due to a bend in the watercourse.

e. Scour associated with bedform movement through the watercourse.

f. Scour due to low-flow incisement.

Total scour (Z_t) is the sum of each of these individual components (Z_i) of scour. Total scour can be expressed as:

$$Z_t = FS\,(Z_{long\text{-}term} + Z_{general} + Z_{local} + Z_{bend} + Z_{bedform} + Z_{low\text{-}flow}) \quad (10.9)$$

A multiplying factor (FS) is used depending upon the purposes of the total scour estimation. For example, an FS equal to 1.0 may be appropriate when estimating total scour due to altered conditions in a watershed. However, in that case it would be advisable to estimate maximum and minimums of each individual component of scour and to estimate the range of total scour that can be expected. An FS of 1.3 is often used for the design of toe-down for bank protection. The use of higher FS, such as 1.5, may be justified where underestimation of scour would cause catastrophic failure that may result in loss of life or unacceptable economic consequences.

The following is a discussion of each component of scour that should normally be considered when estimating total scour.

Long-Term Degradation

Long-term degradation can be estimated by the following methods:

a. A trend analysis of historic bed elevation data.

b. Simulation by use of sediment transport modeling such as HEC-6 (USACE, 1991).

c. Application of equilibrium slope analyses.

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.

Simulation modeling may provide useful results; however, that method is dependent upon appropriate hydraulic data for the watercourse (hydraulic geometry and sediment characteristics). Furthermore, the results are 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.

Equilibrium slope is a method that can often be applied to estimate long-term degradation without extensive data or modeling effort. The application of this method does require 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, a reach of armored channel bed, or a constructed facility such as a diversion dam, roadway crossing, and so forth.

Long-term degradation using equilibrium slope analysis is estimated by

$$Z_{long\text{-}term} = L_w \, \Delta S \tag{10.10}$$

where $Z_{long\text{-}term}$ is the bed elevation change, in feet, at a distance, L_w, in feet, upstream of the pivot point and ΔS is the decrease in bed slope, in ft/ft from the existing slope. Equilibrium slope analysis resulting in an increase in bed slope upstream from the pivot point would indicate an aggradational zone rather than long-term degradation.

Application of Equation 10.10 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). Grade control structures are to be constructed within the channelized reach at a distance of 2,000 feet between structures. The long-term degradation at the toe of each drop structure is estimated by:
>
> $$Z_{long\text{-}term} = (2000\,ft)(0.0042\text{-}0.0028\,ft/ft)$$
>
> $$= 2.8 \text{ feet}$$

Several methods are recommended by Pemberton and Lara (1984) for performing equilibrium slope analyses; the Schoklitsch bedload equation (Shulits, 1935), the Meyer-Peter, and Muller (1948) bedload equation, the Shields (1936) diagram, and Lane's (1952) relation for critical tractive force. The limitations and assumptions of each method should be carefully evaluated when making the selection of a preferred method. Often, more than one method can be used and the

results compared. Corroborating results by two or more methods would increase reliance on those results. However, there often is considerable deviation in results by the various methods. In which case, independent data, regional experience and/or engineering judgement must be used in selecting the equilibrium slope.

General Scour

General scour is that component of total scour that would occur during the passage of a design flood. 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. For major watercourses, general scour would often be estimated by a sediment transport model study, such as the use of HEC-6 (USACE, 1991). General scour in minor watercourses can be estimated by the following equation (Zeller, 1981):

$$Z_{general} = Y_{max}\left[\frac{0.0685 v^{0.8}}{Y_h^{0.4} S_e^{0.3}} - 1\right] \tag{10.11}$$

where: $Z_{general}$ is the general scour depth, in feet,

Y_{max} is maximum depth of flow, in feet,

Y_h is the hydraulic depth, in feet,

v is the average velocity of flow, in ft/sec, and

S_e is the energy slope (or bed slope if uniform flow is assumed), in ft/ft.

The reference by Zeller (1981) should be consulted prior to applying this equation. If Equation 10.11 yields negative results, a value of zero is to be used for general scour.

Local Scour

Local scour is that component of total scour that is caused by flow irregularities. If the transport rate of sediment away from the local region is greater than the transport rate into the region, a scour hole develops. As the depth of scour is increased, the strength of the vortex or vortices is reduced, the transport rate is reduced and equilibrium is reestablished and scouring ceases.

Flow irregularities can occur in natural watercourses due to bends or restrictions along the banks. Flow irregularities also occur due to constructed facilities such as bank lining, bank protection works (such as groins), hydraulic structures across the watercourse (such as diversion dams or grade control structures), and structures in the watercourse (such as bridges or culverts). Bridge scour, including the local component of bridge scour, is discussed in Section 10.13.4.

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Local scour at unprotected culvert outlet.

Generally, local scour depths are much larger than long-term degradation or general scour. But, 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.

Five methods for estimating local scour due to natural restrictions and bends, or bank lining are presented by Pemberton and Lara (1984). The USBR Method I is for wide, sand bed watercourses with d_{50} ranging from 0.5 to 0.7 mm, and slopes from 0.004 to 0.008 ft/ft. That method probably has limited application in Maricopa County. USBR Method II is recommended for subcritical flow and includes consideration of watercourse curvature. The Lacey (1930) equation is for subcritical flow, includes consideration of watercourse curvature, and requires the use of bed material size and a “silt factor.” The Blench (1969) equation is a function of unit discharge (q in cfs/foot width), Blench’s “zero bed factor” and a factor for watercourse curvature. The Neill (1973) equation is based on flow depth and velocity and the estimation of “competent velocity.” These equations can be used to estimate the local scour due to bank lining or similar applications. The report by Pemberton and Lara (1984) or the individual references should be consulted prior to application of any method. Often, more than one method can be applied and the results compared. Engineering judgment and experience are needed when selecting the value for local scour.

Local scour downstream of a hydraulic structure can be estimated by empirical equations. Flow over the structure can be either submerged or free falling depending on tailwater conditions. For free falling conditions, three local scour equations are available. The Schoklitsch (1932) equation requires hydraulic parameters including the effective drop height and the bed material particle size. The Veronese (1937) equation requires hydraulic parameters including effective drop height, but is independent of bed material grain size and may overestimate local scour for some watercourses in Maricopa County. The Zimmerman and Maniak (1967) equation is a function of

the d_{85} bed material particle size, but is independent of many hydraulic parameters and does not consider the drop height. Therefore, that equation should only be used for relatively low (possibly not greater than half of the approach flow depth) drop heights. The Pemberton and Lara (1984) or the original references should be consulted when selecting or applying any of these equations.

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Culvert causes backwater resulting in upstream aggradation.

For a submerged structure, the local scour depth can be estimated by the Simons, Li & Associates (1986) equation. The equation is a function of drop height and other hydraulic parameters, but is independent of bed material grain size. It may overestimate scour depth for coarse bed material watercourses. That reference should be consulted when using that equation.

Bend Scour

Bend scour may need to be estimated if not included as a component of local scour (see above). For sand-bed watercourses, Zeller (1981) presents a bend scour equation. That reference should be consulted in its use and application.

Bedform Trough Depth

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.

$$Z_{bedform} = 0.5 d_h \qquad (10.12)$$

Bedform trough depth should be estimated for dunes that occur during lower regime flow, and antidunes that occur during upper regime flow. Simons and Senturk (1992) provide dune height equations. Dune height is estimated by:

$$d_h = 0.066 Y_h^{1.21} \tag{10.13}$$

where: d_h is dune height, in feet, and

y is hydraulic depth of flow, in feet.

Antidune height is estimated by:

$$d_h = 0.28 \pi Y_h F_r^2 \tag{10.14}$$

where: d_h = antidune height, in feet,

Y_h is hydraulic depth of flow, in feet, and

F_r is Froude Number.

Dunes form during lower regime flow, typically at F_r less than about 0.7, and antidunes form during the upper regime flow and may form during the transition from lower to upper regime flows. Therefore, antidunes can be expected for F_r greater than about 0.7. Antidune height will usually be greater than dune height. In the transition region, about 0.7 to 1.0 F_r, the larger of either dune or antidune height should be used.

Low-Flow Incisement

The normal irregularities in the bed of a watercourse (both natural and man-made) result in a low-flow channel. That channel is formed by the predominance of a low-flow condition or due too low-flows that persist after a flood. The magnitude of low-flow incisement may best be estimated by 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.

10.13.3 Limits to Scour from Armoring

Armoring is the process in an alluvial watercourse wherein 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 the inflow of bed material load in excess of transport capacity or those watercourses for which the bed material does 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.

Armoring can be a limiting agent to scour, and, in fact, the placement of riprap as a watercourse liner or around hydraulic structures is an "engineered" armoring. Therefore, when considering scour, particularly long-term and general scour, the potential for armoring should be considered.

The above photo indicates failed riprap from road overtopping and high exit velocities from culverts. Note head cutting.

Several methods are available for evaluating the potential for armoring. The incipient motion method (see Section 10.11.2) is commonly used and easily applied. Other methods include use of the Meyer-Peter, Muller equation (Sheppard, 1960), the competent bottom velocity method (Mavis and Laushey, 1948), Lane's tractive force method (Lane, 1952), and Yang's incipient motion relation (Yang, 1973). The user should consult those references when making application of those methods to evaluate armor potential.

10.13.4 Bridge Scour

Total Scour at Bridges

Scour at bridges must consider all reasonable components of scour that can apply to detrimentally impact a bridge pier or abutment. The total scour (Z_t) at a bridge is typically expressed as:

$$Z_t = FS\,(Z_{long\text{-}term} + Z_{local} + Z_{contraction}) \qquad (10.15)$$

where FS is a factor of safety which is set at 1.0 for most conditions, but under certain conditions of hazard, including potential economic loss or uncertainty in analyses, could be set higher than 1.0.

The component of long-term scour ($Z_{long\text{-}term}$) can be estimated by procedures discussed in Section 10.13.2. The potential for armoring (Section 10.13.3) may be considered, but should be used cautiously to limit scour depth.

The procedure in *Evaluating Scour at Bridges*, HEC-18 (USDOT, 2001b) should be consulted when estimating scour at bridges. Usually the largest component of scour is from local scour at the pier or abutment. Certain scour equations include the angle of attack of the flow, and therefore, bend scour is not normally added because it can be accounted for in the local scour.

Contraction scour occurs when the flow area of the watercourse is reduced because of natural conditions or because of the bridge approaches encroaching into the watercourse. Two equations are provided in HEC-18 (USDOT, 2001b) for contraction scour. One is for live bed conditions; that is, when there is bed material transport from upstream of the bridge. For that condition, a modified version of Laursen's live-bed contraction scour equation (Laursen, 1960) is used. The second is for clear water conditions; that is, when there is little or no sediment transport from upstream of the bridge. For that condition, Laursen's clear-water contraction scour equation (Laursen, 1963) is used. The HEC-18 publication (USDOT, 2001b) should be consulted when estimating contraction scour.

Pier Scour

The commonly used pier scour equations are the Colorado State University equation (Richardson and others, 2001) and Froehlich (1988). Both of those equations are considered in the HEC-RAS program for bridge pier scour (USACE 2001 and 2001b); however, only the Colorado State University equation is recommended in HEC-18 (USDOT, 2001b). The Froehlich equation has been shown to compare well with observed data. Those references should be consulted when estimating pier scour.

Abutment Scour

The commonly used abutment scour equations are the HIRE equation (Richardson and others, 2001) and Froehlich (1989). Those equations are provided both in HEC-18 (USDOT, 2001b) and the HEC-RAS program (USACE 2001a and 2001b). Those references should be consulted when estimating abutment scour.

Watercourse Stability at Highways

The stability of the watercourse at and near highway structures should be considered if channel instability is suspected. Procedures to investigate watercourse stability are provided in HEC-20 (USDOT 2001c).

Bridge Scour Countermeasures

Procedures to provide bridge scour countermeasures are provided in USDOT (2001a).

10.14 APPROACH TO SEDIMENTATION ANALYSIS

10.14.1 Issues For Consideration

Table 10.4 presents a checklist of potential problems relating to channel movement/scour and the causative factors that should be examined.

TABLE 10.4
CHECKLIST OF POTENTIAL PROBLEMS RELATING TO CHANNEL MOVEMENT AND SCOUR

Potential Problem	Yes	No
Long term degradation or aggradation		
Reservoirs		
Mining		
Urbanization		
Watershed changes		
General scour		
Downstream variable water surface relationship		
Contraction and expansion		
Bed configuration and movement		
Live-bed scour		
Clear-water scour		
Bends		
Natural stream constriction		
Floodplain encroachment		
Berms from sediment deposits		
Island or bar formations		
Debris		
Growth of vegetation in floodplain or channel		
Bed and sediment characteristics		
Armoring		
Lateral migration		

10.14.2 Three Tier Approach for Sediment Transport Analysis

The following table provides a three tiered approach to undertaking sediment transport analysis. The degree of effort increases from top to bottom and it is assumed that the tasks of a preceding tier are undertaken prior to starting the more comprehensive tasks.

The first tier represents a qualitative approach. This is the least sophisticated which requires the lowest effort. The tasks listed herein provide background information for more detailed analysis. Many of these tasks would be undertaken as part of a basic drainage study. The bed and bank material analysis consists of identifying the material present for the project area and the areas immediately upstream and downstream of the site. Planform characteristics relate to the geometry of the watercourse bed, such as presence of dunes or anti-dunes. Information about land use changes provides insight as to the past and present supply of sediment and changes in runoff characteristics due to increased imperviousness and changes in hydraulic travel time. Review of past and present aerial photographs provides information as to the extent of streambed migration and the stability of the fluvial system. Coupled with information about flood history, this provides the designer with helpful information about watershed responses to flood events. Hydrologic analysis of the on-site and off-site contributing drainage areas provides the designer with the magnitude and frequency of runoff expected. Finally, as explained previously in this chapter, the information gathered from above is used with rudimentary geomorphologic relationships to provide insight as to the expectation of sedimentation or scour.

The second tier identifies quantitative, end result methods of analysis consistent with the methods presented in this chapter. The final level of analysis is dynamic modeling of sediment transport that requires extensive knowledge of the sediment transport process. That level is well beyond the scope of this chapter. Furthermore, this list is not all-inclusive. Additional methods may be necessary depending upon the specifics of the project under study. Interested readers should review the references cited at the end of this chapter for further guidance.

TABLE 10.5
TASK LIST FOR SEDIMENT TRANSPORT INVESTIGATIONS

Sediment Transport Analysis (Qualitative)
Determination of Planform Characteristics
Lane Relation and other Geomorphic Relationships
Aerial Photograph Interpretations
Bed and Bank Material Analysis (visual inspection)
Land Use Changes
Flood History
Rainfall/Runoff Relationships
Sediment Transport Analysis (Quantitative)
Watershed Sediment Yield
Detailed Bed and Bank Material Analysis
Profile Analysis
Incipient Motion Analysis
Armoring Potential
Sediment Transport Capacity
Equilibrium Slope Analysis
Sediment Continuity Analysis
Quantification of Vertical and Horizontal Channel Response
Bend Scour
Low Flow Channel Incisement
Gravel Mining Impacts
Contraction Scour
Local Abutment Scour
Local Pier Scour
Cumulative Channel Adjustment
Lateral Migration[a]
Sediment Transport Analysis (Sediment Routing in Time & Space)
Data Inventory Modeling
Watershed Sediment Modeling
Instream Mining Response
Single Event Stream Bed Modeling
Long Term Bed Modeling

a. See - *State Standard for Watercourse System Sediment Balance, Guideline 1, Lateral Migration Setback Allowance for Riverine Floodplains in Arizona,* SSA 5-96, Arizona Department of Water Resources, September 1996.

10.15 REFERENCES

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