ARIZONA DEPARTMENT OF TRANSPORTATION

HIGHWAY DRAINAGE DESIGN MANUAL
HYDRAULICS

Final Report

January, 2007

Arizona Department of Transportation
206 South 17th Avenue
Phoenix, Arizona 85007

REVISIONS, AMENDMENTS AND CORRECTIONS TO THIS MANUAL ARE IN PROGRESS; ADOT REQUESTS THAT FINDINGS AND SUGGESTIONS BE SENT TO THE DRAINAGE SECTION MANAGER, KEN AKOH-ARREY AT kakoh-arrey@azdot.gov.

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CHAPTER 1

INTRODUCTION

1.1 Introduction

1.1.1 General

This manual presents approaches, methods, and procedures for the design of drainage structures on ADOT highway projects. Although many procedures are systematically presented, they are only a tool. The designer of the facility remains responsible for analyzing the situation and selecting the approach to solve the problem including the choice of the correct tool.

Computer programs take out the intensive effort for the performance of many computations. However, they are no replacement for the judgments necessary. The user of computer programs must understand how the program performs the calculations and what assumptions are made, i.e., how appropriate is the use of the computer program to the problem being addressed. In this manual several procedures are presented that are more efficiently performed by the use of computer programs. In some areas of the manual, examples are given that were done using a computer program. The principles necessary for solving the necessary formulas and equations are presented in the manual. The manual does NOT present directions on how to run the computer program, the designer must refer to the appropriate user manual of those programs.

1.1.2 Background

Much of this manual uses the AASHTO Model Drainage Manual as the lead information document with additional information from the Flood Control District of Maricopa County Hydraulics Manual and the Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona. The American Association of State Highway and Transportation Officials (AASHTO) Task Force on Hydrology and Hydraulics produced the Model Drainage Manual as part of their continuing work to assist the Standing Committee on Highways, Subcommittee on Design, in developing guidelines and in formulating policy. The Task Force which was established in 1970 has also produced a series of guides which are published as the AASHTO Highway Drainage Guidelines (HDG). The drainage guidelines provide an overview of the subject area and references to appropriate design procedures.

The manual has been developed to give the designer a basic working knowledge of hydraulics complete with example problems. All basic design elements are included such that the designer can design highway drainage with minimal assistance. However, this manual cannot provide guidance on complex hydrologic or hydraulic problems and is no substitute for experience or engineering judgment. References to specific computer programs, AASHTO guidelines, manuals and regulations will be noted within the manual. It is expected that the designer will be knowledgeable in the use of the referenced items.

1.1 Introduction (continued)

1.1.3 References

References relating to the chapter content are identified at the end of each chapter. In addition to the specific subject matter references, the designer should have the following reference material available:

ADOT Hydrology Manual
ADOT Roadway Design Guidelines

A good hydraulics text, such as
Chow, V.T. Open Channel Hydraulics, McGraw-Hill. 1970.
French, Richard H., “Open Channel Flow”, McGraw-Hill, 1985
Henderson, F.M. Open Channel Flow, Macmillan. 1966

1.2 User Instruction

1.2.1 Instructions

This manual is for the design of drainage structures on ADOT projects. The methods described herein shall be used where appropriate for the design of drainage structures that are for ADOT projects or for drainage structures that will be within ADOT right-of-way. Where joint projects are to be developed through IGA’s or JPA’s there should be early discussions on the methods to be followed.

1.3 Questions and Comments

1.3.1 Content Questions

Any comments or questions should be addressed to the ADOT Roadway Engineering Group, Drainage Section. Questions about a particular computer program or application must be referred to the originating Agency or Company.

1.3.2 Errors, Additions And Updates

If errors are discovered in the ADOT Hydraulics Manual, they should be reported to the ADOT Drainage Section so that corrections can be made.

CHAPTER 2

LEGAL ASPECTS

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Chapter 2 Legal Aspects
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2.1 Overview

2.1.1 Introduction

Various drainage laws and rules applicable to highway facilities are discussed in this chapter. The intention is only to provide information and guidance on the engineer's role in the legal aspects of highway drainage. This chapter should not in any way be treated as a manual upon which to base legal advice or make legal decisions. It is also not a summary of all existing drainage laws, and most emphatically, this chapter is not intended as a substitute for legal counsel.

The following generalizations can be made in reaching the proper conclusion regarding liability:

- A goal in highway drainage design should be to perpetuate natural drainage, insofar as practicable.
- The courts look with disfavor upon infliction of injury or damage that could reasonably have been avoided by a prudent designer, even where some alteration in flow is legally permissible.
- The laws relating to the liability of government entities are undergoing radical change, with a trend toward increased government liability.

2.1.2 Order Of Law Supremacy

The descending order to law supremacy is Federal, State, and local, and, except as provided for in the statutes or constitution of the higher level of government, the superior level is not bound by laws, rules, or regulations of a lower level. State permit requirements are an example of law supremacy. Federal agencies do not secure permits issued by State agencies, except as required by Federal law. Many laws of one level of government are passed for the purpose of enabling that level to comply with or implement provisions of laws of the next higher level. In some instances, however, a lower level of government may promulgate a law, rule or regulation which would require an unreasonable or even illegal action by a higher level. An example is a local ordinance which would require an expenditure of State funds for a purpose not intended in the appropriation. Many such conflicts in the laws of different levels of government involve constitutional interpretation and must be determined case by case.

2.1.3 ADOT Responsibility

ADOT is generally treated like a private party in drainage matters. The undertaking of a public improvement is liable like an individual for damage resulting from negligence or an omission of duty. As a general rule, ADOT is under no legal duty to construct drainage improvements unless highway improvements necessitate drainage — as in those situations in which street grading and paving or construction increase or alter storm runoff.

ADOT can be held liable for negligent construction of drainage improvements, for negligent maintenance and repair of drainage improvements and if it fails to provide a proper outlet for drainage improvements. Liability will attach where ADOT:

- collects surface water and casts it in a body onto private property where it did not formerly flow;
- diverts, by means of artificial drains, surface water from the course it would otherwise have taken, and casts it in a body large enough to do substantial injury on private land, where, but for the artificial storm drain, it would not go; and
- fills up, dams back, or otherwise diverts a stream of running water so that it overflows its banks and flows on the land of another.

2.1 Overview (continued)

2.1.3 ADOT Responsibility (continued)

Design Guidance

Drainage matters range from the simple to the complicated. If the facts are ascertained and a plan developed before initiating a proposed improvement, the likelihood of an injury to a landowner is remote and ADOT should be able to undertake such improvements relatively assured of no legal complications.

If the designer needs guidance on a particular drainage problem or improvement, the request for guidance should describe as a minimum:

- the watercourse under study,
- what are the problems involved, and what causes them (obstructions, topography, development - present and future),
- the proposed improvements that will make the situation better,
- how the proposal requires that the natural drainage be modified,
- what is the potential liability for doing something versus doing nothing,
- who will benefit from the proposed improvements, and
- in general, how what is proposed is "reasonable."

2.2 Federal Laws

2.2.1 Introduction

Federal law does not deal with drainage per se, but many laws have implications which affect drainage design. These include laws concerning:

- flood insurance and construction in flood hazard areas,
- navigation and construction in navigable waters,
- water pollution control,
- environmental protection,
- protection of fish and wildlife, and
- coastal zone management.

Federal agencies formulate and promulgate rules and regulations to implement these laws, and highway hydraulic engineers should attempt to keep informed regarding proposed and final regulations.

2.2.2 Constitutional Power

Federal law consists of the Constitution of the United States, Acts of Congress, regulations which government agencies issue to implement these acts, Executive Orders issued by the President, and case law. The Congress of the United States is granted constitutional power to regulate "commerce among the several states." A part of that power is the right to legislate on matters concerning the instrumentalities of interstate commerce such as navigable waters. The definition of navigable waters expands and contracts depending upon the breadth required to adequately carry out the Federal purpose. The result is that Congress can properly assert regulatory authority over at least some aspects of waterways that are not in themselves subject to navigation.

2.2 Federal Laws (continued)

2.2.3 Executive Orders

Presidential Executive Orders (E.O.) have the effect of law in the administration of programs by Federal agencies. While executive orders do not directly apply to State highway departments, these requirements are usually implemented through general regulations. Two Executive Orders that directly affect drainage are:

Executive Order 11990, Protection of Wetlands, DOT Order 5660.1A, 23 CFR 777.
Purpose - to avoid direct or indirect support of new construction in wetlands wherever there is a practicable alternative.

Executive Order 11988, Floodplain Management, as amended by Executive Order 12148, DOT Order 5650.2, FHPM 6-7-3-2, 23 CFR 650, Subpart A, 771.
Purpose - to avoid the long- and short-term adverse impacts associated with the occupancy and modification of floodplains, and to restore and preserve the natural and beneficial values served by floodplains.

2.3 Federal Agencies

2.3.1 General

Basically four Federal agencies have the lead to carry out existing Federal regulations, other Federal agencies may have review and comment responsibilities regarding impacts of projects to the environment. When the designer becomes involved in obtaining approvals from the Federal agencies, he should be aware that these agencies do not always work in concert. Quite often they will not be in agreement with each other. This can result in significant project delays unless early coordination is initiated and diligently pursued. These conflicts between Federal agencies occur as a result of their having different rules; some are "regulators" while others are "resource" motivated. For this reason they will have different goals and, in some instances, definitions of such things as wetlands. When conflicts occur, it is best to quickly determine which agency has primary responsibility and attempt to address their needs.

2.3.2 Federal Highway Administration

Some of the more significant Federal laws affecting the Federal Highway Administration are listed below with a brief description of their subject area.

- *Department of Transportation Act (80 Stat. 941, 49 U.S.C. 1651 et seq.).* This Act established the Department of Transportation and set forth its powers, duties and responsibilities to establish, coordinate and maintain an effective administration of the transportation programs of the Federal Government.
- *Federal-Aid Highway Acts (23 U.S.C. 101 et seq.).* The Federal-Aid Highway Acts provide for the administration of the Federal-Aid Highway Program. Proposed Federal-Aid projects must be adequate to meet the existing and probable future traffic needs and conditions in a manner conducive to safety, durability and economy of maintenance, and must be designed and constructed according to standards best suited to accomplish these objectives and to conform to the needs of each locality.

2.3 Federal Agencies (continued)

2.3.2 Federal Highway Administration (continued)

- *Federal-Aid Highway Act of 1970 (84 Sta. 1717, 23, U.S.C. 109 (h)).* This act provided for the establishment of general guidelines to insure that possible adverse economic, social and environmental effects relating to any proposed Federal-aid project have been fully considered in developing the project. In compliance with the Act, the Federal Highway Administration issued process guidelines for the development of environmental action plans. These guidelines are contained in the Federal-Aid Highway Program Manual Volume 7, Chapter 7, Section 1 (FHPM 7-7-1), and in 23 CFR 795 et seq.
- *Federal-Aid Highway Act of 1966 (80 Stat. 766), amended by the Act of 1970 (84 Stat. 1713), 23 U.S.C. 109 (g).* This act required the issuance of guidelines for minimizing possible soil erosion from highway construction. In compliance with these requirements, the Federal Highway Administration issued guidelines which are applicable to all Federal-Aid highway projects. These guidelines are included in FHPM 6-7-1-1, 6-7-3-1, 6-7-3-2. Regulatory material is found in 23 CFR 650.201.

The Federal Highway Administration (FHWA) has been authorized to implement certain functions in the application of Federal regulations.

- <u>Section 404 of the Clean Water Act</u> - The Federal Highway Administration has the authority to implement the Section 404 Permit Program (Clean Water Act of 1977) for Federal-Aid highway projects processed under 23 CFR 771.115 (b) categorical exclusions. This authority was delegated to the Federal Highway Administration by the Corps of Engineers to reduce unnecessary Federal regulatory controls over activities adequately regulated by another agency. This permit is granted for projects where the activity, work or discharge is categorically excluded from environmental documentation because such activity does not have individual or cumulative significant effect on the human environment.

2.3.3 US Coast Guard

The Coast Guard (USCG) has regulatory authority under Section 9 of the Rivers and Harbors Act of 1899, 33 U.S.C. 401 (delegated through the Secretary of Transportation in accordance with 49 U.S.C. 1655 (g)) to approve plans and issue permits for bridges and causeways across navigable rivers. As outlined in 23 CFR 650 the area of jurisdiction of the USCG and FHWA is established as follows:

The FHWA has the responsibility under 23 U.S.C. 144(h) to determine that a USCG permit is not required. This determination shall be made at an early stage of project development so that any necessary coordination can be accomplished during environmental processing.

2.3 Federal Agencies (continued)

2.3.3 US Coast Guard (continued)

The USCG has the responsibility:
(1) to determine whether or not a USCG permit is required for the improvement or construction of a bridge over navigable waters except for the exemption exercised by FHWA as stated above, and

(2) to approve the bridge location, alignment and appropriate navigational clearances in all bridge permit applications.

For more information related to navigational clearances for bridges see the Federal-Aid Highway Program Manual 6-7-1-1.

2.3.4 US Corps of Engineers

The Corps of Engineers has regulatory authority over the construction of dams, dikes or other obstructions (which are not bridges and causeways) under Section 9 (33 U.S.C. 401). The Corps also has authority to regulate Section 10 of the River and Harbor Act of 1899 (33 U.S.C. 403) that prohibits the alteration or obstruction of any navigable waterway with the excavation or deposition of fill material in such waterway. Section 11 of the River and Harbor Act of 1899 (33 U.S.C. 404) authorizes the Secretary of the Army to establish harbor lines. Work channelward of those lines requires separate approval of the Secretary of the Army and work shoreward requires Section 10 permits.

<u>Section 404 of the Clean Water Act (33 U.S.C. 1344) prohibits the unauthorized discharge of dredged or fill material into waters of the United States, including navigable waters.</u> Such discharges require a permit. The term "discharges of fill material" means the addition of rock, sand, dirt, concrete or other material into the waters of the United States incidental to construction of any structure. The Corps of Engineers has granted Nationwide General Permit for twenty-six categories of certain minor activities involving discharge of fill material. Under the provisions of 33 CFR 330.5(a)(15), fill associated with construction of bridges across navigable waters of the United States, including cofferdams, abutments, foundation seals, piers, temporary construction and access fills are authorized under the Nationwide Section 404 Permit providing such fill has been permitted by the U. S. Coast Guard under Section 9 of the River and Harbor Act of 1899 as part of the bridge permit. Therefore, formal application of the Corps of Engineers for a Section 404 Permit is not required unless bridge approach embankment is located in a wetland area contiguous to said navigable stream. The Corps of Engineers has Section 404 regulatory authority over streams the Coast Guard has placed in the "advance approval" category. This category of navigable streams is defined as navigable in law but not actually navigated other than by logs, log rafts, rowboats, canoes and motorboats. Notably this regulation does not apply to the actual excavation or "dredging of material," provided this material is not reintroduced into any regulated waterway including the one from which it was removed.

The 1992 Energy and Water Development Appropriation Act provides guidance to use the 1987 Manual of the U.S. Army Corps of Engineers in the delineation of wetlands. This allows more flexibility in the definition and determination of wetlands.

2.3 Federal Agencies (continued)

2.3.4 US Corps of Engineers (continued)

Section 404 of the Clean Water Act (33 U.S.C. 1344) requires any applicant for a Federal permit for any activity that may affect the quality of waters of the United States to obtain water quality certification from the State certifying agency. In Arizona this is the Department of Environmental Quality (ADEQ).

2.3.5 US Environmental Protection Agency

The EPA is authorized to prohibit the use of any area as a disposal site when it is determined that the discharge of materials at the site will have an unacceptable adverse effect on municipal water supplies, shell-fish beds and fishery areas, wildlife, or recreational areas (Section 404 (c)), Clean Water Act (33 U.S.C. 1344).

EPA is authorized under the Section 402 of the Clean Water Act (33 U.S.C. 1344) to administer and issue a "National Pollutant Elimination Discharge System" (NPDES) permit for point source discharges, provided prescribed conditions are met.

- EPA or a state under the delegated authority issues NPDES individual or a general permit for storm water discharge associated with industrial activities involving any disturbance of five acres (approximately 2 ha) or more. Highway construction activities are classified as industrial activities.

2.3.6 US Fish and Wildlife Service

The Fish and Wildlife Act of 1956 (16 U.S.C. 742 et seq.), the Migratory Game-Fish Act (16 U.S.C. 760c-760g) and the Fish and Wildlife Coordination Act (16 U.S.C. 611-666c) express the concern of Congress with the quality of the aquatic environment as it affects the conservation, improvement and enjoyment of fish and wildlife resources. The Fish and Wildlife Coordination Act requires that "whenever the waters of any stream or body of water are proposed or authorized to be impounded, diverted, the channel deepened, or the stream or other body of water otherwise controlled or modified for any purpose whatever, including navigation and drainage, by any department or agency of the United States, or by any public or private agency under Federal permit or license, such department or agency shall first consult with the United States Fish and Wildlife Service, Department of the Interior, and with the head of the agency exercising administration over the wildlife resources of the particular state with a view to the conservation of wildlife resources by preventing loss of and damage to such resources as well as providing for the development and improvement thereof." The Fish and Wildlife Service's role in the permit review process is to review and comment on the effects of a proposal on fish and wildlife resources. It is the function of the regulatory agency (e.g., Corps of Engineers, U.S. Coast Guard) to consider and balance all factors, including anticipated benefits and costs in accordance with NEPA, in deciding whether to issue the permit (40 FR 55810, December 1, 1975).

2.4 Federal Emergency Management Agency (FEMA)

2.4.1 National Flood Insurance Program (NFIP)

The Flood Disaster Protection Act of 1973 (Pl 93-234, 87 Stat. 975) denies Federal financial assistance to flood prone communities that fail to qualify for flood insurance. The Act does require communities to adopt certain land use controls in order to qualify for flood insurance. These land use requirements could impose restrictions on the construction of highways in floodplains and floodways in communities that have qualified for flood insurance. A floodway, as used here and as used in connection with the National Flood Insurance Program, is that portion of the floodplain required to pass a flood that has a 1-percent chance of occurring in any 1-year period without cumulatively increasing the water surface elevation more than 1 ft.

2.4.2 NFIP Impact on Highways

The National Flood Insurance Act of 1968, as amended, (42 U.S.C. 4001-4127) requires that communities adopt adequate land use and control measures to qualify for insurance. Federal criteria promulgated to implement this provision contain the following requirements that can affect certain highways:

- In riverine situations, when the Administrator of the Federal Insurance Administration has identified the flood prone area, the community must require that, until a floodway has been designated, no use, including land fill, be permitted within the floodplain area having special flood hazards for which base flood elevations have been provided, unless it is demonstrated that the cumulative effect of the proposed use, when combined with all other existing and reasonably anticipated uses of a similar nature, will not increase the water surface elevation of the 100-year flood more than 1 ft at any point within the community.

- After the floodplain area having special flood hazards has been identified and the water surface elevation for the 100-year flood and floodway data have been provided, the community must designate a floodway which will convey the 100-year flood without increasing the water surface elevation of the flood more than 1 ft at any point and prohibit, within the designated floodway, fill, encroachments and new construction and substantial improvements of existing structures which would result in any increase in flood heights within the community during the occurrence of the 100-year flood discharge.

- The participating cities and/or counties agree to regulate new development in the designated floodplain and floodway through regulations adopted in a floodplain ordinance. The ordinance requires that development in the designated floodplain be consistent with the intent, standards and criteria set by the National Flood Insurance Program.

Additional information regarding the NFIP and the necessary actions regarding the construction of highways in floodplains and floodways in communities that have qualified for flood insurance is presented in Appendix 2B.

2.5 State Drainage Law

2.5.1 General

State drainage law is derived mainly from two sources: (1) common law and (2) statutory law.

- **Common law** is that body of principles which developed from immemorial usage and custom and which receives judicial recognition and sanction through repeated application. These principles were developed without legislative action and are embodied in the decisions of the courts.

- **Statutory law** is enacted by legislatures to enlarge, modify, clarify, or change the common law applicable to particular drainage conditions. This type of law is derived from constitutions, statutes, ordinances and codes.

In general, the common law rules of drainage predominate unless they have been enlarged or superseded by statutory law. In most instances where statutory provisions have been enacted, it is possible to determine the intent of the law. If, however, there is a lack of clarity in the statute, the point in question may have been litigated for clarification. In the absence of either clarity of the statute or litigation, a definitive statement of the law is not possible, although the factors that are likely to be controlling may be indicated.

2.5.2 Classification Of Waters

State drainage laws originating from common law, or court-made law, first classified the water that was being dealt with, after which the rule that was pertinent to the particular classification was applied to obtain a decision.

The first step in the evaluation of a drainage problem is to classify the water as surface water, stream water, floodwater, or groundwater. These terms are defined below. Once the classification has been established, the rule that applies to the particular class of water determines responsibilities with respect to disposition of the water.

- <u>Surface Waters</u> - Surface waters are those waters which have been precipitated on the land from the sky or forced to the surface in springs, and which have then spread over the surface of the ground without being collected into a definite body or channel.

- <u>Stream Waters</u> - Stream waters are former surface or ground waters which have entered and now flow in a well-defined natural watercourse, together with other waters reaching the stream by direct precipitation or rising from springs in the bed or banks of the watercourse. A watercourse in the legal sense refers to a definite channel with bed and banks within which water flows either continuously or intermittently.

- <u>Flood Waters</u> - Flood waters are former stream waters which have escaped from a watercourse (and its overflow channels) and flow or stand over adjoining lands. They remain floodwaters until they disappear from the surface by infiltration or evaporation, or return to a natural watercourse.

2.5 State Drainage Law (continued)

2.5.2 Classification Of Waters (continued)

- Ground Waters - In legal considerations, ground waters are divided into two classes, percolating waters and underground streams. The term "percolating waters" generally includes all waters which pass through the ground beneath the surface of the earth without a definite channel. The general rule is that all underground waters are presumed to be percolating and to take them out of the percolating class, the existence and course of a permanent channel must be clearly shown. Underground streams are waters passing through the ground beneath the surface in permanent, distinct, well-defined channels.

2.5.3 Disposition of Water

Two major rules have been developed by the courts regarding the disposition of surface waters. One is known as the civil law rule of natural drainage. The other is referred to as the common enemy doctrine. Modification of both rules has tended to bring them somewhat closer together, and in some cases the original rule has been replaced by a compromise rule known as the reasonable use rule.

Much of the law regarding stream waters is founded on a common law maxim that states "water runs and ought to run as it is by natural law accustomed to run." Thus, as a general rule, any interference with the flow of a natural watercourse to the injury or damage of another will result in liability. This may involve augmentation, obstruction and detention, or diversion of a stream. However, there are qualifications.

In common law, flood waters are treated as a "common enemy" of all people, lands and property attacked or threatened by them. In ground water law, the "English Rule," which is analogous to the common enemy rule in surface water law, is based on the doctrine of absolute ownership of water beneath the property by the landowner.

2.5.4 Civil Law - Surface Water

The civil law rule is based upon the perpetuation of natural drainage. The rule places a natural easement or servitude upon the lower land for the drainage of surface water in its natural course and the natural flow of the water cannot be obstructed by the servient owner to the detriment of the dominant owner. Most states following this rule have modified it so that the owner of upper lands has an easement over lower lands for drainage of surface waters and natural drainage conditions can be altered by an upper proprietor provided the water is not sent down in a manner or quantity to do more harm than formerly.

Under the common enemy doctrine, surface water is regarded as a common enemy which each property owner may fight off or control as he will or is able, either by retention, diversion, repulsion, or altered transmission. Thus, there is not cause of action even if some injury occurs causing damage. In most jurisdictions, this doctrine has been subject to a limitation that one must use his land so as not to unreasonably or unnecessarily damage the property of others.

2.5 State Drainage Law (continued)

2.5.4 Civil Law - Surface Water (continued)

Under the reasonable use rule, each property owner can legally make reasonable use of his land, even though the flow of surface waters is altered thereby and causes some harm to others. However, liability attaches when his harmful interference with the flow of surface water is "unreasonable." Whether a landowner's use is unreasonable is determined by a nuisance-type balancing test. The analysis involves several questions.

- Was there reasonable necessity for the actor to alter the drainage to make use of his land?
- Was the alteration done in a reasonable manner?
- Does the utility of the actor's conduct reasonably outweigh the gravity of harm to others?

2.5.5 Civil Law - Stream Waters

Where natural watercourses are unquestioned in fact and in permanence and stability, there is little difficulty in application of the rule. Highways cross channels on bridges or culverts, usually with some constriction of the width of the channel and obstruction by substructure within the channel, both causing backwater upstream and acceleration of flow downstream. The changes in regime must be so small as to be tolerable by adjoining owners, or there may be liability of any injuries or damages suffered.

Surface waters from highways are often discharged into the most convenient watercourse. The right is unquestioned if those waters were naturally tributary to the watercourse and unchallenged if the watercourse has adequate capacity. However, if all or part of the surface waters have been diverted from another watershed to a small watercourse, any lower owner may complain and recover for ensuing damage.

2.5.6 Civil Law - Flood Waters

Considering flood waters as a common enemy permits all affected landowners including owners of highways, to act in any reasonable way to protect themselves and their property from the common enemy. They may obstruct its flow from entering their land, backing or diverting water onto lands of another without penalty, by gravity or pumping, by diverting dikes or ditches, or by any other reasonable means.

Again, the test of "reasonableness" has frequently been applied, and liability can result where unnecessary damage is caused. Ordinarily, the highway designer should make provision for overflow in areas where it is foreseeable that it will occur. There is a definite risk of liability if such waters are impounded on an upper owner or, worse yet, are diverted into an area where they would not otherwise have gone. Merely to label waters as "flood waters" does not mean that they can be disregarded.

The "English Rule" has been modified by the "Reasonable Use Rule" which states in essence that each landowner is restricted to a reasonable exercise of his own right and a reasonable use of his property in view of the similar right of his neighbors.

2.5 State Drainage Law (continued)

2.5.6 Civil Law - Flood Waters (continued)

The key word is "reasonable." While this may be interpreted somewhat differently from case to case, it can generally be taken to mean that a landowner can utilize subsurface water on his property for the benefit of agriculture, manufacturing, irrigation, etc. pursuant to the reasonable development of his property although such action may interfere with the underground waters of neighboring proprietors. However, it does generally preclude the withdrawal of underground waters for distribution or sale for uses not connected with any beneficial ownership or enjoyment of the land from whence they were taken.

A further interpretation of "reasonable" in relation to highway construction would view the excavation of a deep "cut section" that intercepts or diverts underground water to the detriment of adjacent property owners as unreasonable. There are also cases where highway construction has permitted the introduction of surface contamination into subsurface waters and thus incurred liability for resulting damages.

2.6 Statutory Law

2.6.1 Introduction

The inadequacies of the common law or court-made laws of drainage led to a gradual enlargement and modification of the common law rules by legislative mandate. In the absence of statute, the common law rules adopted by State courts determine surface water drainage rights. If the common law rules have been enlarged or superseded by statutory law, the statute prevails. In general, statutes have been enacted that affect drainage in one way or another in the subject areas described below.

2.6.2 Eminent Domain

In the absence of an existing right, public agencies may acquire the right to discharge highway drainage across adjoining lands through the use of the right of eminent domain. Eminent domain is the power of public agencies to take private property for public use. It is important to remember, however, that whenever any property is taken under eminent domain, the State must show just cause and the private landowner must be compensated for his loss.

2.6.3 Water Rights

The water right which attaches to a watercourse is a right to the use of the flow, not ownership of the water itself. This is true under both the riparian doctrine and the appropriation doctrine. This right of use is a property right, entitled to protection to the same extent as other forms of property, and is regarded as real property. After the water has been diverted from the stream flow and reduced to possession, the water itself becomes the personal property of the riparian owner or the appropriator.

- Riparian Doctrine - Under the riparian doctrine, lands contiguous to watercourses have prior claim to waters of the stream solely by reason of location and regardless of the relative productive capacities of riparian and nonriparian lands.

2.5 State Drainage Law (continued)

2.6.3 Water Rights (continued)

- Doctrine of Prior Appropriation - The essence of this doctrine is the exclusive right to divert water from a source when the water supply naturally available is not sufficient for the needs of all those holding rights to its use. Such exclusive right depends upon the effective date of the appropriation, the first in time being the first in right.

Arizona generally operates under the Doctrine of Prior Appropriation. The important thing for highway designers to keep in mind in the matter of water rights is that proposed work in the vicinity of a stream should not impair either the quality or quantity of flow of any water rights to the stream.

2.7 Local Laws And Applications

2.7.1 Local Laws

Local governments (cities, counties, improvement districts) have ordinances and codes which require consideration during design. For example, zoning ordinances can have a substantial effect on the design of a highway and future drainage from an area. On occasion, a question may arise as to whether the State must comply with local ordinances. Generally, the State is not legally required to comply with local ordinances except where compliance is required by specific State statute. Quite often, however, the State may act in conformance with local ordinances as a matter of courtesy when it can be done without imposing a burden (cost) on the State. There are instances where the presence of an Executive Order from the Governor or by an Intergovernmental Agreement the State is to comply with local ordinances.

2.7.2 Special Matters

Irrigation Ditches - In situations in which an irrigation ditch intersects a drainage basin, the irrigation ditch does not have to take underground waters diverted by a tile-drain. However, the surface drainage must be accepted if the irrigation ditch is constructed in a way into which surface water would naturally flow.

2.8 Role Of The Designer

2.8.1 Responsibility

The designer has a two-fold responsibility for the legal aspects of highway drainage. First, the designer should know the legal principles involved and apply this knowledge to his designs; and, secondly, he should, as necessary, work closely with the legal staff of his organization in the preparation and trial of drainage cases. The duties of the designer include direct legal involvement in the following areas:

- conduct investigations, advise and provide expert testimony on the technical aspects of drainage claims involving existing highways; and
- provide drainage design information during right-of-way acquisition to assist appraisers in evaluating damages and provide testimony in subsequent condemnation proceedings, when necessary.

2.8 Role Of The Designer (continued)

2.8.2 Investigating Complaints

It is imperative that drainage complaints be dealt with promptly and in an unbiased manner. This means accepting the fact that the flooding is a serious problem for the complainer, and not accepting anyone's preconceived conclusions. All facts must be assembled and analyzed before deciding on what happened and why it happened. Also, it is well to list any other agency that could possibly have responsibility for a remedy to the flooding.

When the designer is requested to investigate a complaint, the following guidelines are recommended:

Step 1 **Determine Facts About The Complaint**

- Show on a map the location of the problem on which the complaint is based.

- Clearly determine the basis for the complaint (what was flooded, complainer's opinion as to what caused the flooding, description of the alleged damages, dates, times and durations of flooding).

- Briefly relate the history of any other grievances that were expressed prior to the claim presently being investigated.

- Obtain approximate dates that the damaged property and/or improvements were acquired by those claiming damages.

- Collect facts about the specific flood event(s) involved.

  Obtain rainfall data (dates, amounts, time periods and locations of gages). Rainfall data are often helpful regardless of the source.

  Document observed highwater information at or in the vicinity of the claim. Locate highwater marks on a map and specify datum. Always try to obtain highwater marks both upstream and downstream of the highway and the time the elevations occurred.

  Determine the duration of flooding at the site of alleged damage. Determine the direction of flood flow at the damaged site. Describe the condition of the stream before, after, and during flood(s). Was the growth in the channel light, medium, heavy; were there drift jams; does the stream carry much drift in flood stage; was the flow fast or sluggish; did light, moderate, or severe erosion occur?

  Document the flood history at the site. Was the highway overtopped by the flood? If so, what was the depth of overtopping; and, if possible, estimate a flow velocity across the highway. Obtain narratives of any eyewitnesses to the flooding. Obtain facts about the flood(s) from sources outside the Department, such as newspaper accounts, witnesses, measurements by other agencies (USGS, Corps of Engineers, SCS and individuals), maps and Weather Bureau rainfall records.

2.8 Role Of The Designer (continued)

2.8.2 Investigating Complaints (continued)

Step 1 Determine Facts About The Complaint (continued)

- State facts about the highway crossing involved.

  Show profile of the highway across the stream valley. Give the date of the original highway construction and dates of all subsequent alterations to the highway, and describe what the alterations were. Describe what existed prior to the highway, such as county road, city street, or abandoned railroad embankment, etc. Also include a description of the drainage facilities and drainage patterns that were there prior to the highway. Give a description of the existing drainage facilities. Give the original drainage design criteria, or give capacity and frequency of the existing facility based upon current criteria.

- List possible effects by others.

  Are there any other stream crossings in the vicinity of the damaged site that could have affected the flooding (pipelines, highways, streets, railroads, dams)? Have there been any significant man-made changes to the stream or watershed that might affect the flooding?

Step 2 Analyze The Facts

From the facts decide what should be done to relieve the problem regardless of who has responsibility for the remedy. Could others possibly provide assistance?

Step 3 Make Conclusions And Recommendations

- What were the contributing factors leading to the alleged flood damage?

- Specify feasible remedies. (This should be done without any regard for who has responsibility to affect a remedy.)

The list under Determine Facts About The Complaint is not all-inclusive, nor is it intended that the entire list will be applied in each case. This outline is given as a guide to the type and scope of information desired from an investigation of a drainage complaint. It is advantageous to have available hydraulic design documentation as outlined in the Documentation Chapter of this manual. When the report is completed, the designer should again analyze the facts, consider the conclusions and recommendations and prepare a response to the complainer explaining the results of the investigation. Documentation of the facts and findings is important in the event there is future action.

2.8.3 As A Witness

The designer should accept the responsibility of providing expert testimony in highway drainage litigation. Witness duty ordinarily requires considerably more time of a witness than the time spent in the courtroom. The best use of the designer's time can be arranged by consulting with legal counsel to determine what types of information and data will be needed, types of presentation needed and when testimony will be required.

2.8 Role Of The Designer (continued)

2.8.3 As A Witness (continued)

Testimony often involves presenting technical facts in layman's language so that it will be clearly understood by those in the courtroom. The designer's testimony generally describes the highway drainage system involved in the alleged injury or damage, and how that system affects the complainant. Design considerations and evidence of conditions existing prior to construction of the highway are important points.

The designer who is to serve as a witness should bear one fact in mind; the purpose of the court is to administer justice. Testimony should have one purpose -- to bring out all known facts relevant to the case so that justice can better be served. Following are some pointers in being a witness:

- Do not be apprehensive. Your purpose is to present the facts as you know them and that is all that will be expected.
- Tell the truth and do not try to color, shade or change your testimony to help either side.
- Do not try to memorize your story. There is no more certain way to cross yourself than to memorize your story and try to fit this story with the questions being asked.
- Work with your lawyer in preparing your testimony and stick to the facts as you know them.
- Stick to the facts and what you personally know.
- Do not be afraid of lawyers and give your information honestly.
- Speak clear and loud enough to be heard by everyone involved in the courtroom proceeding.
- Answer all questions directly and never volunteer information the question does not ask for.
- If you do not understand a question, ask that it be explained. If you still do not understand what is being asked, explain that you cannot give an answer to that question.
- Remember that some questions do not have to be answered with a yes or no.
- If you do not know the answer to a question, just admit it. It is to your credit to be honest, rather than try to have an answer for everything that is asked you.
- Never lose your temper or show prejudice in favor of one side that is not supported by facts.

2.9 References

American Association of State Highway and Transportation Officials. Highway Drainage Guidelines, Chapter V — The Legal Aspects of Highway Drainage. 1982.

Federal Emergency Management Agency. National Flood Insurance Program and Related Regulations. 1987.

U.S. Army Corps of Engineers. Handbook of How to Compute a Floodway. 1987. (Copies of this publication can be obtained from — FEMA Region V, 175 West Jackson Blvd., Fourth Flood, Chicago Illinois 60604.)

Appendix A- Federal Policies

A.1 Introduction

The following section lists the Federal legislation which contains the Federal policies which might affect drainage design and construction. This section gives the legislative reference, regulations reference, purpose, applicability, general procedures and agency for coordination and consultation. For more detailed information about specific Federal policies, the applicable legislation should be consulted. Note: Abbreviations are given at the end of this section.

A.2 Environmental

1. National Environmental Policy Act: 42 U.S.C. 4321-4347 (P.L. 91-190 and 94-81). Reference - 23 CFR 770-772, 40 CFR 1500-1508, CEQ Regulations, Executive Order 11514 as amended by Executive Order 11991 on NEPA responsibilities.
   Purpose - consider environmental factors through systematic interdisciplinary approach before committing to a course of action.
   Applicability - all highway projects.
   General Procedures - Procedures set forth in CEQ regulations and 23 CFR 771.
   Coordination - appropriate Federal, State, and local agencies.

A.3 Land And Water Usage

These requirements are contained in 23 CFR 771 (FHPM 7-7-1).

1. **Executive Order 11990, Protection of Wetlands, DOT Order 5660.1A, 23 CFR 777.**
   Purpose - to avoid direct or indirect support of new construction in wetlands wherever there is a practicable alternative.
   Applicability - federally undertaken, financed, or assisted construction, and improvements in or with significant impacts on wetlands.
   General Procedures - evaluate and mitigate impacts on wetlands. Specific finding required in final environmental document.
   Coordination - DOI (FWS), EPA, USCE, NMFS, NRCS, State agencies.

   Executive Order 11990, May 24, 1977, orders each Federal agency to:
   - take action to minimize the destruction, loss or degradation of wetlands and to preserve and enhance the natural and beneficial values to wetlands;
   - avoid undertaking or providing assistance for new construction in wetlands unless the head of the agency finds that there is no practicable alternative and all practicable measures are taken to minimize harm which may result from the action; and
   - to consider factors relevant to the proposal's effects on the survival and quality of the wetlands.

Appendix A- Federal Policies

A.3 Land And Water Usage (continued)

2. **Emergency Wetlands Resources Act of 1986: 16 U.S.C. - 3901 note (P.L. 99-645).**
   Purpose - to promote the conservation of wetlands in the U. S. in order to maintain the public benefits they provide.
   Applicability - all projects which may impact wetlands.
   General Procedures - (1) preparation of a National Wetlands Priority Conservation Plan which provides priority with respect to Federal and State acquisition, (2) provides direction for the National Wetlands Inventory Project.
   Coordination - FWS.

3. **Federal Water Pollution Control Act (1972), as amended by the Clean Water Act (1977 & 1987): 33 U.S.C. 1251-1376 (P.L. 92-500, 95-217, 100-4), DOT Order 5660.1A, FHWA Notices N5000.3 and N5000.4, FHPM 6-7-3-3, 23 CFR 650, Subpart B, E, 771, 33 CFR 209, 40 CFR 120, 122-125, 128-131, 133, 125-136, 148, 230-231.**
   Purpose - restore and maintain chemical, physical, and biological integrity of the Nation's waters through prevention, reduction, and elimination of pollution.
   Applicability - any discharge of a pollutant into waters of the U.S.
   General Procedures - (1) obtain permit for dredge or fill material from USCE or State agency, as appropriate (Section 404), (2) permits for all other discharges are to be acquired from EPA or appropriate State agency (Section 402), (3) water quality certification is required from State water resource agency (Section 401), (4) all projects shall be consistent with the State nonpoint source pollution management program (Section 319).
   Coordination - USCE, EPA, designated State water quality control agency, designated State non-point source pollution agency.

4. **Executive Order 11988, Floodplain Management, as amended by Executive Order 12148, DOT Order 5650.2, FHPM 6-7-3-2, 23 CFR 650, Subpart A, 771.**
   Purpose - to avoid the long- and short-term adverse impacts associated with the occupancy and modification of floodplains, and to restore and preserve the natural and beneficial values served by floodplains.
   Applicability - all construction of Federal or federally aided buildings, structures, roads, or facilities which encroach upon or affect the base floodplain.
   General Procedures - (1) assessment of flood hazards, (2) specific finding required in final environmental document.
   Coordination - FEMA, State and local agencies.

   Executive Order 11988, May 24, 1977, requires each Federal agency, in carrying out its activities, to take the following actions:

   - to reduce the risk of flood loss, to minimize the impact of floods on human safety, health and welfare and to restore and preserve the natural and beneficial values served by floodplains; and
   - to evaluate the potential effect of any actions it may take in a floodplain, to ensure its planning programs reflect consideration of flood hazards and floodplain management.

Appendix A- Federal Policies

A.3 Land And Water Usage (continued)

4. **Executive Order 11988, Floodplain Management, (continued)**
   These requirements are contained in the Federal-Aid Highway Program Manual (FHPM), Volume 6, Chapter 7, Section 3, Subsection 2, and were published in the Federal Register, April 26, 1979 (44 FR 24678), and in 23 CFR 650, Subpart A.
5. **Endangered Species Act of 1973, as amended: 16 U.S.C. 1531-1543 (P.L. 93-205, 94-359, 95-632, 96-159, 97-304), 7 CFR 355, 50 CFR 17, 23, 25-29, 81, 217, 222, 225-227, 402, 424, 450-453.**
   Purpose - conserve species of fish, wildlife and plants facing extinction.
   Applicability - any action that is likely to jeopardize continued existence of such endangered/threatened species or result in destruction or modification of critical habitat.
   General Procedures - consult with the Secretary of the Interior or Commerce, as appropriate.
   Coordination - DOI (FWS), Commerce (NMFS).

6. **Fish and Wildlife Coordination Act: 16 U.S.C. 661-666c (P.L. 85-624, 89-72, 95-616).**
   Purpose - conservation, maintenance, and management of wildlife resources.
   Applicability - (1) any project which involves impoundment (surface area of 4.05 hectares or more), diversion, channel deepening, or other modification of a stream or other body of water, (2) transfer of property by Federal agencies to State agencies for wildlife conservation purpose.
   General Procedures - coordinate early in project development with FWS and State fish and wildlife agency.
   Coordination - DOI (FWS), State fish and wildlife agencies.

Appendix A- Federal Policies

A.4 Abbreviations

Following are the abbreviations used in the above descriptions of Federal policies.

BIA - Bureau of Indian Affairs
BLM - Bureau of Land Management
CEQ - Council on Environmental Quality
CERCLA - Comprehensive Environmental Response, Compensation, and Liability Act
CFR - Code of Federal Regulations
DOA - Department of the Army
DOD - Department of Defense
DOI - Department of the Interior
DOT - Department of Transportation
EPA - Environmental Protection Agency
FEMA - Federal Emergency Management Agency
FHPM - Federal-Aid Highway Program Manual
FHWA - Federal Highway Administration
FIFRA - Federal Insecticide, Fungicide, and Rodenticide Act
FWPCA - Federal Water Pollution Control Act
FWS - Fish and Wildlife Service
HUD - Housing and Urban Development
NMFS - National Marine Fisheries Service
NPS - National Park Service
NRCS - National Resource Conservation Service; formerly Soil Conservation Service (SCS)
OCZM - Office of Coastal Zone Management
P.L. - Public Law
RCRA - Resource Conservation and Recovery Act
SARA - Superfund Amendments and Reauthorization Act
SEE - Social, Economic, and Environmental
SIP - State Implementation Plan
Stat. - Statute
TVA - Tennessee Valley Authority
UMTA - Urban Mass Transportation Administration
U.S.C. - United States Code
USCE - U.S. Corps of Engineers
USCG - U.S. Coast Guard
USFS - U.S. Forest Service

Appendix B Federal Emergency Management Agency (FEMA)

B.1 National Flood Insurance Program

The Flood Disaster Protection Act of 1973 (Pl 93-234, 87 Stat. 975) denies Federal financial assistance to flood prone communities that fail to qualify for flood insurance. Formula grants to States are excluded from the definition of financial assistance, and the definition of construction in the Act does not include highway construction; therefore, Federal aid for highways is not affected by the Act. The Act does require communities to adopt certain land use controls in order to qualify for flood insurance. These land use requirements could impose restrictions on the construction of highways in floodplains and floodways in communities that have qualified for flood insurance. A floodway, as used here and as used in connection with the National Flood Insurance Program, is that portion of the floodplain required to pass a flood that has a 1-percent chance of occurring in any 1-year period without cumulatively increasing the water surface elevation more than 1 ft (0.3 m).

B.2 Flood Insurance

The National Flood Insurance Act of 1968, as amended, (42 U.S.C. 4001-4127) requires that communities adopt adequate land use and control measures to qualify for insurance. Federal criteria promulgated to implement this provision contain the following requirements that can affect certain highways.

In riverine situations, when the Administrator of the Federal Insurance Administration has identified the flood prone area, the community must require that, until a floodway has been designated, no use, including land fill, be permitted within the floodplain area having special flood hazards for which base flood elevations have been provided, unless it is demonstrated that the cumulative effect of the proposed use, when combined with all other existing and reasonably anticipated uses of a similar nature, will not increase the water surface elevation of the 100-year flood more than 1 ft (0.3 m) at any point within the community.

- After the floodplain area having special flood hazards has been identified and the water surface elevation for the 100-year flood and floodway data have been provided, the community must designate a floodway which will convey the 100-year flood without increasing the water surface elevation of the flood more than 1 ft (0.3 m) at any point and prohibit, within the designated floodway, fill, encroachments and new construction and substantial improvements of existing structures which would result in any increase in flood heights within the community during the occurrence of the 100-year flood discharge.

- The participating cities and/or counties agree to regulate new development in the designated floodplain and floodway through regulations adopted in a floodplain ordinance. The ordinance requires that development in the designated floodplain be consistent with the intent, standards and criteria set by the National Flood Insurance Program.

Appendix B Federal Emergency Management Agency (FEMA)

B.3 Local Community

The local community with land use jurisdiction, whether it is a city, county, or State, has the responsibility for enforcing National Flood Insurance Program (NFIP) regulations in that community if the community is participating in the NFIP. Consistency with NFIP standards is a requirement for Federal-aid highway actions involving regulatory floodways. The community, by necessity, is the one who must submit proposals to Federal Emergency Management Agency (FEMA) for amendments to NFIP ordinances and maps in that community should it be necessary. The highway agency should deal directly with the community and, through them, deal with FEMA. Determination of the status of a community's participation in the NFIP and review of applicable NFIP maps and ordinances are, therefore, essential first steps in conducting location hydraulic studies and preparing environmental documents.

B.4 NFIP Maps

Where NFIP maps are available, their use is mandatory in determining whether a highway location alternative will include an encroachment on the base floodplain. Three types of NFIP maps are published:

- Flood Hazard Boundary Map (FHBM),
- Flood Boundary and Floodway Map (FBFM), and
- Flood Insurance Rate Map (FIRM).

A FHBM is generally not based on a detailed hydraulic study and, therefore, the floodplain boundaries shown are approximate. A FBFM, on the other hand, is generally derived from a detailed hydraulic study and should provide reasonably accurate information. The hydraulic data from which the FBFM was derived are available through the regional office of FEMA. This is normally in the form of computer input data records for calculating water surface profiles. The FIRM is generally produced at the same time using the same hydraulic model and has appropriate rate zones and base flood elevations added.

Communities may or may not have published one or more of the above maps depending on their level of participation in the NFIP. Information on community participation in the NFIP is provided in the "National Flood Insurance Program Community Status Book" which is published semiannually for each State.

B.5 Coordination With FEMA

It is intended that there will be coordination with local Flood Plain Administrator in situations where administrative determinations are needed involving a regulatory floodway or where flood risks in NFIP communities are significantly impacted. The circumstances which would ordinarily require coordination with local Floodplain Administrator include the following:

- When a proposed crossing encroaches on a regulatory floodway and, as such, would require an amendment to the floodway map.
- When a proposed crossing encroaches on a floodplain where a detailed study has been performed but no floodway designated and the maximum 1-ft (0.3-m) increase in the base flood elevation would be exceeded.

Appendix B Federal Emergency Management Agency (FEMA)

B.5   Coordination With FEMA (continued)

- When a local community is expected to enter into the regular program within a reasonable period and detailed floodplain studies are under way.
- When a local community is participating in the emergency program and base FEMA flood elevation in the vicinity of insurable buildings is increased by more than 1-ft (0.3-m). Where insurable buildings are not affected, it is sufficient to notify FEMA of changes to base flood elevations as a result of highway construction.

The draft Environmental Impact Statement or Environmental Assessment (EIS/EA) should indicate the NFIP status of affected communities, the encroachments anticipated and the need for floodway or floodplain ordinance amendments. Coordination means furnishing to FEMA the draft EIS/EA and, upon selection of an alternative, furnishing to FEMA, through the community, a preliminary site plan and water surface elevation information and technical data in support of a floodway revision request as required. If a determination by FEMA would influence the selection of an alternative, a commitment from FEMA should be obtained prior to the final environmental impact statement (FEIS) or a finding of no significant impact (FONSI). Otherwise this later coordination may be postponed until the design phase.

B.6   Consistent With Floodways

In many situations it is possible to design and construct highways in a cost-effective manner such that their components are excluded from the floodway. This is the simplest way to be consistent with the standards and should be the initial alternative evaluated. If a project element encroaches on the floodway but has a very minor effect on the floodway water surface elevation (such as piers in the floodway), the project may normally be considered as being consistent with the standards, if hydraulic conditions can be improved so that no water surface elevation increase is reflected in the computer printout for the new conditions.

B.7   Revisions Of Floodway

Where it is not cost-effective to design a highway crossing to avoid encroachment on an established floodway, a second alternative would be a modification of the floodway itself. Often, the community will be willing to accept an alternative floodway configuration to accommodate a proposed crossing provided NFIP limitations on increases in the base flood elevation are not exceeded. This approach is useful where the highway crossing does not cause more than a 1-ft (0.3-m) rise in the base flood elevation. In some cases, it may be possible to enlarge the floodway or otherwise increase conveyance in the floodway above and below the crossing in order to allow greater encroachment. Such planning is best accomplished when the floodway is first established. However, where the community is willing to amend an established floodway to support this option, the floodway may be revised.

The responsibility for demonstrating that an alternative floodway configuration meets NFIP requirements rests with the community. However, this responsibility may be borne by the agency proposing to construct the highway crossing. Floodway revisions must be based on the hydraulic model which was used to develop the currently effective floodway but updated to reflect existing encroachment conditions. This will allow determination of the increase in the base flood elevation that has been caused by encroachments since the original floodway was established. Alternate floodway configurations may then be analyzed.

Appendix B Federal Emergency Management Agency (FEMA)

B.7 Revisions Of Floodway (continued)

Base flood elevations increases are referenced to the profile obtained for existing conditions when the floodway was first established.

B.8 Data For Revisions

Data submitted to FEMA, through the community, in support of a floodway revision request should include the following.

- Copy of current regulatory Flood Boundary Floodway Map, showing existing conditions, proposed highway crossing and revised floodway limits.
- Copy of computer printouts (input, computation and output) for the current 100-year model and current 100-year floodway model.
- Copy of computer printouts (input, computation and output) for the revised 100-year floodway model. Any fill or development that has occurred in the existing flood fringe area must be incorporated into the revised 100-year floodway model.
- Copy of engineering certification is required for work performed by private subcontractors.

The revised and current computer data required above should extend far enough upstream and downstream of the floodway revision area in order to tie back into the original floodway and profiles using sound hydraulic engineering practices. This distance will vary depending on the magnitude of the requested floodway revision and the hydraulic characteristics of the stream.

If input data representing the original hydraulic model are unavailable, an approximation should be developed. A new model should be established using the original cross section topographic information, where possible, and the discharges contained in the Flood Insurance Study that established the original floodway. The model should then be run confining the effective flow area to the currently established floodway and calibrate to reproduce within 0.10 ft the "With Floodway" elevations provided in the Floodway Data Table for the current floodway. Floodway revisions may then be evaluated using the procedures outlined above.

B.9 Allowable Floodway Encroachment

When it would be demonstrably inappropriate to design a highway crossing to avoid encroachment on the floodway and where the floodway cannot be modified such that the structure could be excluded, FEMA will approve an alternate floodway with backwater in excess of the 1-ft maximum only when the following conditions have been met:

- A location hydraulic study has been performed in accordance with Federal-Aid Highway Program Manual (FHPM) 6-7-3-2, FHWA, "Location and Hydraulic Design of Encroachments on Flood-plains" (23 CFR 650, Subpart A) and FHWA finds the encroachment is the only practicable alternative.
- The constructing agency has made appropriate arrangements with affected property owners and the community to obtain flooding easements or otherwise compensate them for future flood losses due to the effects of backwater greater than 1 ft.

Appendix B Federal Emergency Management Agency (FEMA)

B.9 Allowable Floodway Encroachment (continued)

- The constructing agency has made appropriate arrangements to assure that the National Flood Insurance Program and Flood Insurance Fund will not incur any liability for additional future flood losses to existing structures which are insured under the Program and grandfathered in under the risk status existing prior to the construction of the structure.
- Prior to initiating construction, the constructing agency provides FEMA with revised flood profiles, floodway and floodplain mapping and background technical data necessary for FEMA to issue revised Flood Insurance Rate Maps and Flood Boundary and Floodway Maps for the affected area, upon completion of the structure.

Highway Encroachment On A Floodplain With A Detailed Study (FIRM)

In communities where a detailed flood insurance study has been performed but no regulatory floodway designated, the highway crossing should be designed to allow no more than 1-ft increase in the base flood elevation based on technical data from the flood insurance study. Technical data supporting the increased flood elevation shall be submitted to the local community and through them to FEMA for their files.

Highway Encroachment On A Floodplain Indicated On An FHBM

In communities where detailed flood insurance studies have not been performed, the highway agency must generate its own technical data to determine the base floodplain elevation and design encroachments in accordance with FHPM 6-7-3-2. Base floodplain elevations shall be furnished to the community, and coordination carried out with FEMA as outlined previously where the increase in base flood elevations in the vicinity of insurable buildings exceeds 1 ft.

Highway Encroachment on Unidentified Floodplains

Encroachments that are outside of NFIP communities or NFIP identified flood hazard areas should be designed in accordance with FHPM 6-7-3-2 of the Federal Highway Administration.

B.10 Levee Systems

For purposes of the National Flood Insurance Program (NFIP), FEMA will only recognize in its flood hazard and risk mapping effort those levee systems that meet, and continue to meet, minimum design operation, and maintenance standards that are consistent with the level of protection sought through the comprehensive floodplain management criteria as outlined in the NFIP. The levee system must provide adequate protection from the base flood. Information supporting this must be supplied to FEMA by the community or other party seeking recognition of such a levee system at the time a flood risk study or restudy is conducted, when a map revision is sought based on a levee system, and upon request by the Administrator during the review of previously recognized structures. The FEMA review will be for the sole purpose of establishing appropriate risk zone determinations for NFIP maps and shall not constitute a determination by FEMA as to how a structure or system will perform in a flood event. For more information on the requirements related to levee systems see "National Flood Insurance Program and Related Regulations", Federal Emergency Management Agency, Revised October 1, 1986 and Amended June 30, 1987 (44 CFR 65.10).

CHAPTER 3

DESIGN PHILOSOPHY

Chapter 3 Design Philosophy
Table Of Contents

3.1 Introduction

3.1.1 Purpose

ADOT Policies for Highway Drainage are presented in Chapter 600 of the Roadway Design Guidelines. The purpose of this chapter is to outline design philosophies that guide and determine the multitude of variables that influence drainage design. The following sections of this chapter will present information concerning hydraulic design of drainage structures and related Federal and ADOT policies.

3.1.2 Policy vs. Criteria

Policy and criteria statements are closely related - criteria being the numerical or specific guidance that is founded in broad policy statements. For this manual, the following definitions of policy and criteria will be used:

Policy - is a definite course of action or method of action, selected to guide and determine present and future decisions.

> Following is an example of a policy statement:
> "The designer will size the drainage structure to accommodate a flood compatible with the projected traffic volumes."

Design Criteria - are the standards by which a policy is carried out or placed in action.

> The design criteria for designing the structure might be:
> "For projected traffic volumes less than or equal to 750 vehicles per day, drainage structures shall be designed for a 10-year flood (exceedence probability - 10%). For projected traffic volumes greater than 750 vehicles per day, a drainage structure shall be designed for a 25-year flood (exceedence probability - 4%)."

Policy statements describe the goal, design criteria provides the measurement for determining if the goal is achieved.

3.2 General Hydraulic Design Philosophy

3.2.1 Introduction

An "adequate" drainage structure may be defined as one that meets the following policies:

- the design of the structure meets or exceeds ADOT standard engineering practice, and
- the design is consistent with what a reasonably competent and prudent designer would do under similar circumstances.

3.2 General Hydraulic Design Philosophy (continued)

3.2.1 Introduction (continued)

General Policies

In determining an “adequate” drainage structure, the following points are made in regard to the design process.:

- It is the designer's responsibility to provide an adequate drainage structure for the design conditions established. The designer is not required to provide a structure that will handle all conceivable flood flows under all possible site conditions.
- The detail of design studies should be commensurate with the risk associated with the encroachment and with other economic, engineering, social, or environmental concerns.
- The design flood event serves as criteria for evaluating the adequacy of a proposed design. The "design flood" is the recurrence interval of the flood for which the drainage structure is sized to satisfy the varied criteria determined applicable.
- The predicted value of the base (100-year) flood serves as the present engineering standard for evaluating flood hazards and as the basis for regulating floodplains under the National Flood Insurance Program
- It is standard engineering practice to use the predicted value of the 100-year flood as the basis for evaluating flood hazards; however, flows larger than this value may be considered for complex, high risk or unusual cases that require special studies or risk analyses.
- The developed hydraulic performance of a drainage structure depicts the relationship between floodwater stage (or elevation) and flood flow magnitudes. For drainage facilities that affect developed properties, the performance data should include the 100-year flood. With the performance data, the designer can evaluate the adequacy of the design.

The design process usually has several phases: concept development, preliminary design, and final design. For the design of many drainage structures these phases are rapidly stepped through, especially for experienced designers. In working thorough the phases the studies listed below are normally conducted as a part of the design of highway drainage structures and serve as a means of achieving an “adequate” drainage design:

- hydrologic analysis,
- hydraulic analysis, and
- engineering evaluation of developed alternatives.

These studies are discussed further in the following sections.

3.2 General Hydraulic Design Philosophy (continued)

3.2.2 Hydrologic Analysis

The initial step is to determine the hydraulic loading, i.e., the flood flow to be accommodated. The designer must ascertain the magnitude of the flood flow using the methods and judgments appropriate for the situation under consideration. ADOT hydrology methods and practices are described in the ADOT Hydrology Manual.

3.2.3 Hydraulic Analysis

The next step in the design process involves preliminary selections of alternative designs that are judged to meet the site conditions and to accommodate the flood flows selected for analysis. The hydraulic analysis is made utilizing appropriate formulas, physical models or computer programs for purposes of defining, calibrating and checking the performance of the preliminary designs for the range of flows to be considered.

3.2.4 Engineering Evaluation

The final step in the design process is the engineering evaluation of the preliminary designs and selection of the final design. Engineering evaluation is the approach followed in defining, evaluating, and selecting a final design. This process involves consideration and balancing of a number of factors. Some of these factors are:

- effectiveness in achieving desired results,
- costs including construction and maintenance,
- legal considerations,
- flood hazards to highway users and neighboring property owners,
- environmental and social concerns, and
- other site specific concerns.

3.3 Coordination

3.3.1 Interagency Coordination

The Arizona Department of Transportation (ADOT) in the process of developing transportation facilities will traverse many watersheds; department coordination with responsible local agencies is essential to ensure that proposed facilities are compatible with the long-term plans for the area. Coordination between concerned agencies during the project planning phase will help produce a design that is more satisfactory to all. Discussions with the local floodplain administrator should identify local studies, concerns, and any partnership opportunities. Coordination may also be necessary with other State and Federal agencies. See Chapter 2 for the individual responsibility of Federal agencies.

3.3 Coordination

3.3.2 Intra-agency Coordination

Early planning and location studies are conducted by the Predesign Section of the Roadway Engineering Group. Drainage designs performed during the pre-design phase shall be coordinated with the Drainage Section so that duplication of effort is minimized and all those who might be involved in future project work will be informed of any ongoing studies and study results.

3.4 Stormwater Management Plan

3.4.1 Introduction

During the concept development phase, a stormwater management plan should be developed that adequately address project goals. The storm water management plan should specifically identify the criteria of the many elements that are to be considered in achieving a design that meets or exceeds the required and/or desired performance.

For the areas under consideration the following questions must be addressed:

- What is the impact of the facility on the existing conditions?
- What is the impact of the existing condition on the facility?

The elements that are to be addressed are discussed below.

3.4.2 Quantity

Determinations of stormwater quantity are necessary for evaluating the impact of a project. The methods discussed in the ADOT Hydrology manual are to be used to determine the magnitude of discharges to be evaluated.

3.4.3 Flood Hazards

Floodflow characteristics at a highway stream crossing should be carefully analyzed to determine their effect upon the highway as well as to evaluate the effects of the highway upon the floodflow. It is important to identify flood hazards prior to any highway involvement to determine how the flood hazard will be affected with the proposed highway project. Flood hazards should include effects to private property both upstream and downstream (i.e., changes to flooding such as overtopping floodwaters diverted onto previously unaffected property). The impacts of any proposed action must be understood before the final selection is made of the recommended alternative. The number one goal in dealing with off-project flows is to perpetuate the natural drainage conditions.

3.4 Stormwater Management Plan (continued)

3.4.4 Floodplain Encroachment Considerations

A primary drainage consideration for facility sizing of a stream crossing is the evaluation of the impact of floodplain encroachments. Hydraulic and environmental considerations of highway river crossings and encroachments are presented in the FHWA Highways in the River Environment, Training and Design Manual (1990). The Manual provides examples of typical river environments and identifies possible local, upstream and downstream effects of highway encroachments.

The principal factors to be considered when designing a stream crossing that involves encroachment within a floodplain are:

- river type (straight or meandering),
- river characteristics (stable or unstable),
- river geometry and alignment,
- hydrology,
- hydraulics,
- floodplain flow,
- economic (land use and land ownership) considerations, and
- environmental considerations.

A detailed evaluation of these factors is part of the hydraulics study. Specific crossing components to be determined include:

- the geometry and length of the approaches to the crossing,
- the location of the longitudinal encroachment in the floodplain,
- the amount of allowable longitudinal encroachment into the main channel,
- the type and size of structure, bridge or culvert, and the means to ensure the stability of the structure against flood flows, and
- the required river training works to ensure that river flows approach the crossing or the encroachment in a complementary way.

3.4.5 Environmental Considerations

It is important to document the drainage considerations that affect the environment for the proposed project including all alternatives that will receive consideration. The identification of drainage impacts on environmental considerations early in the planning process can prevent major implementation problems as the project development proceeds.

3.4 Stormwater Management Plan (continued)

3.4.6 Water Quality

The two major sources of contaminants into surface waters are from soil erosion and deposition and from the deposited contaminants on the roadway surface. In general, erosion and sediment transport should be limited by developing and implementing an erosion and sediment control plan which addresses both temporary and permanent control practices. Information regarding temporary erosion and sediment control is provided in the **ADOT Erosion and Pollution Control Manual**. Background information is provided in the Erosion and Sediment Control Chapter of this manual. Permanent erosion control measures are presented in the energy dissipator and bank protection chapters.

Common impacts of excessive erosion include:
- Turbidity which reduces in-stream photosynthesis and results in reduced food supply and aquatic habitat,
- Introduction of soil nutrients into waters that cause algal blooms, which reduces water clarity and depletes oxygen,
- Sedimentation of stream bottoms that blankets fauna and destroys spawning areas, and
- Removal of top soil that leaves hard, rocky and infertile soil, which is difficult to revegetate.

Quantification of the levels of contaminants that are being washed off a roadway is complicated by the variable effects of and the periods between storm events. The contributory factors are rainfall intensity, roadway surface characteristics and particle size. The varying interaction of these factors makes it difficult to precisely estimate the impact that discharge will have on water quality. A listing and description of common contaminants found on roadways is presented in Appendix 3-A, Table 3-1. The table includes examples of the contaminants, the analytical determination for identifying them and their primary sources.

Several broad categories of degradation have been developed to delineate or describe levels of stormwater impacts:
- Aesthetic deterioration: Undesirable general appearance features (dirty, turbid, or cloudy) and actual physical features (odors, floating debris, oil films, scum, or slime) are present.
- Dissolved oxygen depletion: When the oxygen demand of bacteria is stimulated by the organics, the subsequent reduction in oxygen levels can disturb the balance between lower forms and the food chain. Unoxidized nitrogen compounds (ammonia) can also cause problems.
- Pathogen concentrations: High concentrations of several pathogens can reduce the acceptable uses of the receiving waters.
- Suspended solids: The physical buildup of solids can cover productive bottoms, be aesthetically objectionable and disrupt flow and navigation.
- Nutrients: Accelerated eutrophication that stimulates growth of aquatic vegetation can cause a water body to become aesthetically objectionable, deplete dissolved oxygen and decrease recreational value by creating odor and overgrowth. Advanced eutrophication can lead to sediment buildup, which reduces storage capabilities.

3.4 Stormwater Management Plan (continued)

3.4.6 Water Quality (continued)

- Toxicity: The two types of toxins generally found in stormwater (metals and pesticides/persistent organics) may build up in sensitive areas over the long term. At high levels, they can have serious shock effects on aquatic life. Low levels can become significant by accumulation up the flood chain.
- Hazardous spills: Depending on the characteristics of the spill, serious water quality problems can result.

3.4.7 Permits

Specific Federal and State drainage permits that will be needed for a highway project must be identified early in the planning stages. Prior to initiating design work, the designer must review the environmental document to identify regulatory commitments, constraints and any permits required. The permits required are usually:

- stormwater discharge permits, (NPDES)
- dredge and fill permits,(404)

3.4.8 Construction Considerations

Many serious construction problems arise because important drainage and water-related factors are overlooked or neglected in the planning and development phases of the project. Such problems include:

- soil erosion,
- sediment deposition,
- pollution of streams, lakes, and rivers,
- destruction of wildlife habitat,
- destruction of wetlands, and
- impairment of utility systems.

With proper planning, many problems can be avoided or cost effective solutions developed to minimize damages. Consideration of these possible problems is required during context-sensitive design development.

3.4.9 Maintenance Considerations

The stormwater management plan must recognize the need for erosion and sediment controls and provide a conceptual approach for managing the impacts. The maintenance requirements of possible alternatives must be evaluated. Experience in the area is the best indicator of maintenance problems and interviews with maintenance personnel could be extremely helpful in identifying maintenance concerns with potential designs. Reference to highway maintenance and flood reports, damage surveys, newspaper clippings and interviews with local residents could be helpful in evaluating potential maintenance problems.

3.5 State Policies

The applicable State policies regarding design criteria are presented in Chapter 600 of the ADOT Roadway Design Guidelines and in the ADOT Erosion and Pollution Control Manual.

Appendix A

Table 3-1
Listing of Common Stormwater Contaminants

Classification	Examples	Analytical Determination	Primary Sources
Particulates	Dust and dirt, stones, sand gravel, grain, glass, plastics, metals, fine residue	Settleable solids	Pavement, vehicle, atmosphere, litter, maintenance
Heavy metals	Lead, zinc, iron, copper, nickel, chromium, mercury	Specific heavy metal via atmospheric absorption	Vehicle, atmospheric fallout and washout
PCB, pesticides, herbicides	Chlorinated hydrocarbons, organic-phosphorous	Gas chromatography	Spraying of vegetation
Inorganic salts	$CaCl_2$, NaCl, SO_4, Br solids, conductivity	Cl, SO_4, Br, non volatile	Deicing salts, atmos-pheric washout, vehicle
Organic matter	Vegetation, dust and dirt, humus, roadway accumulations, oil, fuels	Volatile fraction hexane extractables (oil and grease), BOD, COD, TOC	Vehicular airborne fallout, vegetation, vehicle, litter, aerosols
Nutrients	Nitrogen, phosphorus	TKN, NO_2, NO_3, PO_4	Fertilizer
Pathogenic Bacteria (indicators)	Coliforms	TC, FC, FS and other specific indicators	Soil, litter, excreta, bird droppings
Other	Asbestos, rubber, special compounds	Chemical diffraction and electron microscopy, special techniques	Vehicle, specific additives

CHAPTER 4

DOCUMENTATION

Chapter 4 Documentation
Table Of Contents

4.1 Overview

4.1.1 Introduction

An important facet during the design or analysis of any hydraulic facility is the documentation. Documentation of design as used in this chapter is the compilation and preservation of the design and related information on which the design and decisions were based. This includes drainage area and other maps, field survey information, source references, photographs, engineering calculations and analyses, measured and other data and flood history including narratives from newspapers and individuals such as highway maintenance personnel and local residents who witnessed or had knowledge of an unusual event.

Appropriate documentation of the design of any hydraulic facility is essential for:

- public information,
- rational evaluation of expenditure of public funds,
- reference by designers when improvements, changes, or rehabilitations are made to the facilities,
- adequate and efficient reviews, and
- development of defense in matters of litigation.

Frequently, it is necessary to refer to plans, specifications and analysis long after the actual construction has been completed. Documentation facilitates evaluation of the performance of structures after flood events. In the event of a failure, it is essential that contributing factors be identified in order that recurring damage can be avoided.

This chapter presents guidelines for collecting and organizing the information, results, and findings obtained regarding the design of hydraulic structures so as to provide as complete a history of the design process as is practical.

4.1.2 Goal

The major goal of providing good documentation is to record the design procedure that was used and show how the final design and decisions were arrived at. Often there is expressed the myth that avoiding documentation will prevent or limit litigation losses as it supposedly precludes providing the plaintiff with incriminating evidence. This is seldom if ever the case and documentation should be viewed as the record of reasonable and prudent design analysis based on the best available technology.

Thus, good documentation should provide the following:

- demonstrate that reasonable and prudent actions were taken;
- identifying the situation at the time of design;
- document that rationally accepted procedures and analysis were used which were commensurate with the perceived site importance and flood hazard;
- providing a continuous history to facilitate future changes;
- providing the file data necessary to quickly evaluate any future site problems that might occur during the facilities service life; and
- expediting project development by clearly providing the reasons and rationale for specific design decisions.

4.2 ADOT Practice

4.2.1 Introduction

Following are ADOT's practices related to documentation of hydrologic and hydraulic designs and analyses.

1. The amount of detail of documentation for each design or analysis shall be commensurate with the risk and the importance of the facility.
2. Documentation shall be organized to be as concise and complete as practicable so that knowledgeable designers can understand years hence what was done.
3. Documentation shall include all data and information used for project development. The documents should be in a format that clearly conveys the information without adding burden for storage and copying.
4. Documentation shall be organized into reports that logically lead the reader from past history through the problem background, into the findings and through the design process.
5. Report shall include an executive summary at the beginning to assist users in finding detailed information.

The drainage report shall include all related information and data, criteria, assumptions and judgments, identification of methods and computer programs, calculations, analyses, and results used in developing conclusions and recommendations related to drainage requirements. Discussions shall address inputs, design approach, results and conclusions. Identify published data, reports, memos, letters and interviews used. If circumstances are such that the drainage facility is sized by other than normal procedures or if the size of the facility is governed by factors other than hydrologic or hydraulic factors, a narrative summary detailing the design basis shall be included in the documentation file. Additionally, the designer shall include in the drainage report items not listed herein but which are useful in understanding the analysis, design, findings and final recommendations.

4.2.2 Documentation Process

The documentation of the design is an ongoing process and is part of each step in the hydrologic and hydraulic analysis and design process. This increases the accuracy of the documentation, provides data for future steps in the plan development process, and provides consistency in the design even when different designers are involved at different times of the plan development process. Documentation shall be provided whenever any information is gathered, analyzed, evaluated or used. This can occur during any of the pre-construction phases of the project life.

The designer is responsible for documenting what hydrologic analyses, hydraulic design, and related information is gathered and developed during the project development process. The documentation file shall contain design/analysis data and all information that influenced the design. A complete hydrologic and hydraulic design and analysis documentation file, where practicable, should include such items as:

- identification and location of the facility, including photographs (ground and aerial) and vicinity maps
- topographic and contour maps, with drainage areas delineated

4.2 ADOT Practice (continued)

4.2.2 Documentation Process (continued)

- history of performance of existing structure(s), including:
    - interviews (local residents, adjacent property owners and maintenance forces),
    - newspaper clippings,
    - judgments and assumptions,
    - design notes and correspondence relating to design decisions,
    - design computations,
    - engineering cost estimates.

4.2.3 Data Types

There are three basic types of data to be documented. The types are acquired, observed and calculated.

Acquired data:

- photographs,
- mapping,
- survey,
- flood insurance studies and maps by FEMA,
- soil information,
- flood studies,
- anecdotal reports by Department personnel, newspapers and abutting property owners, and
- as-built plans and subsurface borings

Observed data:

- field trip report(s) which may include:
    - video cassette recordings,
    - audio tape recordings,
    - still camera photographs,
    - movie camera films,
    - written analysis of findings with sketches.

Calculated Data:

- hydrology
- hydraulic performance
- plans

Appendix 4-A provides a checklist of data documentation to assist in determination of completeness.

4.3 Drainage Reports

4.3.1 General

Often, the design process requires two stages of development, a preliminary stage where approximations of size are made and evaluated, and a final stage where the design is completed and contract documents are prepared.

As stated in section 4.1 Introduction:

- Documentation shall be organized into reports that logically lead the reader from past history through the problem background, into the findings and through the design process.

- Documentation shall include all data and information used for project development.

- The amount of detail of documentation for each design or analysis shall be commensurate with the risk and the importance of the facility.

- Documentation shall be organized to be as concise and complete as practicable so that knowledgeable designers can understand years hence what was done.

- Report shall include an executive summary at the beginning to assist users in finding detailed information.

Circumstances may warrant or require special solutions that are not addressed by routine forms or formulas. In such cases the report should reference the formula used and their source. If necessary a typical calculation may be shown in detail to clarify the application of the formula and logic of the solution.

The results of the calculations are to be presented in tabular summary form. Summary forms should provide space for each of the critical variables used in the calculations.

Inclusion of calculations and computer data should be organized into appendices with a summary of results as the initial entry. This same summary will often be included in the body of the report. Appendix 4-B presents a listing of items which are to be included for the design of various hydraulic structures.

4.3.2 Preliminary Drainage Report

Preliminary hydraulic reports should be as complete as possible but must be tailored to satisfy the requirements of the project. The preliminary Drainage report includes the hydrology, evaluation of existing conditions, and the stormwater management plan. If there are any issues as to how to develop the final designs, they can be addressed at this time.

4.3.3 Final Drainage Report

The final Drainage report contains all the information developed for the project, including the information that was contained in the preliminary drainage report.

4.4 References

American Association of State Highway and Transportation Officials. Highway Drainage Guidelines. 1992

Appendix 4 - A

4A.1 Project Documentation Check List

REFERENCE DATA

Maps:
USGS Quad (Scale, Date)
USGS Other
ADOT
Local Zoning Maps
Flood Hazard Delineation (Quad.)
Floodplain Delineation (HUD)
Local Land Use
Soils Maps
Geologic Maps
Aerial Photos (Scale, Date)

Studies By External Agencies:
USGS Gages & Studies
USCE Floodplain Information Report
SCS Watershed Studies
Local Watershed Management Studies
US Forest Service Studies
Interim Floodplain Studies
Water Resource Data
Regional Planning Data
Utility Company Plans

High Water Elevations:
Survey
External Sources
Personal Reconnaissance

Flood History:
Newspaper Flood Reports
External Sources
Personal Reconnaissance
Maintenance Records
Photographs

Internal Reports:
Environmental Reports
Reconnaissance Report
Location Report
Drainage Survey Inspection Report
Hydraulic Design Report
Structure Inspection Report

Appendix 4 - A

4A.1 Project Documentation Check List (continued)

HYDROLOGY

Technical Resources:
Drainage Manual

Discharge Calculations:
Drainage Areas
Soils Parameters
Hydrograph Parameters
Rainfall Runoff Models:
- Rational Formula
- HEC-1

Regression Models:
- Regional Regression Equations
- Area-Discharge Curves
- Log-Pearson Type III

HYDRAULIC DESIGN

Technical Resources:
ADOT Hydraulic Manual
FHWA Manuals

Design Procedures:
Frequency and magnitude of discharges used.
Topography used for analysis.
Hydraulic performance of existing facility for design discharges.
Analysis of hydraulic performance of proposed facility for design discharges.

Design Appurtenances:
Dissipators
Riprap
Erosion & Sediment Control

Computer Programs:
USCE HEC-RAS Water Surface Profile
FHWA HY8 Culvert Design

Appendix 4 - B

4-B.1 Design Process Documentation File Contents

4-B.1.1 Introduction

The following design inputs and results shall be included in the documentation file. The intent is not to limit the data to only those items listed, but rather identify minimum requirements, as appropriate, consistent with the hydraulic design procedures as outlined in this manual. Inclusion of calculations and computer data should be organized into appendices with a summary of results as the initial entry. This same summary will often be included in the body of the report.

Circumstances may warrant or require special solutions that are not addressed by routine forms or formulas. In such cases the report should reference the formula used and their source. If necessary a typical calculation may be shown in detail to clarify the application of the formula and logic of the solution. The results of the calculations are to be presented in tabular summary form. Summary forms should provide space for each of the critical variables used in the calculations.

4-B.1.2 Hydrology

- contributing watershed area size and identification of source (map name, etc.);
- design frequencies and basis for selection;
- identification of design values, how determined, and discussion of any unusual variance from normal usage;
    - -soil characterization
    - -development characterization
    - -rainfall amount and distribution
    - -hydrograph parameters (time of concentration, storage coefficient)
- discharges for the design frequencies to be evaluated.

4-B.1.3 Bridges

- observed highwater, dates and discharges;
- potential flood hazards to adjacent properties;
- existing roadway geometry (plan and profile);
- proposed roadway geometry (plan and profile);
- cross section(s) used;
- roughness coefficient ("n" value) assignments;
- allowable headwater elevation and basis for its selection;
- identification of the method used for computation of water surface elevations;
- stage-discharge curve for existing and proposed conditions for the design frequencies to be evaluated.
- through-bridge and channel velocity estimates for the design frequencies to be evaluated.
- calculated backwater, velocity and scour for the design frequencies to be evaluated.
- magnitude and frequency of overtopping flood, if applicable;
- bridge scour results;
- copies of all computer analyses;
- economic analysis of design and alternatives;
- complete hydraulic study report;

Appendix 4-B

4-B.1 Design Process Documentation File Contents (continued)

4-B.1.4 Culverts

- observed highwater, dates and discharges;
- potential flood hazard to adjacent properties;
- existing roadway geometry (plan and profile);
- proposed roadway geometry (plan and profile);
- allowable headwater elevation and basis for its selection;
- cross section(s) used for the downstream channel tailwater elevations;
- roughness coefficient assignments ("n" values);
- culvert entrance type;
- stage discharge information for existing and proposed conditions for the design frequencies to be evaluated;
- outlet velocity predictions for the design frequencies to be evaluated;
- predicted scour for the design frequencies to be evaluated;
- culvert outlet appurtenances and energy dissipation calculations and designs;
- copies of all computer analyses.

4-B.1.5 Open Channels

- observed highwater, dates and discharges;
- cross section(s) used in the design water surface determinations and their locations;
- roughness coefficient assignments ("n" values), existing and proposed conditions;
- identification of the method used for computation of water surface elevations;
- channel velocity and locations determinations;
- stage discharge curves for the design frequencies to be evaluated;
- water surface profiles through the reach for the design frequencies to be evaluated;
- design or analysis of materials proposed for the channel bed and banks for the design frequencies to be evaluated;
- energy dissipation calculations and designs for the design frequencies to be evaluated, and
- copies of all computer analyses.

4-B.1.6 Storm Drains

- complete drainage area map;
- design frequency;
- information concerning outfalls and existing storm drains;
- information concerning utilities and other design considerations;
- schematic of storm drain system layout;
- computations for inlets and pipes, including hydraulic grade lines.

Appendix 4 – B

4-B.1 Design Process Documentation File Contents (continued)

4-B.1.7 Pump Stations

- maximum allowable headwater elevations and related probable damage;
- inflow design hydrograph from drainage area to pump;
- sump dimensions;
- available storage amounts;
- pump sizes and operations;
- starting sequence and elevations;
- pump calculations and design report;
- line storage and pit storage capacity;
- flood frequency curve for the attenuated peak discharge.

4-B.1.8 Detention/Retention Basins

- maximum allowable headwater elevations and related probable damage;
- inflow design hydrograph from drainage area to basin;
- basin dimensions;
- available storage amounts;
- Stage-storage curve;
- Stage-discharge curve;
- Inflow hydrograph;
- Outflow hydrograph.

CHAPTER 5

DATA COLLECTION

Chapter 5 Data Collection
Table Of Contents

5.1 Overview

5.1.1 Introduction

A well-planned data collection program leads to a more orderly and effective analysis and design process. It is appropriate and efficient to identify the types of data that will be required prior to beginning the engineering analysis. The effort necessary for data collection and compilation shall be tailored to the importance of the project. **Not all of the data discussed in this chapter will be needed for every project.**

Data collection for a specific project must be tailored to:

- scope of the engineering analysis,
- project cost,
- complexity of site conditions and hydraulics,
- social, economic environmental and archaeological requirements,
- regulatory requirements, and
- unique project requirements

5.1.2 Data Requirements

In this chapter is an outline of the data types that are normally required for drainage analysis and design, possible sources and other aspects of data collection. The following subjects are presented in this chapter:

- Types and Sources of Data
- Topographic Survey Information
- Field Reviews
- Data Evaluation

Uniform or standardized survey requirements for all projects may prove uneconomical or data deficient for a specific project. Special instructions outlining data requirements may have to be provided to the survey party by the designer for unique sites.

5.1.3 Survey Methods/Computation Accuracy

The publication "Accuracy of Computed Water Surface Profiles," U.S. Army Corps of Engineers, Dec. 1986, focuses on determining relationships between:

- survey technology and accuracy employed for determining stream cross sectional geometry,
- degree of confidence in selecting Manning's roughness coefficients, and
- the resulting accuracy of hydraulic computations.

The publication also presents methods for determining the upstream and downstream limits of data collection for a hydraulic study requiring a specified degree of accuracy.

5.1 Overview (continued)

5.1.3 Survey Methods/Computation Accuracy (continued)

Computer software has been developed to perform the calculations for the various routines presented in this manual. HY-11, Survey Accuracy, is available from the McTrans Center, University of Florida, Gainesville, FL

5.2 Types Of Data Needed

5.2.1 General

The designer must compile the data that are specific to the subject site. Following are the major types of data that may be required:

- existing and proposed land use data in the subject drainage area and in the general vicinity of the facility;
- anticipated changes in land use and/or watershed characteristics;
- floodplain, environmental regulations, and archaeological data;
- watershed characteristics;
- stream reach data (especially in the vicinity of the facility);
- hydrologic and meteorologic data (stream flow and rainfall data related to maximum or historical peak as well as low-flow discharges and hydrographs applicable to the site);
- permit requirements, and;
- other physical data in the general vicinity of the facility such as utilities, easements, etc.

Watershed, stream reach and site characteristic data, as well as data on other physical characteristics can be obtained from a field reconnaissance of the site. Examination of available maps and aerial photographs of the watershed is also an excellent means of defining physical characteristics of the watershed.

Following is a brief description of the major data topics that relate to drainage facility analysis and design. Additional discussion is contained in the AASHTO Highway Drainage Guidelines, Volume II.

5.2 Types Of Data Needed (continued)

5.2.2 Drainage Surveys

The designer should inspect the site and its contributing watershed to determine the required field and/or aerial drainage survey to be undertaken as part of the hydraulic analysis and design. Survey requirements for small drainage facilities such as small culverts are less extensive than those for major facilities such as bridges. However, the purpose of each survey is to provide an accurate picture of the conditions within the zone of hydraulic influence of the facility.

Appendix A contains instructions for typical minor and major drainage surveys.

Following are the data that can be obtained or verified:
- contributing drainage area characteristics;
- stream reach data - cross sections and thalweg profile;
- existing structures;
- location and survey for development, existing structures, etc., that may affect the determination of allowable flood levels, capacity of proposed drainage facilities, or acceptable outlet velocities;
- drift/debris characteristics;
- general ecological information about the drainage area and adjacent lands; and
- high water elevations including the date of occurrence.

Much of these data must be obtained from an on-site inspection. It is often much easier to interpret published sources of data after an on-site inspection. Only after a thorough study of the area and a complete collection of all required information should the designer proceed with the final design of the hydraulic facility. All pertinent data and facts gathered through the survey shall be documented as explained in Chapter 4, Documentation.

5.2.3 Watershed Characteristics

Contributing Size - The size of the contributing drainage area expressed in acres or square miles, is determined from some or all of the following:

- Direct field surveys with conventional surveying instruments.
- Use of USGS or project topographic maps, aerial photographs, digital elevation models, together with field checks to determine any changes in the contributing drainage area such as:
  - terraces,
  - lakes, sinks,
  - debris or mud flow barriers,
  - reclamation/flood control structures,
  - irrigation diversions, and
  - storm drainage systems.

5.2 Types Of Data Needed (continued)

5.2.3 Watershed Characteristics (continued)

In determining the size of the contributing drainage area, any subterranean flow or any areas outside the physical boundaries of the drainage area that have runoff diverted into the drainage area being analyzed shall be included in the total contributing drainage area. In addition, it must be determined if floodwaters are diverted out of the basin before reaching the site.

Slopes - The slope of the stream and the average slope of the watershed (basin slope) should be determined. Hydrologic and hydraulic procedures in other chapters of this manual are dependent on watershed slopes as well as other factors.

Watershed Land Use

- Define and document the present and expected future land use, particularly the location, degree of anticipated urbanization and data source.
- Information on existing use and future urbanization trends may be obtained from:
  - field review
  - aerial photographs (conventional and infrared),
  - zoning maps and master plans,
  - USGS and other maps,
  - municipal planning agencies, and
  - landsat (satellite) images.

Specific information about particular tracts of land can often be obtained from owners, developers, realtors and local residents. Care should be exercised in using data from these sources since their reliability may be questionable and these sources may not be aware of future development within the watershed that might affect specific land uses.

Existing land use data for small watersheds can best be determined or verified from a field survey. Field surveys shall also be used to update information on maps and aerial photographs, especially in basins that have experienced changes in development since the maps or photos were prepared. Infrared aerial photographs may be particularly useful in identifying types of urbanization at a point in time.

Streams, Rivers, Ponds, Lakes, Wetlands and Detention Basins

At all streams, rivers, ponds, lakes and wetlands that will affect or may be affected by the proposed structure or construction, the following data shall be secured. These data are essential in determining the expected hydrology and may be needed for regulatory permits.

- Outline of the boundary (perimeter) of the water body for the ordinary highwater.
- Elevation of normal as well as high water for various frequencies.
- Detailed description of any natural or manmade spillway or outlet works including dimensions, elevations and operational characteristics.

5.2 Types Of Data Needed (continued)

5.2.3 Watershed Characteristics (continued)

Streams, Rivers, Ponds, Lakes, Wetlands and Detention Basins (continued)

- Detailed description of any emergency spillway works including dimensions and elevations.
- Description of adjustable gates, soil and water control devices.
- Profile along the top of any dam and a typical cross section of the dam.
- Use of the water resource (stock water, fish, recreation, power, irrigation, municipal or industrial water supply, etc.).
- Existing conditions of the stream, river, pond, lake or wetlands as to turbidity and silt.
- Riparian ownership(s) as well as any water rights.

Environmental Considerations

The need for environmental data in the engineering analysis and design stems from the need to investigate and mitigate possible impacts due to specific design configurations. Environmental data needs may be summarized as follows:

- Information necessary to define the environmental sensitivity of the facility's site relative to impacted surface waters, e.g., water use, water quality and standards, aquatic and riparian wildlife biology and wetlands information.
- Information necessary to determine environmentally compatible designs, e.g., circulation patterns and sediment transport data.
- Data needs at wetlands are unique and can be identified through coordination with AZ Game and Fish, U.S. Fish and Wildlife Service, etc.

5.2.4 Site Characteristics

A complete understanding of the physical nature of the natural channel or stream reach is of prime importance to a good hydraulic design — particularly at the site of interest. Any work being performed, proposed or completed, that changes the hydraulic efficiency of a stream reach must be studied to determine its effect on the stream flow. The designer should be aware of plans for channel modifications, and any other changes that might affect the facility design.

The stream may be classified as:

- rural or urban,
- improved or unimproved,
- narrow or wide,
- rapid or sluggish flow,
- stable, transitional, or unstable,
- sinuous, straight, braided, alluvial, or incised, and
- perennial or intermittent flow.

5.2 Types Of Data Needed (continued)

5.2.4 Site Characteristics (continued)

Geomorphological data are important in the analysis of channel stability and scour. Types of needed data are:

- sediment transport and related data,
- stability of form over time (braided, meandering, etc.),
- scour history/evidence of scour, and
- bed and bank material identification

Chapter 7, Channels presents additional information regarding the site characteristics that should be considered in the analysis of channels.

Roughness Coefficients

Roughness coefficients, ordinarily in the form of Manning's n values shall be estimated for the entire flood limits of the stream. A tabulation of Manning's n values with descriptions of their applications can be found in Chapter 7, Channels.

Stream Profile

Stream bed profile data shall be obtained and these data should extend sufficiently upstream and downstream to determine the average slope and to encompass any proposed construction or aberrations. Identification of "headcuts" which could migrate to the site under consideration are particularly important. If the stream has flow, the profile data of the water surface shall also be obtained. If there is a stream gage relatively close, the discharge, date and hour of the reading shall be obtained.

Stream Cross Sections

Stream cross section data shall be obtained that represent the conditions at the structure site. Stream cross section data should also be obtained at other locations where stage-discharge and related calculations will be necessary.

Existing Structures

The location, size, description, condition, observed flood stages and channel section relative to existing structures on the stream reach and near the site shall be secured in order to determine their capacity and effect on the stream flow. Any structures, downstream or upstream, which may cause backwater or retard stream flow shall be investigated. Also, the manner in which existing structures have been functioning with regard to such things as scour, overtopping, debris and ice passage, fish passage, etc. shall be noted. With bridges, these data shall include span lengths, type of piers and substructure orientation which usually can be obtained from existing structure plans. The necessary culvert data includes size, inlet and outlet geometry, slope, end treatment, culvert material and flow line profile. "As built" highway construction plans may be available to obtain required bridge and/or culvert data. Photographs and high water profiles or marks of flood events at the structure and past flood scour data can be valuable in assessing the hydraulic performance of the existing facility.

Controlling Flood Levels

Development and property use adjacent to the proposed site, both upstream and downstream, may determine controlling flood levels. Floor elevations of structures or fixtures shall be noted. In the absence of upstream development, the presence of downstream development may determine appropriate overflow points when an overtopping design of the highway is considered.

5.2 Types Of Data Needed (continued)

5.2.4 Site Characteristics (continued)

Flood History

The history of past floods and their effect on existing structures are of exceptional value in making flood hazard evaluation studies, as well as needed information for sizing structures. Information may be obtained from newspaper accounts, local residents, flood marks or other positive evidence of the height of historical floods. Changes in channel and watershed conditions since the occurrence of the flood shall be evaluated in relating historical floods to present conditions.

Recorded flood data are available from agencies such as:

- U.S. Army Corps of Engineers,
- U.S. Geological Survey,
- U.S. Soil Conservation Service,
- Federal Emergency Management Agency,
- Bureau of Reclamation,
- Local Flood Control Districts, and
- AZ Department of Water Resources.

Anecdotal information may be available from ADOT maintenance personnel or local or State public safety personnel.

Scour Potential

Scour potential is an important consideration relative to the stability of the structure over time. Scour potential will be determined by a combination of the stability of the natural materials at the facility site, tractive shear force exerted by the stream and sediment transport characteristics of the stream. Data on natural materials can be obtained by tests of the site materials.

Bed and bank material samples sufficient for classifying channel type, stability and gradations, as well as a geotech study to determine the substrata if scour studies are needed, will be required. The various alluvial river computer model data needs will help clarify what data are needed. Also, these data are needed to determine the presence of bed forms so a reliable Manning's roughness coefficient as well as bed form scour can be estimated.

Controls Affecting Design Criteria

Many controls will affect the criteria applied to the final design of drainage structures including allowable headwater level, allowable flood level, allowable velocities, and resulting scour and other site specific considerations. Data and information related to such controls can be obtained from Federal, State and local regulatory agencies and site investigations to determine what natural or man-made controls shall be considered in the design.

5.2 Types Of Data Needed (continued)

5.2.4 Site Characteristics (continued)

Controls Affecting Design Criteria (continued)

The redirection of floodwaters can significantly affect the hydraulic performance of a site. Some actions that redirect flows are irrigation facilities, debris jams, mud flows and highways or railroads. In addition, there may be downstream and upstream controls which shall be documented.

Downstream Control - Any ponds or reservoirs, along with their spillway elevations and design levels of operation, shall be noted as their effect on backwater and/or streambed aggradation may directly influence the proposed structure. Also, any downstream confluence of two or more streams shall be studied to determine the effects of backwater or streambed change resulting from that confluence. Gravel mining operations shall be noted.

Upstream Control - Upstream control of runoff in the watershed shall be noted. Conservation and/or flood control reservoirs in the watershed may effectively reduce peak discharges at the site and may also retain some of the watershed runoff. Capacities and operation designs for these features shall be obtained. The reservoir sponsors often have complete reports concerning the operation and design of proposed or existing conservation and/or flood control reservoirs. Gravel mining operations shall be noted.

5.3 Sources of Data

5.3.1 Sources

Much of the data and information necessary for the design of highway drainage facilities may be obtained from some combination of the sources listed below:

The major and most common data sources are

Meteorological Data:	National Weather Service
Watershed Characteristics;	
Size, slope, watercourses:	USGS maps
Soil:	National Resource Council, Soil Conservation Service Soil Maps and Studies
Floodplain Studies:	Arizona Department of Water Resources Local Flood Control Districts
Environmental Data:	Arizona Game and Fish Department US Forest Service US Fish and Wildlife US Bureau of Land Management

5.3 Sources of Data (continued)

5.3.2 Geographic Information Systems (GIS)

Geographic information systems (GIS) may be used as a source of georeferenced hydrologic data required in hydraulic design decision making. For example, Maricopa County has developed a GIS database containing the land cover, soil type and topography. This database may then be used to produce the existing and ultimate development hydrographs as well as an array of maps, graphs and tables needed to complete the hydrologic analysis.

5.3.3 National Flood Insurance Program

Many streams have been analyzed for local flood insurance studies. In these cases, data collection is normally unnecessary since the discharges and hydraulic models are normally available from the Federal Emergency Management Agency. Even though these studies are a good source of data, their technical content should be reviewed prior to using the data. Many of the studies are outdated and/or will not reflect changes that may have occurred in the study reach since its initial publication.

5.4 Survey Information

5.4.1 General

Complete and accurate survey information is necessary to develop a design that will best serve the requirements of a site. The amount of survey data gathered shall be commensurate with the importance and cost of the proposed structure and the expected flood hazard. Data collection shall be as complete as possible during the initial survey in order to avoid repeat visits. Thus, data needs must be identified and tailored to satisfy the requirements of the specific location and size of the project early in the project design phase.

At many sites photogrammetry is an excellent method of securing the topographical components of drainage surveys. Planimetric and topographic data covering a wide area are easily and cost effectively obtained in many geographic areas. A supplemental field survey is required to provide data in areas obscured on the aerial photos (underwater, heavy vegetation, etc.).

5.4.2 ADOT Requirements

Instructions for hydraulic surveys are contained in the ADOT survey manual. An outline of these requirements is presented in Appendix A of this chapter. Example of forms and check lists are provided in Appendix B.

5.5 Field Reviews

5.5.1 On Site Inspection

The designer shall make field reviews in order to become familiar with the site. The most complete survey data cannot adequately depict all site conditions or substitute for personal inspection by someone experienced in drainage design. Factors that most often need to be confirmed by field inspection are:

- selection of roughness coefficients,
- evaluation of apparent flow direction and diversions,
- flow concentration,
- observation of land use and related flood hazards,
- geomorphic relationships,
- highwater marks or profiles and related frequencies,
- existing structure size and type, and
- existence of wetlands.

The field visit shall be made before final hydraulic design is undertaken. There are several criteria that shall be established before making the field visit. Can any needed information be obtained from maps, aerial photos, or by telephone calls? What kind of equipment should be taken, and most important, what exactly are the critical items at this site? Photographs shall be taken. As a minimum, photos shall be taken looking upstream and downstream from the site as well as along the contemplated highway centerline in both directions. Details of the streambed and banks should also be photographed along with structures in the vicinity both upstream and downstream. Close up photographs complete with a scale or grid shall be taken to facilitate estimates of the streambed gradation.

5.5.2 Check List

Sample forms for use in identifying and cataloging field information are shown in Appendix C.

5.6 Data Evaluation

5.6.1 Objective

Once the needed data have been collected, the next step is to compile it into a usable format. The designer must ascertain whether the data contains inconsistencies or other unexplained anomalies which might lead to erroneous calculations or results. The main reason for analyzing the data is to draw all of the various pieces of collected information together and to fit them into a comprehensive and accurate representation of the hydrologic and hydraulic characteristics of a particular site.

5.6.2 Evaluation

Experience, knowledge and judgment are important parts of data evaluation. It is in this phase that reliable data shall be separated from that which is less reliable and historical data combined with that obtained from measurements. The data shall be evaluated for consistency and to identify any changes from established patterns. Reviews shall be made of such things as previous studies, old plans, etc., for types and sources of

5.6 Data Evaluation (continued)

5.6.2 Evaluation (continued)

data, how the data were used and any indications of accuracy and reliability. Historical data shall be reviewed to determine whether significant changes have occurred in the watershed and whether these data can be used. Data should always be subjected to careful study by the designer for accuracy, reliability and compatibility with the intended use.

Basic data, such as streamflow data derived from non-published sources, shall be evaluated and summarized before use. Maps, aerial photographs, Landsat images and land use studies shall be compared with one another and with the results of the field survey and any inconsistencies resolved. General references shall be consulted to help define the hydrologic character of the site or region under study and to aid in the analysis and evaluation of data.

5.6.3 Sensitivity

Often sensitivity studies can be used to evaluate data and the importance of specific data items to the final design. Sensitivity studies consist of conducting a design with a range of values for specific data items. The effect on the final design can then be established. This is useful in determining what specific data items have major effects on the final design and the importance of possible data errors. Time and effort shall then be spent on the more sensitive data items making sure these data are as accurate as possible. This does not mean that inaccurate data are accepted for less sensitive data items, but it allows prioritization of the data collection process given a limited dollar and time allocation.

The results of this type of data evaluation shall be used so that as reliable a description as possible of the site can be made within the allotted time and the resources committed to this effort. The effort of data collection and evaluation shall be commensurate with the importance and extent of the project and/or facility.

5.7 References

FHWA. Hydraulic Design Series HDS-2, Hydrology. 1996.
Geographic Information Systems: An Introduction (Star and Estes, 1990),
Geographic Information Systems: A Management Perspective (Aronoff, 1990),
Geographic Information Systems — A Guide to the Technology (Antenucci, et. al., 1992)

Appendix A - Field Visit Documentation Forms

Form 1
FIELD VISIT DOCUMENTATION FORM

DATE:___________________________________
PROJECT:______________________________
BY:_______________________________________

STRUCTURE TYPE

SIZE OR SPAN
 # OF BARRELS OR SPANS

CLEAR HT

ABUT TYPES

INLET TYPE
EXISTING WTWY COVER
OVERFLOW BEGINS @ EL.

MAX AHW
REASON:

UP OR DOWNSTREAM RESTRICTION:

OUTLET CHANNEL, BASE
MANNING'S n VALUE:
TYPE OF MATERIAL IN STREAM

PIERS: TYPE

SKEW
INLET
OUTLET
% GRADE OF ROAD
% GRADE OF STREAM
LENGTH OF OVERFLOW
CHECK FOR DEBRIS

SIDE SLOPES
HEIGHT OF BANKS

PONDING

CHECK BRIDGES UPSTREAM AND DOWNSTREAM

CHECK LAND USE UPSTREAM AND DOWNSTREAM

SURVEY REQUIRED? YES___ NO

REMARKS:

Appendix A - Field Visit Documentation Forms

Form 2
HYDRAULIC FIELD VISIT DOCUMENTATION LIST

I. GENERAL PROJECT DATA

1. Project Number:_________________ 2. County:
3. Road Name:
4. Site Name:____________________, Station _______________ M.P.
5. Site Description: ( ) Cross drain, ( ) Irrigation, ( ) Storm Drain, ( ) Long. Encroach, ( ) Ch. Change, ( ) Other
6. Survey Source: ( ) Field, ( ) Aerial, ( ) Other
7. Date Survey Received:_____________, from________________________(name)
8. Site Inspected by______________________________ on _____________
(name) (date)

II. OFFICE PREPARATION FOR INSPECTION

1. Reviewed:
   Aerial Photos - ( ) Yes, Photo #'s ___________, ( ) None Available
   Mapping/Maps - ( ) Yes, Map #'s ____________, ( ) None Available
   Reports - ( ) Yes, ( ) No, ( ) None Available at this time
   Permanent File - ( ) Yes, ( ) No, ( ) No file data found

2. Special Requirements and Problems Identified for Field Checking:

( ) Hydrologic Boundary - obtain hydrologic channel geometry
( ) Adverse Flood History - obtain HW Marks/dates/eye witness
( ) Irrigation Ditch - obtain several Water Right depths
( ) Permits Req'd - ( ) COE, ( ) Dam, ( ) Coast Guard, ( ) FEMA
( ) Other
( ) Adverse Channel Stability and Alignment History - Check for headcutting, bank caving, braiding, increased meander activity
( ) Structure Scour - check flow alignment, scour at culvert outlet or evidence of bridge scour
( ) Obtain bed/bank material samples at

III. FIELD INSPECTION
(The following details obtained at the site are annotated on the Drainage Survey)
1. Survey appears correct: - ( ) Yes, ( ) Apparent errors are:
____ which were resolved by:____

2. Flooding Apparent? - ( ) No, ( ) Yes, HW marks obtained, ( ) Yes but HW marks not obtained because
____

Appendix A - Field Visit Documentation Forms

Form 2 (Continued)
HYDRAULIC FIELD VISIT DOCUMENTATION LIST (continued)

3. Do all Floods Reach Site? - ( ) Yes, ( ) No and details obtained, ( ) No but details not obtained because ____

4. Do Floodwaters Enter Irrigation Ditch ? - ( ) N/A, ( ) No, ( ) Yes and details obtained, ( ) Yes but details not obtained because____ ____

5. Hydrologic Channel Geometry obtained? - ( ) Yes, ( ) No because ___________________________ ____

6. Channel Unstable? - ( ) No , ( ) Yes because of ( ) headcutting observed and ( ) amount/location obtained, ( ) bank caving, ( ) braiding, ( ) increased meander activity, ( ) Other ___________________ ____

7. Structure Scour in Evidence? - ( ) No, ( ) Minor, ( ) Yes and ( ) obtained bed/bank samples and ( ) noted any flow alignment problems, ( )Yes and ( ) bed/bank material samples not obtained and ( ) flow alignment not noted because ____ ____ ____

8. Irrigation facility? - ( ) No, ( ) Yes and several water right related depths obtained, ( ) Yes and No water right related depths obtained because____ ____

9. Manning's n obtained? - ( ) Yes, ( ) No because____ ____

10. Property damage due to backwater? - ( ) No ( ) Yes and elevation/property type checked, ( ) Yes but elevation/property type not obtained because____ ____

11. Environmental Hazards Present? - ( ) No, ( ) Yes, details obtained, ( ) Yes, details not obtained because ____ ____

12. Ground Photos Taken? - ( ) Upstream floodplain and all property, ( ) Downstream floodplain and all property, ( ) Site looking from downstream, ( ) Site looking from upstream, ( ) Evidence of channel instability, ( ) Evidence of scour, ( ) Channel Material w/scale, ( ) Existing structure inlet/outlet, ( ) Other ____

13. Effective drainage area visually verified? ( ) Yes, ( ) No because_____________________________ ____

Appendix B - Sources Of Data

Principal Hydrologic Data Sources

•Meteorological Data
  National Oceanic and Atmospheric Agency (NOAA)
  National Climatic Data Center
  37 Battery Park Avenue
  Federal Building
  Asheville, North Carolina 28801
  (704) 271-4800 FAX (704) 271-4876
• Regional and local flood studies
• U.S. Geological Survey regional and any site studies
• Surveyed high water marks and site visits
• Hydrology data from others:
  Local Flood Control Districts

Principal Watershed Data Sources

• U.S. Geological Survey maps ("Quad" sheets)
  U.S. Geological Survey
  Rocky Mountain Mapping Center
  Mail Stop 504
  Denver Federal Center
  Denver, Colorado 80225
  (303) 236-5829
• EROS aerial photographs
  U.S. Geological Survey
  EROS Data Center
  Sioux Falls, South Dakota 57198
  (605) 594-6151
• U.S. Geological Survey
• State and local maps and aerial photos
• State geological maps
• Soil Conservation Service and BLM Soils Maps
• County Soils Maps

Principal Site Data Sources

• ADOT files for existing facilities
• Field or aerial surveys from others

Appendix B - Sources Of Data (continued)

Principal Regulatory Data Sources

• Federal floodplain delineations and studies

Federal Emergency Management Agency
Flood Map Distribution Center
6930 (A-F) San Tomas Road
Baltimore, Maryland 21227-6227
(800) 358-9616

• State floodplain delineations and studies from Local Flood Control Districts and AZ Dept. of Water Resources.

Principal Environmental Data Sources

• U.S. Environmental Protection Agency data and studies
• U.S. Corps of Engineers data and studies
• U.S. Geological Survey water quality data
• Arizona Department of Water Resources water quality data
• Environmental statements prepared by other Federal, State and local agencies as well as private parties

Principal Demographic, Economic and Political Data Sources

• Agency files for existing facilities,
• Agency plans for proposed facilities,
• Agency field or aerial surveys,
• Agency planning, budgeting and scope documents,
• Agency reports, memorandums, minutes and verbal communications.

Other Data Sources

For Federal agencies, the designer should begin contact with the local office.

• U.S. Bureau of Reclamation (USBR)
• Central Arizona Project (CAP)
• U.S. Bureau of Land Management (BLM)
• U.S. Environmental Protection Agency (EPA)
• U.S. Federal Emergency Management Agency (FEMA)
• U.S. Fish and Wildlife Service (USFWS)
• U.S. Forest Service (USFS)
• U.S. Soil Conservation Service (SCS)
• U.S. Corps of Engineers (COE)
• U.S. Geological Survey (USGS)
• Federal Highway Administration (FHWA)
• National Weather Service (NWS)

Appendix B - Sources Of Data (continued)

Other Data Sources (continued)

- National Oceanic and Atmospheric Administration (NOAA)
- Indian Natural Resource Agencies
- Salt River Project (SRP)
- Arizona Department of Environmental Quality (ADEQ)
- Arizona Department of Water Resources (ADWR)
- Arizona State Land Department (ASLD)
- Local irrigation, drainage, flood control and watershed districts
- Municipal governments
- Private citizens
- Private industry

CHAPTER 6

EROSION AND SEDIMENT CONTROL

Chapter 6 Erosion and Sediment Control
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6.1 Erosion and Sediment Control Guidelines

6.1.1 Background

Erosion and sedimentation are natural, geologic processes whereby soil materials are detached and transported from one location and deposited in another, primarily due to rainfall and runoff. The construction and operation of highway facilities can result in accelerated erosion and sedimentation. This accelerated process can result in significant impacts such as expensive maintenance problems, unsightly conditions, instability of slopes and disruption of ecosystems. For this reason, the design process must give consideration to minimization of erosion and sedimentation.

6.1.2 Federal Policy

As a result of the National Environmental Policy Act of 1969, much attention has been directed to the control of erosion and sedimentation: numerous State and Federal regulations and controls governing land disturbing activities have been developed and published. There are Federal control requirements exerted by numerous agencies (Corps of Engineers, Environmental Protection Agency, Fish and Wildlife Service, etc.) through their administration of various permitting requirements (Section 404, Section 402 and the NPDES Program of the Federal Water Pollution Control Act (FWPCA), and Section 9 and 10 of the River and Harbor Act).

6.1.3 AASHTO Policy

The American Association of State Highway Officials' policy for erosion and sediment control is stated in the publication, "A Policy on Geometric Design of Highways and Streets," as follows.

"Erosion prevention is one of the major factors in design, construction and maintenance of highways. It should be considered early in the location and design stages. Some degree of erosion control can be incorporated into the geometric design, particularly in the cross section elements. Of course, the most direct application of erosion control occurs in drainage design and in the writing of specifications for landscaping and slope planting.

Erosion and maintenance are minimized largely by the use of flat side slopes, rounded and blended with natural terrain; serrated cut slopes; drainage channels designed with due regard to width, depth, slopes, alignment and protective treatment; inlets located and spaced with erosion control in mind; prevention of erosion at culvert outlets; proper facilities for ground water interception; dikes, berms and other protective devices; sedimentation devices to trap sediment at strategic locations; and protective ground covers and planting."

Although some standardization of methods for minimizing soil erosion in highway construction is possible, guidelines for erosion control are of a general nature because of the wide variation in climate, topography, geology, soils, vegetation, water resources and land use encountered in different projects throughout the State.

6.1 Erosion and Sediment Control Guidelines

6.1.4 ADOT Policy

Since highway construction may involve the disturbance of large land areas, control of erosion and sedimentation is a major concern. The following reference is to be consulted for application of the principles and goals presented herein. Erosion and sediment control practices can be subdivided onto temporary and permanent. Temporary controls are those that are used during the construction phase and are presented in the following document:

ADOT Erosion and Pollution Control Manual for Highway Design and Construction

Some permanent controls are presented in Section 6.4.

6.2 Planning for Permanent Erosion And Sediment Control

6.2.1 Introduction

For a permanent erosion and sediment control program to be effective, it must be considered and measures taken in the project planning stage. These planned measures, when conscientiously constructed, will result in orderly development without environmental degradation. The design of erosion and sediment control systems involves the application of common sense planning and design of actions that will minimize the adverse impacts of soil erosion, transport and deposition.

6.2.2 Guidelines

The following basic guidelines govern the development and implementation of a sound permanent erosion and sediment control plan.

- The elements should be planned to take advantage of the topography, soils, waterways and natural vegetation at the site.
- Onsite erosion control measures should be applied to reduce the gross erosion from the site.

These guidelines should be tied together in the planning process, which identifies potential erosion and sediment control problems and control measures before construction begins.

6.2.3 Sediment Control Measures

Control measures are actions that either retard erosion or remove sediment from runoff. Measures to retard erosion include stabilizing surface treatments, vegetation retention, sodding, mulching, and seeding. Measures for removal of sediment include filtration buffers and sediment traps.

6.3 Factors Influencing Erosion

6.3.1 Principal Factors

The inherent erosion potential of any area is determined by four principal factors: soil characteristics, vegetative cover, topography, and climate. Although each of these factors is discussed separately herein, they are interrelated in determining erosion potential.

6.3.2 Soil Characteristics

The properties of soil which influence erosion by rainfall and runoff are ones which affect the infiltration capacity of a soil and those which affect the resistance of a soil to detachment and being carried away by falling or flowing water. Soils containing high percentages of fine sands and silt are normally the most erodible. As the clay and organic matter content of these soils increases, the erodibility decreases. Clays act as a binder to soil particles, thus reducing erodibility. However, while clays have a tendency to resist erosion, once eroded they are easily transported by water. Soils high in organic matter have a more stable structure that improves their permeability. Such soils resist raindrop detachment and infiltrate more rainwater. Clear, well-drained and well-graded gravels and gravel-sand mixtures are usually the least erodible soils. Soils with high infiltration rates and permeabilities reduce the amount of runoff.

6.3.3 Vegetative Cover

Vegetative cover plays an important role in controlling erosion in the following ways:

- shields the soil surface from the impact of falling rain,
- holds soil particles in place,
- maintains the soil's capacity to absorb water,
- slows the velocity of runoff, and
- removes subsurface water between rainfalls through the process of evapotranspiration.

Limiting and staging the removal of existing vegetation and decreasing the area and duration of exposure can significantly reduce soil erosion and sedimentation. Special consideration should be given to the maintenance of existing vegetative cover on areas of high erosion potential such as erodible soils, steep slopes, drainageways and the banks of streams.

6.3.4 Topography

The size, shape and slope characteristics of a watershed influence the amount and rate of runoff. As both slope length and gradient increase, the rate of runoff increases and the potential for erosion is magnified. Slope orientation can also be a factor in determining erosion potential.

6.3 Factors Influencing Erosion (continued)

6.3.5 Climate

The frequency, intensity and duration of rainfall are fundamental factors in determining the amounts of runoff produced in a given area. As both the volume and velocity of runoff increase, the capacity of runoff to detach and transport soil particles also increases. Where storms are frequent, intense or of long duration, erosion risks are high. Seasonal changes in temperature, as well as variations in rainfall, help to define the high erosion risk period of the year. When precipitation falls as snow, no erosion will take place. However, in the spring the melting snow adds to the runoff and erosion hazards are high. Because the ground is still partially frozen, its absorptive capacity is reduced. Frozen soils are relatively erosion-resistant. However, soils with high moisture content are subject to uplift by freezing action, and are usually very easily eroded upon thawing.

6.4 Control Measures And Practices

6.4.1 Introduction

Following is a discussion of the commonly used permanent erosion and sediment control practices with comments regarding application. The **ADOT Erosion and Pollution Control Manual for Highway Design and Construction** shall be used for design detailing and construction guidelines regarding temporary control measures.

6.4.2 Channel Lining

One means of reducing erosion during highway construction and operation is through the use of properly designed linings in drainage channels. Linings may be rigid, such as Portland cement or asphaltic concrete, or flexible, such as vegetation or rock riprap. Flexible linings of erosion resistant vegetation and rock riprap should be used whenever feasible. When vegetation is chosen as the permanent channel lining, it may be established by seeding or sodding. Installation by seeding usually requires protection by one of a variety of temporary lining materials until the vegetation becomes established.

Use Consideration

Flexible linings are generally less expensive to install than rigid linings. They permit infiltration and exfiltration, have a natural appearance, especially after vegetation is established, and provide a filtering media for runoff contaminants. Vegetative and rock riprap liners provide less improvement in conveyance over natural conditions and the resultant acceleration of flow volume and peak is less than with rigid linings.

Flexible linings do have the disadvantage of being limited in the depth of flow that they can accommodate without erosion occurring. As a result, the channel may provide a low capacity for a given cross-sectional area when compared to a rigid lining. Also limited right-of-way, unavailability of rock, or the inability to establish vegetation may preclude the use of flexible linings. In these instances, rigid linings may be the only alternative.

6.4 Control Measures And Practices (continued)

Design Detailing

Rigid Channel Linings - For rigid channel linings, such as concrete or soil cement, there is no maximum permissible depth for the flow velocities normally encountered in highway drainage work, since no erosion can occur. Thus, the maximum flow depth is based only on the freeboard requirement for the channel. See the Bank Protection Chapter for more design detailing related to rigid channel linings

6.4.3 Outlet Protection

The outlets of pipes and structurally lined channels are points of critical erosion potential. To prevent scour at stormwater outlets, a flow transition structure is needed which will absorb the initial impact of the flow and reduce the flow velocity to a level that will not erode the receiving channel or area.

For low flows and low velocities, the most commonly used device for absorption of the impact of flow is a riprap apron. They are constructed at a zero grade for a distance that is related to the outlet flow rate and the tailwater level. See the ADOT Erosion and Pollution Control Manual for additional information. Where the flow force is excessive, structural energy dissipators can be used. See Chapter 11, Energy Dissipator for additional information regarding energy dissipators.

Design Detailing

Permissible velocity guidelines for grass and earth-lined channels are presented in Chapter 7, Channels for aid in the determination of outlet protection needs.

6.5 Erosion Sediment Control Plan

6.5.1 Control Plan

Depending on the amount of land disturbance that a project will impact, a formal storm water pollution prevention plan (SWPPP) may be required. The **ADOT Erosion and Pollution Control Manual for Highway Design and Construction** shall be consulted for development of the SWPPP. As stated in the above referenced manual "The plan is to identify potential sources of pollution and describe practices that will be implemented to reduce erosion, minimize sedimentation and eliminate non-storm water pollutants for the site." This plan is to be prepared by a Registered Landscape Architect. The drainage designer may need to assist the SWPPP preparer with the collection of hydrologic and hydraulic data. The designer should inspect the site to verify natural drainage patterns, drainage areas, general soil characteristics and off-site factors.

The base data should reflect such characteristics as:
- land slopes,
- natural drainage patterns,
- unstable stream reaches and flood marks,
- watershed areas,
- existing vegetation (noting special vegetative associations),
- critical areas such as steep slopes, eroding areas, rock outcroppings and seepage zones,
- unique or noteworthy landscape values to protect,
- adjacent land uses — especially areas sensitive to sedimentation or flooding, and
- critical or highly erodible soils that should be left undisturbed.

6.6 References

Arizona Department of Transportation, Intermodal Transportation Division, Erosion and Pollution Control Manual for Highway Design and Construction

American Association of State Highway and Transportation Officials, A Policy on Geometric Design of Highways and Streets. 1994.

American Association of State Highway and Transportation Officials, Guideline Volume III, Erosion and Sediment Control in Highway Construction.

U.S. Department of Interior, Bureau of Reclamation, Hydraulic Design of Stilling Basins and Energy Dissipators, Engineering Monograph No. 25.

U.S. Department of Transportation, Federal Highway Administration, Design of Stable Channels with Flexible Linings, Hydraulic Engineering Circular No. 15. 1975.

U.S. Department of Transportation, Federal Highway Administration, Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14.

CHAPTER 7

CHANNELS

Chapter 7 Channels
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Chapter 7 Channels
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7.1 Introduction

7.1.1 Purpose

This chapter provides a discussion of the hydraulic behavior of channels. Guidance in the analysis and design of channels is provided through:

- presentation of the appropriate ADOT policy, philosophy and design criteria,
- discussion of geomorphic factors that affect channel behavior,
- presentation of an outlined design procedure,
- discussion of analysis of channels by computer programs such as HEC-RAS, and
- demonstration of design techniques by example problems.

7.1.2 Objectives in hydraulic design of channels.

An objective in the hydraulic design of channels is to provide a means to convey concentrated flows in a controlled manner with acceptable impacts to the environment. The objective in performing a hydraulic design or analysis of a channel is to assess:

- the hydraulic forces and predict the future behavior of streams and channels,
- impacts caused by changes in water surface profiles,
- impacts caused by changes in lateral flow distributions,
- impacts caused by changes in velocity or direction of flow,
- need for and value of channel linings for the control of erosion.

7.1.3 Concepts

Open channels are a natural or man-made conveyance for water in which:

- the water surface is exposed to the atmosphere, and
- the gravity force component in the direction of motion is the driving force.

There are various types of open channels encountered by the designer of transportation facilities:

- stream channel,
- roadside channel or ditch,
- irrigation channel,
- drainage ditch, and
- culvert in low flow.

7.1 Introduction (continued)

7.1.3 Concepts (continued)

Stream channels are:
- usually natural channels with their size and shape determined by natural forces,
- usually compound in cross section with a main channel for conveying low flows and a floodplain to transport flood flows, and
- usually shaped geomorphologically by the long-term history of sediment load and water discharge that they experience.

Artificial channels include roadside channels, irrigation channels and drainage ditches that are:
- man-made channels with regular geometric cross sections, and
- unlined, or lined with artificial or natural material to protect against erosion.

While the principles of open channel flow are the same regardless of the channel type, stream channels and artificial channels (primarily roadside channels) will be treated separately in this chapter as needed. The term roadside channel or ditch is used for those elements that collect and convey surface or sheet flow of storm water runoff from the highway and adjacent lands. Stream channel is used for elements that convey concentrated stream flows. Usually the alignment and profile of roadside channels and ditches is governed by the highway cross-section description. Relocated stream channels usually have an alignment independent of the roadway.

7.1.4 Design Goals and Considerations

7.1.4.1 ADOT Design Goals

Hydraulic design associated with natural channels and roadway ditches is a process that identifies and evaluates alternatives according to established criteria.

The range of design channel discharges shall be selected based on class of roadway, consequences of traffic interruption, flood hazard risks, economics, and local site conditions.

- Safety of the general public shall be an important consideration in the selection of cross-sectional geometry of artificial drainage channels.
- A stable channel is the goal for all man-made channels.
- Environmental impacts of channel modifications, including disturbance of habitat, wetlands, and streambank stability shall be assessed.
- The design of man-made drainage channels shall consider the frequency and type of maintenance expected and provide for access of maintenance equipment.
- Changes in water surfaces shall not significantly increase flood damage to property,
- Changes in velocity should not significantly alter the channel behavior nor significantly increase damage to adjacent property.

7.1 Introduction (continued)

7.1.4.2 Design Criteria

7.1.4.2.1 General

The following criteria apply to the design of hydraulic structures in natural or roadside channels and may be altered as appropriate when approved by the Drainage Section.

- As necessary, the hydraulic effects of flood plain encroachments should be evaluated for a peak discharge of the 100-year recurrence interval on any highway facility. Other discharges may need to be reviewed for a full understanding of the impact of the facility on the stream and surrounding environment.

- The effects of changes such as channel realignment, slope modification, section modification, and conveyance alteration must be adequately analyzed. Adverse effects such as bend and bank instability evaluated for possible mitigation requirements.

- If relocation of a stream channel is unavoidable, the cross-sectional shape, meander pattern, roughness, sediment transport, and slope should, in so far as practicable, conform to the existing conditions. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.

- Where overtopping would permit storm water to breakout of ADOT right-of-way, the minimum freeboard shall be 1-foot.

  For channels with flows having Froude Numbers equal to or greater than 0.86, the minimum freeboard should be the larger of 1-foot or the value

  $$F= 0.20*(y+v^2/2g)$$

  where:

  F = desired freeboard (1 ft minimum), ft;
  y = depth of flow, ft;
  v = mean velocity, ft/s;
  g = acceleration due to gravity, 32.2 ft/sec^2.

  For leveed channels where the water surface elevation is higher than natural ground, provide an additional 1-foot of freeboard to accommodate surface irregularities and alignment adjustments.

7.1 Introduction (continued)

7.1.4.2 Design Criteria (continued)

- In earthen channels, side slopes shall be flatter than the angle of repose of the soil and/or lining and shall be 2:1 or flatter in the case of rock-riprap lining.

- Streambank stabilization shall be provided, when appropriate, as a result of any stream disturbance such as encroachment and shall extend to include both upstream and downstream banks as well as the local site.

- Flexible linings shall be designed according to the method of allowable tractive force.

- The design discharge for channel linings shall be based on the hazard associated with the failure of lining. For lining that protects the highway embankment from erosion, the design frequency shall be not less that the operational frequency of the highway. Where the failure of roadside channels would increase the flood hazard of adjacent properties the channel shall safely convey the 100-year event within the ADOT r/w. The minimum design discharge for permanent roadside ditch linings shall be a 10-year frequency while temporary linings may be a 2-year frequency flow.

7.2 Stream Morphology

7.2.1 Introduction

The form assumed by a natural stream, which includes its cross-sectional shape as well as its planform, is a function of many variables for which cause-and-effect relationships are difficult to establish. The natural stream channel will assume a geomorphological form that will be compatible with the sediment load and discharge history that it has experienced over time. The stream may be graded or in equilibrium with respect to long time periods, which means that on the average it discharges the same amount of sediment that it receives although there may be short-term adjustments in its bedforms in response to flood flows. On the other hand, the stream reach of interest may be aggrading or degrading as a result of deposition or scour in the reach, respectively. The planform of the stream may be straight, braided, or meandering.

To the extent that a highway structure disturbs this delicate balance by encroaching on the natural channel, the consequences of flooding, erosion, and deposition can be significant and widespread. The hydraulic analysis of a proposed highway structure should include a consideration of the extent of these consequences.

These complexities of stream morphology can be assessed by inspecting aerial photographs and topographic maps for changes in slope, width, depth, meander form, and bank erosion with time. A qualitative assessment of the river response to proposed changes is possible through a thorough knowledge of river mechanics and accumulation of engineering experience. Equilibrium sediment load calculations can be made by a variety of techniques and compared from reach to reach to detect an imbalance in sediment inflow and outflow and thus identify an aggradation/degradation problem. References (FHWA, 1990) should be consulted to evaluate the problem and propose mitigation measures. The proposed methodology should be approved before beginning any in-depth study of a site.

7.2 Stream Morphology (continued)

7.2.2 Levels Of Assessment

The analysis and design of a stream channel will usually require an assessment of the existing channel and the potential for problems as a result of the proposed action. The detail of studies necessary should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream. Observation of the existing stream is the best means of identifying potential locations for channel bank erosion and subsequent channel stabilization. Analytical methods for the evaluation of channel stability can be classified as either hydraulic or geomorphic, and it is important to recognize that these analytical tools should only be used to substantiate the erosion potential indicated through observation. Brief descriptions of the three levels of assessment are as follows:

Level 1
Qualitative assessment involving the application of geomorphic concepts to identify potential problems and alternative solutions. Data needed may include historic information, current site conditions, aerial photographs, old maps and survey notes, bridge design files, maintenance records, and interviews with long-time residents.

Level 2
Quantitative analysis combined with a more detailed qualitative assessment of geomorphic factors generally includes water surface profile and scour calculations. This level of analysis will be adequate for most locations if the problems are resolved and relationships between different factors affecting stability are adequately explained. Data needs will include Level 1 data in addition to the information needed to establish the hydrology and hydraulics of the stream.

Level 3
Complex quantitative analysis based on detailed mathematical modeling and possibly physical hydraulic modeling. These methods are necessary only for high-risk locations, extraordinarily complex problems, and possibly after the fact analysis where losses and liability costs are high. This level of analysis may require professionals experienced with mathematical modeling techniques for sediment routing and/or physical modeling. Data needed will require Level 1 and 2 data as well as field data on bed load and suspended load transport rates and properties of bed and bank materials such as size, shape, gradation, fall velocity, cohesion, density, and angle of repose.

7.2.3 Factors That Affect Stream Stability

An alluvial stream is continually changing its position and slope as a consequence of hydraulic factors acting on its bed and banks. These changes may be slow or rapid and may result from natural environmental changes or man's activity.

7.2 Stream Morphology (continued)

7.2.3 Factors That Affect Stream Stability (continued)

A study of the plan and profile is useful in understanding stream morphology. Factors that affect the stream shape and stability and, potentially, bridge and highway stability at stream crossings, can be classified as geomorphic factors and hydraulic factors.

Geomorphic Factors:

- Size
- Flow habit
- Bed material
- Valley setting
- Flood plains
- Natural levees
- Apparent incision
- Channel boundaries
- Cut banks
- Tree cover on banks
- Sinuosity
- Degree of braiding
- Degree of anabranching
- Width variability
- Bar development

Figure 7-1 depicts examples of the various geomorphic factors.

Hydraulic Factors.

- Magnitude, frequency and duration of floods.
- Bed configuration: ripples, dunes, plane bed, antidunes
- Resistance to flow: Manning' n
- Water surface profiles.

Figure 7-2 depicts the changes in channel classification and relative stability as related to hydraulic factors.

Rapid and unexpected changes may occur in streams in response to man's activities in the watershed. Changes in perviousness can alter the hydrology of a stream, sediment yield, and channel geometry. Channelization, stream channel straightening, stream levees and dikes, bridges and culverts, reservoirs, and changes in land use can have major effects on stream flow, sediment transport, and channel geometry and location. Knowing that man's activities can influence stream stability can help the designer anticipate some of the problems that can occur. Natural disturbances such as floods, drought, earthquakes, landslides, volcanoes, and forest fires can also cause large changes in sediment load and thus major changes in the stream channel. Although difficult to plan for such disturbances, it is important to recognize that when natural disturbances do occur, it is likely that changes will also occur to the stream channel.

7.2 Stream Morphology (continued)

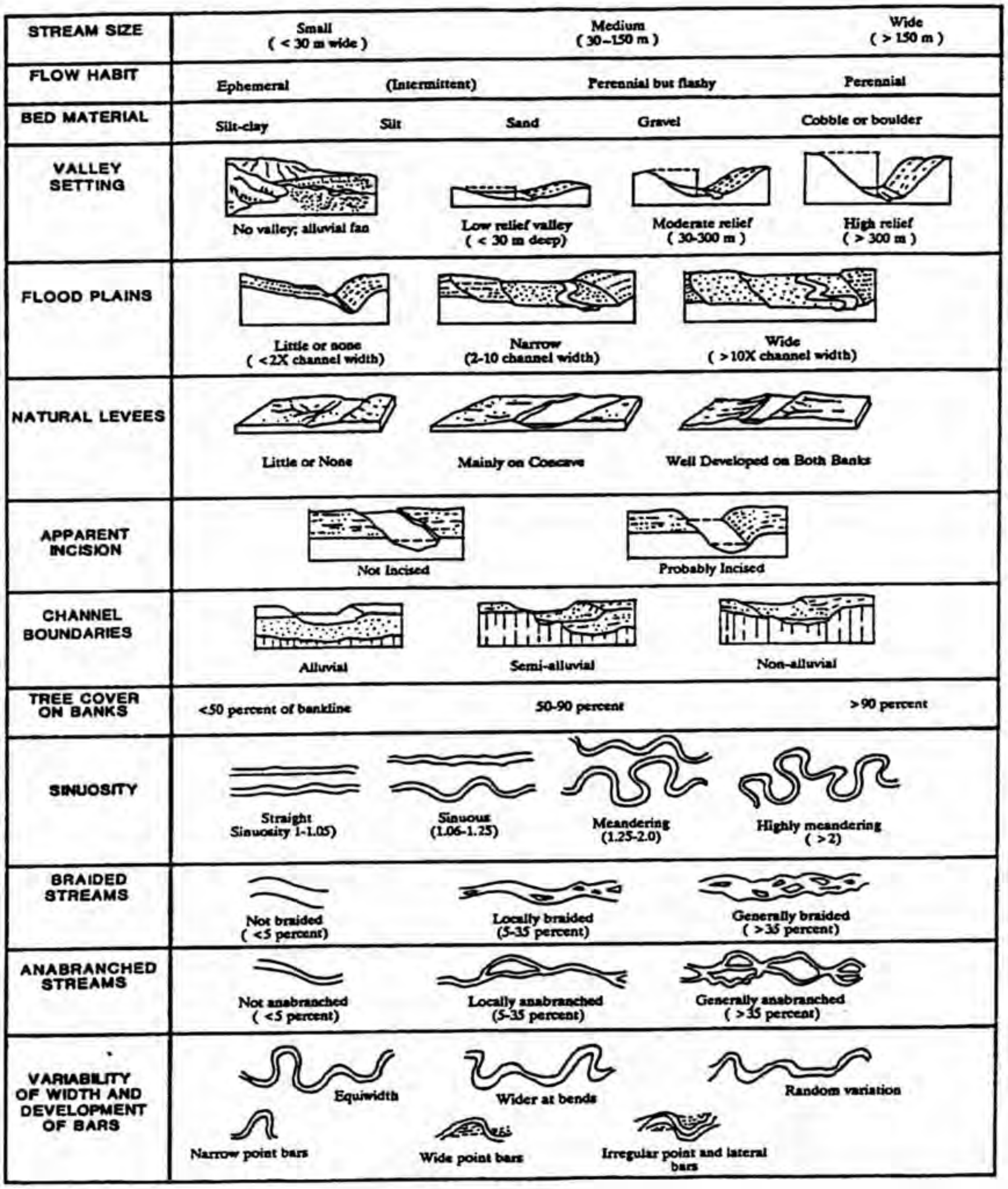


Figure 7-1 Geomorphic Factors That Affect Stream Stability

Source: Adapted From Brice and Blodgett, 1978

7.2 Stream Morphology (continued)

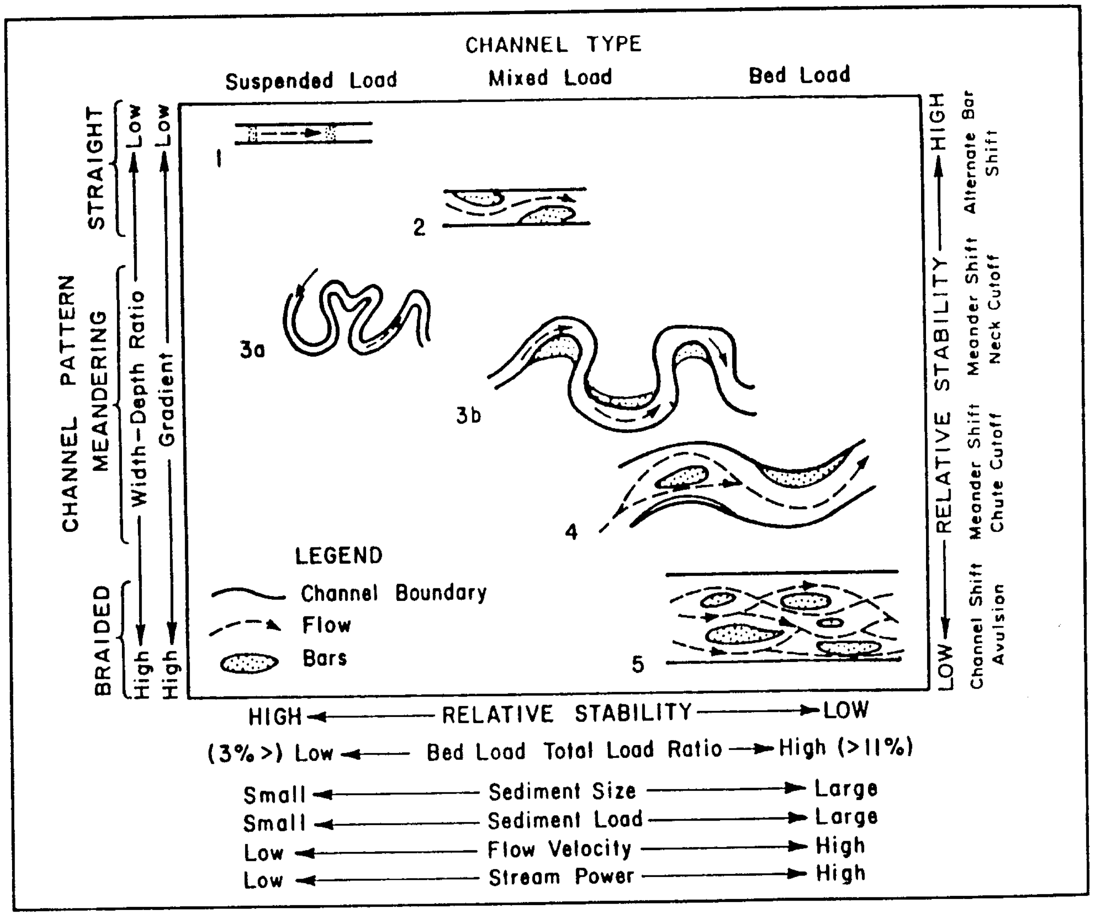


Figure 7-2 Channel Classification And Relative Stability As Hydraulic Factors Are Varied
Source: Atter, Shen et al., 1981

7.2 Stream Morphology (continued)

7.2.4 Stream Classification

David L. Rosgen (Rosgen, 1994 and 1996) developed a stream classification system that is a widely accepted method for classifying streams. This system may be of use in determining the sensitivity of a site to lateral migration, and change rates. This information may be used to address the scour susceptibility of foundation elements located outside the floodplain. Aa, A, and B-type streams will not normally be very active in lateral migration. Foundation elements that are outside the floodplain may be able to be constructed at elevations shallower than the thalweg of the stream. This will require approval of the Drainage Section. D, F, and G-type streams will usually be very active in lateral migration and bank instability: they will need to have the foundation elements be designed with the possibility of being exposed to the main-channel flow at some time in the future.

Rosgen's stream classification system is delineated initially into major, broad, stream categories of A – G as shown in Table 7-1. At this level, which Rosgen refers to as level I, the classification system uses the entrenchment ratio, sinuosity, width/depth ratio, and the channel slope as the delineative criteria for classifying a river. The entrenchment ratio is the ratio of the width of the flood-prone area to the bankfull surface width of the channel. The flood-prone area is defined as the width measured at an elevation that is determined at twice the maximum bankfull depth. The width/depth ratio is the ratio of bankfull channel width to bankfull mean depth. The bankfull mean depth is the bankfull area divided by the bankfull channel width. Sinuosity is the ratio of stream length to valley length and it can also be described as the ratio of valley slope to channel slope. Slope is the water surface slope and can be determined by measuring the difference in water surface elevation per unit stream length. At the broad level classification the slope can be estimated from USGS quadrangle maps.

The broad level classifications are then broken into sub-classes based on the dominant bed material. The stream types are assigned numbers related to the size of the dominant bed material such that 1 is bedrock, 2 is boulder, 3 is cobble, 4 is gravel, 5 is sand, and 6 is silt/clay. This produces 41 major stream types as shown in Table 7-2. Rosgen's classification system also incorporates a continuum concept. The continuum concept is applied where delineative criteria values outside the normal range are encountered but do not warrant a unique stream type. This yields the following sub-categories based on slope: a+ (steeper than 0.10), a (0.04 – 0.099), b (0.02 – 0.039), c (flatter than 0.02), and c- (flatter than 0.001). The continuum concept also allows the entrenchment ratio and sinuosity to vary by $\pm$0.2 unit and sinuosity can vary by $\pm$2.0 units. The expanded classification system that incorporates the continuum concept is shown in Table 7-3. Rosgen refers to the classifications shown in Tables 7-2 and 7-3 as level III.

Stream Type	General Description	Entrenchment Ratio	W/D Ratio	Sinuosity	Slope	Landform/Soils/Features
Aa+	Very steep, deeply entrenched, debris transport streams.	<1.4	<12	1.0 - 1.1	>0.10	Very high relief. Erosional, bedrock or depositional features; debris flow potential. Deeply entrenched streams. Vertical steps with deep scour pools; waterfalls.
A	Steep, entrenched, cascading, step/pool streams. High energy/debris transport associated with depositional soils. Very stable if bedrock or boulder dominated channel	<1.4	<12	1.0 -1.2	0.04 - 0.10	High relief. Erosional or depositional and bedrock forms. Entrenched and confined streams with cascading reaches. Frequently spaced, deep pools in associated step/pool bed morphology.
B	Moderately entrenched, moderate gradient, riffle dominated channel with infrequently spaced pools. Very stable plan and profile. Stable banks.	1.4 - 2.2	>12	>1.2	0.02 - 0.039	Moderate relief, colluvial deposition and/or residual soils. Moderate entrenchment and width/depth ratio. Narrow, gently sloping valleys. Rapids predominate with occasional pools.
C	Low gradient, meandering, point-bar, riffle/pool, alluvial channels with broad well defined floodplains	>2.2	>12	>1.4	<0.02	Broad valleys with terraces in association with floodplains, alluvial soils. Slightly entrenched with well-defined meandering channels. Riffle/pool bed morphology.
D	Braided channel with longitudinal and transverse bars. Very wide channel with eroding banks.	n/a	>40	N/a	<0.04	Broad valleys with alluvial and colluvial fans. Glacial debris and depositional features. Active lateral adjustment with abundance of sediment supply.
DA	Anastomosing (multiple channels) narrow and deep with expansive well vegetated floodplain and associated wetlands. Very gentle relief with highly variable sinuosities. Stable streambanks.	>2.2	Highly variable	Highly variable	<0.005	Broad, low gradient valleys with fine alluvium and/or lacustrine soils. Anastomosed geologic control creating fine deposition with well-vegetated bars that are laterally stable with broad wetland floodplains.
E	Low gradient, meandering riffle/pool stream with low width/depth ratio and little deposition. Very efficient and stable. High meander width ratios.	>2.2	<12	>1.5	<0.02	Broad valley/meadows. Alluvial materials with floodplain. Highly sinuous with stable, well vegetated banks. Riffle/pool morphology with very low width/depth ratio.
F	Entrenched meandering riffle/pool channel on low gradients with high width/depth ratio.	<1.4	>12	>1.4	<0.02	Entrenched in highly weathered material. Gentle gradients with a high width/depth ratio. Meandering, laterally unstable with high bank-erosion rates. Riffle/pool morphology.
G	Entrenched “gully” step/pool and low width/depth ratio on moderate gradients.	<1.4	<12	>1.2	0.02 – 0.039	Gully, step/pool morphology with moderate slopes and low width/depth ratio. Narrow valleys or deeply incised in alluvial or colluvial materials; i.e., fans or deltas. Unstable with grade control problems and high bank erosion rates.

Table 7-1 Summary of Delineative Criteria for Broad-Level Classification

Rosgen's River Classification										
Bed Material	Bedrock	A1a+	A1	B1	C1				F1	G1
	Boulder	A2a+	A2	B2	C2				F2	G2
	Cobble	A3a+	A3	B3	C3	D3		E3	F3	G3
	Gravel	A4a+	A4	B4	C4	D4	DA4	E4	F4	G4
	Sand	A5a+	A5	B5	C5	D5	DA5	E5	F5	G5
	Silt/Clay	A6a+	A6	B6	C6	D6	DA6	E6	F6	G6
Criteria	Entrenchment	<1.4	<1.4	1.4 - 2.2	>2.2	N/A	>4.0	>2.2	<1.4	<1.4
	Sinuosity	1.0 -1.1	1.0 - 1.2	>1.2	>1.2	N/A	Variable	>1.5	>1.2	>1.2
	Width/Depth	<12	<12	>12	>12	>40	<40	<12	>12	<12
	Slope	>0.10	0.04 - 0.099	0.02 - 0.039	<0.02	<0.04	<0.005	<0.02	<0.02	0.02 - 0.039

Table 7-2 Rosgen's River Classification System

		Single Thread Channels														Multiple Channels			
Entrenchment Ratio[1]		Entrenched (<1.4)						Moderately Entrenched (1.4 – 2.2)			Slightly Entrenched (>2.2)					N/A			
Width/Depth Ratio[2]		Low (<12)				Moderate -High (>12)		Moderate (>12)			Very Low (<12)		Moderate – High (>12)			Very High (>40)			Low (<40)
Sinuosity[1]		Low (<1.2)		Moderate (>1.2)		High (>1.2)		Moderate (>1.2)			Very High (>1.5)		High (>1.2)			Low (<1.2)			Low - High (1.2-1.5)
Broad Class		A		G		F		B			E		C			D			DA
Slope Range		>0.10	0.04 to 0.099	0.02 to 0.039	<0.02	0.02 to 0.039	<0.02	0.04 to 0.099	0.02 to 0.039	<0.02	0.02 to 0.039	<0.02	0.02 to 0.039	0.001 to 0.02	less than 0.001	0.02 to 0.039	0.001 to 0.02	less than 0.001	less than 0.005
CHANNEL MATERIAL	Bedrock	A1a+	A1	G1	G1c	F1b	F1	B1a	B1	B1c			C1b	C1	C1c-				
	Boulders	A2a+	A2	G2	G2c	F2b	F2	B2a	B2	B2c			C2b	C2	C2c-				
	Cobbles	A3a+	A3	G3	G3c	F3b	F3	B3a	B3	B3c	E3b	E3	C3b	C3	C3c-	D3b	D3		
	Gravel	A4a+	A4	G4	G4c	F4b	F4	B4a	B4	B4c	E4b	E4	C4b	C4	C4c-	D4b	D4	D4c-	DA4
	Sand	A5a+	A5	G5	G5c	F5b	F5	B5a	B5	B5c	E5b	E5	C5b	C5	C5c-	D5b	D5	D5c-	DA5
	Silt/Clay	A6a+	A6	G6	G6c	F6b	F6	B6a	B6	B6c	E6b	E6	C6b	C6	C6c-	D6b	D6	D6c-	DA6

[1]Values can vary by ±0.2 unit [2]Values can vary by ±2.0 unit

Table 7-3 Rosgen's Stream Classification System

7.3 Alluvial Streams

7.3.1 Classification

The purpose of the stream classification system is to assist the users in assessing stream stability and in choosing the appropriate sediment transport equation. The methods utilized are predicated on bed material sediment size and stream channel slope. Stream morphology and related channel patterns are directly influenced by the width, depth, velocity, discharge, slope, and roughness of channel material, sediment load and sediment size. Changes in any of these variables can result in altered channel patterns. As stream morphology is a result of these mutually adjustable variables, those most directly measurable were incorporated into Rosgen's criteria for stream classification. Stream channel patterns are classified based upon bed material size, channel gradients, and channel entrenchment and confinement.

7.3.1.1 Stream types

There are many properties by which streams may be classified into categories. 14 categories have been identified, see page 7-9, these categories are combined in groupings that represent a characteristic association of properties. A reason for using these classifications is to facilitate the assessment of the stream for engineering purposes, with particular regard to lateral stability.

The benefit is based on two premises: (1.) that lateral stability, as well as several other aspects of stream behavior is reflected in the physical appearance of the stream and its channel, and (2.) the best guide to the behavior of a stream is its behavior during the immediate past.

NOTES ON STREAM TYPES:
Five alluvial stream types have been identified.

These are
- TYPE A: Equiwidth, point-bar system
- TYPE B: Wide bend, point-bar system
- TYPE C: Braided, Point-bar system
- TYPE D: Braided stream, without point bars.
- TYPE E: Anabranched streams

Characteristics and engineering significance is presented in the next paragraphs.

7.3 Alluvial Streams (continued)

7.3.1 Classification (continued)

TYPE A: Equiwidth, point-bar system

The main characteristic is that it has point bars, with lateral bars being rare. It is not braided, and it may have any degree of sinuosity.

Engineering significance--It is the most stable of all stream types, meanders may gradually migrate. Rate of bed load transport is probably small in relation to suspended load.

TYPE B: Wide bend, point-bar system

In addition to many point bars, it may have a few lateral bars and be locally braided. Markings of point bars, if visible, will tend to be concentric. The stream may have any degree of sinuosity. Meanders may be of the neck or chute type.

Engineering significance--this type of stream may have straight reaches that are stable for decades; however, there is a potential for high rates of lateral migration at bends. There is substantial transport of bed material, either sand or gravel.

TYPE C: Braided, Point-bar system

This stream will have a mix of bars; point bars, lateral bars and mid-channel bars. The bars may be of sand, gravel or cobbles. It is locally or generally braided, but has a continuous thalweg. The thalweg may be either sinuous or meandering, may be fairly stable or shift dramatically during floods. The making of point bars tends to be irregular and not concentric. There may be variability in stream width. The main channel may be sinuous, but less than the thalweg.

Engineering significance--This type of stream has the potential for a very high rate of lateral erosion. Rapid movement of the thalweg or chute cutoffs of bends may occur and result in alignment problems. Potentially deep scour may occur at the thalweg, particularly if the bed is silt or sand. The transport of bed load (sand, gravel, or cobbles) probably exceeds the transport of suspended load.

TYPE D: Braided stream, without point bars.

This type of stream has no point bars at banks in the main channel. It will have many mid-channel and lateral bars, the flow may be completely divided. There will be scattered small islands, or islands more numerous than bars. There is usually random variation in the stream width.

Engineering significance--The channel tends to be wide and shallow, requires a long bridge confined by suitable countermeasures. The lateral erosion rates are low to moderate, but the point of erosion is not predictable. The braids shift at each high flow, and unexpected depths of scour may occur where braids join to form a deep channel. The load is transported mainly as bed load, either sand, gravel, or cobbles.

7.3 Alluvial Streams (continued)

7.3.1 Classification (continued)

TYPE E: Anabranched streams

The flow is distinctly divided into channels separated by large islands, which are usually covered with permanent vegetation. Anabranches are likely to be locally braided. Point bars are likely at bends in anabranches. The stream, as well as individual anabranches, may be straight, sinuous, or meandering.

Engineering significance--A long bridge is required unless the stream is crossed at a local point where it is not anabranched. If multiple structures are used, the percent of total flow at each bridge may not be predictable. The stability of each anabranch on streams differs greatly, and should be assessed as though an anabranch were an individual stream.

7.3.1.2 Stream Alignment

Streams may be further classed by considering their alignment. Whether it is straight, meandering, or braided. In general, braided rivers are relatively steep and meandering rivers have more gentle slopes. Meandering rivers are not subject to rapid movement and are reasonably predictable in behavior. Nevertheless they are generally unstable with eroding banks.

The Meandering stream:

A meandering stream consists of pools and crossings. The thalweg, or main current of the channel, flows from the pool through the crossings to the next pool forming the typical s-curve. In pools, the channel cross-section is somewhat triangular. Point bars form on the inside of the bends. In the crossings, the channel cross-section is more rectangular and depths are smaller. At low flows the local slope is steeper and velocities are larger in the crossing than in the pool. At low stages the thalweg is located very close to the outside of the bend.

At higher stages, the thalweg tends to straighten. More specifically, the thalweg moves away from the outside of the bend encroaching on the point bar to some degree. In general, the process of erosion and deposition forms bends. As a meandering stream moves laterally and longitudinally the meander loops move at unequal rates because of the unequal erodability of the banks. The channel geometry depends on the local slope, the bend material, and the geometry of the adjacent bends.

7.3 Alluvial Streams (continued)

7.3.1 Classification (continued)

The Meandering Stream: (continued)

A meandering river has more or less regular inflections that are sinuous in plan. It consists of a series of bends connected by crossings. In the bends, deep pools are carved adjacent to the concave banks by the relatively high velocities. On the inside of the bends, as velocities are lower, sediments are deposited, forming the point bar. Much of the sediment eroded from the outside bank is deposited in the crossing and on the point bar in the next bend downstream. The crossings are short, straight reaches which connect the bends; they are quite shallow compared to the pools in the bendways.

The geometry of meandering rivers is quantitatively measured in terms of (1) meander wavelength, l; (2) meander width, Wm; (3) mean radius of curvature, Rc; (4) mean amplitude, a; and (5) bend deflection angle.

The Braided Stream:

A braided stream is one that consists of multiple and interlacing channels. One cause of braiding is the large quantity of bed load the stream is unable to transport. Another cause of braiding is easily eroded banks. The braided stream presents difficulties because it is unstable, changes alignment rapidly, carries large quantities of sediment, is very wide and shallow even at flood flows and is, in general, unpredictable.

The excess bed load results in braiding when the channel is overloaded with sediment, deposition occurs, the bed aggrades, and the slope of the channel increases in an effort to obtain a graded state. As the channel steepens, the velocity increases, and multiple channels develop. These interlaced multiple channels cause the overall channel system to widen. The multiple channels result as bars of sediment are deposited within the main channel.

Erodible banks contribute to braiding as a response to changing flows, the stream widens at high flows and forms bars at low flow which become stabilized, forming islands. In general, a braided channel has a large slope, a large-bed material load in comparison with its suspended load, and relatively small amounts of silts and clays in the bed and banks.

7.3 Alluvial Streams (continued)

7.3.2 Stream Response To Change

The major complicating factors in river mechanics are: 1) the large number of interrelated variables that can simultaneously respond to natural or imposed changes in a stream system; and 2) the continual evolution of stream channel patterns, channel geometry, bars, and forms of bed roughness with changing water and sediment discharge. In order to better understand the responses of a stream to the actions of man and nature, a few simple hydraulic and geomorphic concepts are presented herein.

Hydraulic geometry is a general term applied to alluvial channels to denote relationships between discharge, Q, and the channel morphology, hydrology, and sediment transport. In alluvial channels, the morphologic, hydraulic and sedimentation characteristics of the channel are determined by a large variety of factors. The mechanics of such factors are not fully understood; however, alluvial streams do exhibit some quantitative hydraulic geometry relations. The hydraulic geometry relations express the integral effects of all the hydrologic, meteorological and geologic variables in a drainage basin.

In general, the hydraulic geometry relations are stated as power functions of the discharge. The dependent variables identified are the channel width, W; the channel depth, Yo; the average velocity, v; the total bed sediment load, Qt; the average friction slope, Sf; and the average Manning's roughness coefficient, n.

At a given cross-section, width, depth and velocity increase systematically with discharge. A base condition is established by the dominant discharge, usually a flow of 1.5 to 2.33 years frequency. The geologic character of the region through which the channel runs initially establishes the longitudinal slope of a channel.

The initial slope is modified to a greater or lesser extent by a stream, or by is ancestral streams. The dependence of stream form on slope, which may be imposed independently of other stream characteristics, is illustrated schematically in Figure 7-3.

Any natural or artificial change that alters channel slope can result in modifications to the existing stream pattern. For example, a cutoff of a meander loop decreases channel sinuosity and increases channel slope. Referring to Figure 7-3, this shift in the plotting position to the right could result in a shift from a relatively tranquil, meandering pattern toward a braided pattern that varies rapidly with time, has high velocities, is subdivided by sandbars, and carries relatively large quantities of sediment. Conversely, it is possible that a slight decrease in slope could change an unstable braided stream into a meandering one.

7.3 Alluvial Streams (continued)

7.3.2 Stream Response To Change (continued)

The different channel dimensions, shapes, and patterns associated with different quantities of discharge and amounts of sediment load indicate that as these independent variables change, major adjustments of channel morphology can be anticipated. Further, a change in hydrology may cause changes in stream sinuosity, meander wavelength, and channel width and depth. A long period of channel instability with considerable bank erosion and lateral shifting of the channel may be required for the stream to compensate for the hydrologic change.

Stream Stability Problems:

For engineering purposes, an unstable channel is one whose rate or magnitude of change is great enough to be a significant factor in the planning or maintenance of a highway crossing during the service life of the structure. The kinds of changes considered are (1.) lateral bank erosion, (2.) degradation or aggradation of the stream bed that continues progressively over a period of years; and (3.) natural short-term fluctuations of streambed elevations that are usually associated with the passage of a flood (scour and fill).

Stability is inferred mainly from the nature of point bars, the presence or absence of cut banks, and the variability of stream width.

Bank Stability

On a laterally unstable channel, or at actively migrating bends on an otherwise stable channel, the point bars are usually wide and unvegetated and the bank opposite to a point is cut and often scalloped by erosion. The crescentic scars of slumping may be visible from place to place along the bank line. The presence of a cut bank opposite to point bar is evidence of instability, even if the point bar is vegetated.

Along an unstable channel, bank erosion tends to be localized at bends, and straight reaches tend to be relatively stable. However, meandering of the thalweg in a straight reach is likely to be a precursor of instability. Bars that occur alternately from one side to the other of a straight reach are somewhat analogous to point bars and are indicative of a meandering thalweg.

Bank Erosion Rates: Although it is theoretically possible to determine bank erosion rates from factors such as water velocity and resistance of the banks to erosion, practical and accurate means of making such determination are still deficient. Past rates of erosion at a particular site provide the best estimate of future rates. In projecting past rates into the future, consideration must be given to the following factors: (1.) the past flow history of the site during the life of the highway crossing. The duration of floods, or of flows near bankfull stages, is probably more important that the magnitude of floods; and (2.) man-induced factors that are likely to affect bank erosion rates. Among the most important of these are urbanization and the clearing of flood plain forests.

7.3 Alluvial Streams (continued)

7.3.2 Stream Response To Change (continued)

Behavior of meander loops

While no two meanders will behave in exactly the same way, meanders on a particular stream reach will tend to conform to one of several modes of behavior. The modes are of

A. extension	B. translation
C. rotation	D. conversion to a compound loop
E. neck cutoff by a closure	F. diagonal cutoff by a chute
	G. neck cutoff by a chute

Mode A represents the typical development of a loop of low amplitude, which decreases in radius as it extends slightly in a downstream direction. Mode B rarely occurs unless meanders are confined by valley sides on a narrow flood plain, or are confined by artificial levees.

Mode C is a pattern typically followed by well-developed meanders on streams that have unstable banks. Mode E also applies to loops on meandering or highly meandering streams, usually of the equiwidth point-bar type. The banks have been sufficiently stable for an elongated loop to form (without being cutoff), but the neck of the loop gradually being closed and cutoff will eventually occur at the neck.

Modes F and G apply mainly to locally braided sinuous or meandering streams having unstable banks. Loops are cutoff by chutes that break diagonally or directly across the neck.

Effect of meander cutoff

The cutoff of a meander will cause a local increase in channel slope, which results in an increase in the growth rate of adjoining meanders, and an increase in channel width at the point of cutoff. On a typical wide-bend point-bar stream the effects of cutoff do not extend very far upstream or downstream.

7.3 Alluvial Streams (continued)

7.3.2 Stream Response To Change (continued)

Assessment of degradation.

Annual rates of degradation averaged from past records such as the closure of a dam give poor estimates of future rates of degradation. Typical situations exhibit an exponential decay function of the rate of channel degradation.

Indicators of degradation are listed in approximate order of reliability:

1. channel scarps, headcuts, and nickpoints
2. gullying of minor side tributaries
3. high and steep unvegetated banks
4. measurements of streambed elevation
5. changes in stream discharge relationships
6. measurement of longitudinal profile

Trends in response to change.

Figure 7-4 illustrates the dependence of river form on channel slope and discharge. It shows that when $SQ^{1/4} \leq .0017$ in a sandbed channel, the stream will meander. Similarly, when $SQ^{1/4} \geq .010$, the stream is braided.

In these equations, S is the channel slope in feet-per-foot and Q is the mean discharge in cfs. Between these values of $SQ^{1/4}$ is the transitional range.

Many U.S. rivers plot in this zone between the limiting curves defining meandering and braided streams. If a stream is meandering but its discharge and slope border on a boundary of the transitional zone, a relatively small increase in channel slope may cause it to change, in time, to a transitional or braided stream.

7.3 Alluvial Streams (continued)

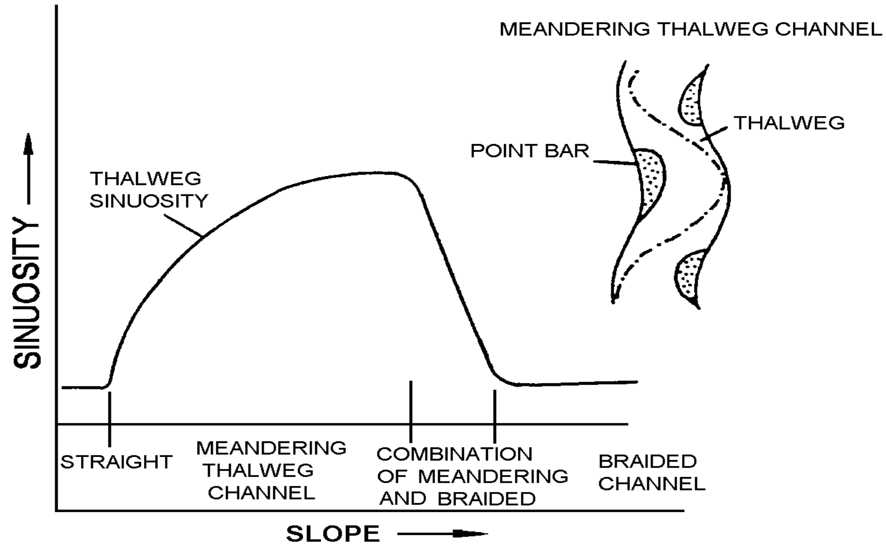


Figure 7-3 Sinuosity versus Slope with Constant Discharge
Source: After, Richardson et al., 1988

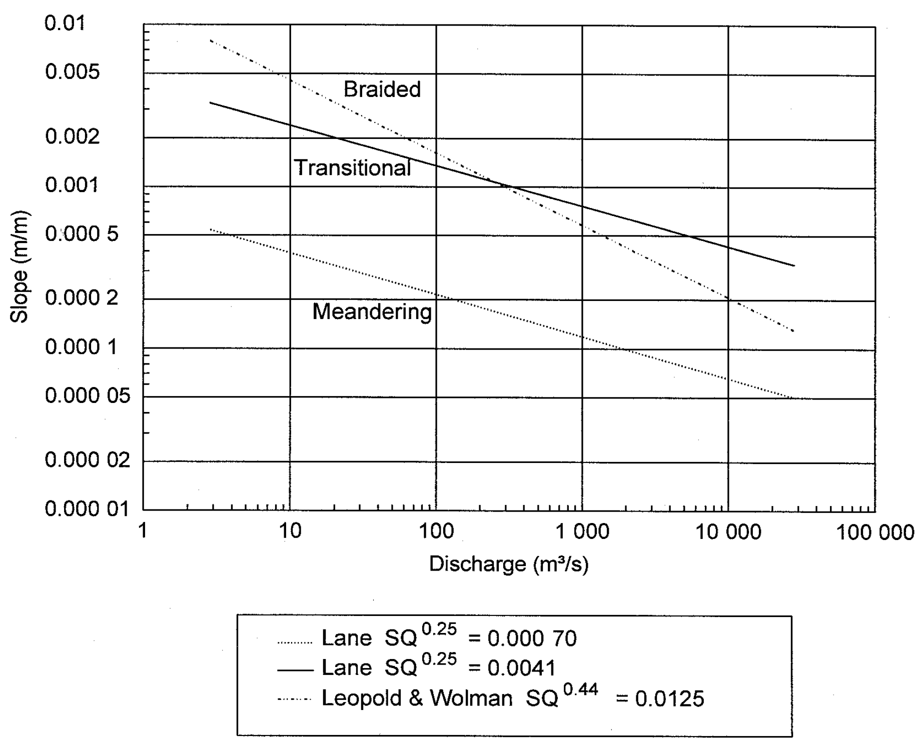


Figure 7-4 Slope-Discharge For Braiding Or Meandering Bed Streams
Source: After, Lane, 1957

7.3 Alluvial Streams (continued)

7.3.2 Stream Response To Change (continued)

The resistance of banks to erosion, an important factor in stream morphology, depends on properties of the bank material and on the vegetal cover. The sediment load carried by the stream is inter-active with the longitudinal slope and the erosion resistance of the bed and bank material. The interaction often is demonstrated by the channel plan form. If a stream lacks sufficient slope to transport the material being supplied it from the drainage basin, the channel will fill until sufficient slope is attained. Attainment of the requisite slope may be accompanied by a change in plan form from sinuous to a braided pattern.

Streams having non-resistant banks, composed mainly of sand and lacking dense vegetal cover, are usually braided and have wide shallow cross-sections. Streams with banks that are resistant because of high clay content or dense vegetal cover often are meandering streams of nearly uniform width and deep, narrow cross-section.

Table 7-5 Qualitative response of alluvial channels

Variable	Change in Variable	Effect on						
		Regime of Flow	River Form	Resistance to flow	Energy Slope	Stability of channel	Area	Stage
Discharge	+ -	+ -	M-->B B-->M	+/- +/-	- +	- +	+ -	- +
Bed Material Size	+ -	- +	M-->B B-->M	+ -	+ -	+/- +/-	+ -	+ -
Bed Load	+ -	+ -	B-->M M-->B	- +	- +	+ -	- +	- +
Wash load	+ -	+ -		- +	- +	+/- +/-	- +	- +
Viscosity	+ -	+ -		- +	- +	+/- +/-	- +	- +
Seepage Force	+ -	- +	B-->M M-->B	+ -	- +	+ -	+ -	+ -
Vegetation	+ -	- +	B-->M M-->B	+ -	- +	+ -	+ -	+ -
Wind	+ -	+ -	M-->B B-->M	- +	+ -	- -	- +	- +

7.3 Alluvial Streams (continued)

7.3.3 Guidelines for Assessing Geomorphic Factors

1.) In evaluating a site or planning countermeasures at a bridge, look for man-induced factors that may lead to problems such as aggradation, degradation, or lateral erosion. If the stream under consideration is tributary to another stream, consider also the effects of man-induced changes on the larger stream.

2.) Compare recent and older aerial photographs. Make an assessment of stream stability and behavior.

3.) Perform a field evaluation of site. Recent aerial photographs may be used as a guide. Look particularly for indications of recent bank erosion and development of bars.

7.3.4 Countermeasures

A countermeasure is defined as a measure incorporated into a highway crossing of a stream to control, inhibit, change, delay, or minimize stream and bridge stability problems. They may be installed at the time of highway construction or retrofitted to resolve stability problems at existing crossings. Retrofitting is good economics and good engineering practice in many locations because the magnitude, location, and nature of potential stability problems are not always discernible at the design stage, and indeed, may take a period of several years to develop. The selection of an appropriate countermeasure for a specific bank erosion problem is dependent on factors such as the erosion mechanism, stream characteristics, construction and maintenance requirements, potential for vandalism, and costs.

7.3.4.1 Meander Migration

The best countermeasure against meander migration is a crossing location on a relatively straight reach of the stream between bends. Other counter measures include the protection of an existing bank line, the establishment of a new flow line or alignment, and the control and constriction of channel flow. Countermeasures identified for bank stabilization and bend control are bank revetments, spurs, retardance structures, longitudinal dikes, vane dikes, bulkheads, and channel relocations. Measures may be used individually or a combination of two or more measures may be used to combat meander migration at a site (FHWA, 1990; and HEC-20, 1991).

7.3.4.2 Channel Braiding

Countermeasures used at braided streams are usually intended to confine the multiple channels to one channel. This tends to increase sediment transport capacity in the principal channel and encourage deposition in secondary channels. The measures usually consist of dikes constructed from the limits of the multiple channels to the channel over which the bridge is constructed. Spur dikes at bridge ends used in combination with revetment on highway fill slopes, riprap on highway fill slopes only, and spurs arranged in the stream channels to constrict flow to one channel have also been used successfully.

7.3 Alluvial Streams (continued)

7.3.4 Countermeasures (continued)

7.3.4.3 Degradation

Degradation in streams can cause the loss of bridge piers in stream channels, and piers and abutments in caving banks. A check dam, which is a low dam or weir constructed across a channel, is one of the most successful techniques for halting degradation on small to medium streams.

Longitudinal stone dikes placed at the toe of channel banks can be effective countermeasures for bank caving in degrading streams. Precautions to prevent outflanking, such as tiebacks to the banks, may be necessary where installations are limited to the vicinity of the highway stream crossing. In general, channel lining alone is not a successful countermeasure against degradation problems (HEC-20).

7.3.4.4 Aggradation

Current measures in use to alleviate aggradation problems at highways include channelization, bridge modification, continued maintenance, or combinations of these.

Channelization may include excavating and cleaning channels, constructing cutoffs to increase the local slope, constructing flow-control structures to reduce and control the local channel width, and constructing relief channels to improve flow capacity at the crossing. Except for relief channels, these measures are intended to increase the sediment transport capacity of the channel, thus reducing or eliminating problems with aggradation.

7.3 Alluvial Streams (continued)

7.3.5 Behavior of Alluvial Channels

7.3.5.1 Regimes of flow in alluvial sand bed channels

In alluvial streams and rivers, the material is eroded, moved and shaped by the flow so that the bed configuration and resistance to flow are a function of the flow and may change to increase or decrease the water surface level. The interaction between the flow of the water-sediment mixture and the sandbed creates different bed configurations that change the resistance to flow and the rate of sediment transport. The actual shape of the river is affected by numerous and interrelated variables.

The flow in alluvial sand bed channels is divisible into two regimes separated by a transition zone. Each regime is characterized by similarities in the

1.) shape of the bed configuration,

2.) mode of sediment transport,

3.) process of energy dissipation, and

4.) phase relation between the bed and water surface.

Lower flow regime	Transition	Upper flow regime
Ripples	ranges from dunes	Plane bed
Dunes with ripples superimposed or antidunes	to plane bed	Antidunes with a.) standing waves or b.) breaking
Dunes	antidunes	Chutes and pools

<u>Lower flow regime</u> --In the lower flow regime, resistance to flow is large and sediment transport is small. The bedform is either ripples or dunes or some combination of the two. The water-surface undulations are out of phase with the bed surface, and there is a relatively large separation zone downstream from the crest of each ripple or dune.

7.3 Alluvial Streams (continued)

7.3.5 Behavior of Alluvial Channels (continued)

Transition zone--The bed configuration in the transition zone is erratic, with the configuration during flow dependent mainly on the antecedent bed condition. Resistance to flow and sediment transport also have some variability. This variability during flow occurs because of the unstable state between bed form, resistance, and changes in depth and slope. Resistance to flow is small for flow over a plane bed; so the shear stress decreases and the bed form changes to dunes, the dunes cause an increase in resistance to flow which increases the shear stress in the bed and the dunes wash out, forming a plane bed, and the cycle continues.

Upper flow regime--In the upper flow regime, resistance to flow is small and sediment transport is large. The usual bed forms are plane bed or antidune. The water surface is in phase with the bed surface except when an antidune breaks, and the fluid does not normally separate from the boundary.

Resistance to flow is the result of grain roughness with the grains moving, of wave formation and subsidence, and of energy dissipation when the antidunes break.

7.3.5.2 Bed Configuration

The bed configurations (roughness elements) that commonly form in sand bed channels are:

(1) plane bed without sediment movement

(2) ripples

(3) ripples on dunes

(4) dunes

(5) plane bed with sediment movement

(6) antidunes

(7) chutes and pools

7.3 Alluvial Streams (continued)

7.3.5 Behavior of Alluvial Channels (continued)

These bed configurations are listed in order of their occurrence with increasing stream power for bed materials having a D50 of less than 0.6 mm. For bed materials coarser than 0.6 mm, dunes form instead of ripples after beginning of motion at small values of stream power.

The different forms of bed roughness are not mutually exclusive in time or space in a stream. Different bed-roughness elements may form side-by-side in a cross section or reach of a natural stream giving a multiple roughness; or the may form in time sequence.

Multiple roughness, the occurrence of different bed-roughness elements side-by-side in a cross-section or reach is related to variation in shear stress in a channel cross-section. The greater the width-depth ratio of a stream, the greater is the probability of a spatial variation in shear stress, stream power, or bed material. Changes in roughness over time, called variable roughness is related to changes over time in shear stress or stream power. A common example of the effect of changing shear stress or stream power is the change in bed form that occurs with changes in depth during a runoff event.

Bed configurations and their associated flow phenomena are described in the following paragraphs, in order of their occurrence with increasing stream power:

Plane bed without sediment movement

The bed configuration at levels of flow with no sediment movement is a remnant of the configuration formed when the flow was sufficient for sediment movement. Prior to beginning motion, the resistance to flow is of rigid-boundary hydraulics. After the beginning of motion, the bed configuration for flat slopes and low velocities would be either ripples, if the bed material is sand or smaller than 0.6 mm, dunes, for coarser material.

Ripples

Ripples are small triangular-shaped elements having gentle upstream slopes and steep downstream slopes. The length ranges from 0.4 to 2.0 feet and height from 0.02 ft. to 0.2 feet. The ripples result in a large resistance to flow, with Manning's "n" ranging from 0.018 to 0.030. The resistance to flow decreases as flow depth increases. The ripple shape is independent of sand size and at large values of Manning's "n", the magnitude of grain roughness is small relative to form roughness. Ripples cause very little, if any, disturbance on the water surface, and the flow contains very little suspended bed material. The bed material concentration is small, ranging from 10 to 200 ppm.

7.3 Alluvial Streams (continued)

7.3.5 Behavior of Alluvial Channels (continued)

Dunes

As the shear stress or the stream power is increased for a bed having ripples, or a plane bed for bed material coarser than 0.6mm, sand waves called dunes form on the bed. At smaller shear-stress values, the dunes have ripples superimposed on their backs. These ripples disappear at large shear values, particularly if the bed material is coarse sand with a D50 greater than 0.4 mm.

Dunes are triangular shaped elements similar to ripples. The length ranges from two feet to many hundreds of feet. The maximum amplitude to which dunes can develop is approximately the average depth. Hence the amplitude of dunes can increase with increasing depth of flow. With dunes, the relative roughness can remain essentially constant or even increase with increasing depth of flow. The resistance to flow is large, Manning's "n" ranges from 0.020 to 0.040. The form roughness for flow with dunes is equal to or larger than sand grain roughness.

Field observations indicate that dunes can form in any sand channel, irrespective of the size of the bed material, if the stream power is sufficiently large to cause transport of the bed material without exceeding a Froude number of unity. Dunes result in boils forming on the surface of the stream. The boils are a reflection of the large separation of the flow caused by the dunes. The water surface is out-of-phase with the bed surface.

Plane bed with movement

As the stream power of the flow increases further, the dunes elongate and reduce in amplitude. This bed configuration is called the transition or washed-out dunes. The next bed configuration is plane bed with movement. With coarse sands, larger slopes are required to affect the change from transition to a plane bed and result in larger velocities and larger Froude numbers. Manning's "n" for plane bed sand channels with sediment movement ranges from 0.010 to 0.013.

Antidunes

Antidunes are physically identical to dunes, however they may move upstream as well as downstream, or stay stationary. Also the water surface waves are in phase with the bed waves. Resistance to flow of antidunes depends on how often the antidunes form, the area of the stream they occupy, and the violence and frequency of their breaking. If the antidunes do not break, resistance to flow is about the same as that for flow over a plane bed. If many antidunes break, resistance to flow is larger, because the breaking waves dissipate a considerable amount of energy. With breaking waves, Manning's "n" varies from 0.012 to 0.02

7.3 Alluvial Streams (continued)

7.3.5 Behavior of Alluvial Channels (continued)

Chutes and pools

At very steep slopes, alluvial channels flow changes to chutes and pools. The resistance to flow may be very large, with Manning's "n" of 0.018 to 0.035.

Bars

In natural channels, some other bed configurations are also found. Bars are bed forms having lengths of the same order as the channel width or greater and heights comparable to the mean depth of the generating flow. Several different types of bars are observable. They are classified as:

a.) Point bars which occur adjacent to the convex banks of channel bends. Their shape may vary with changing flow conditions and motion of bed particles, but they do not move relative to the bend.

b.) Alternate bars which occur in somewhat straighter reaches of channels and tend to be distributed periodically along the reach, with consecutive bars on opposite sides of the channel. Their lateral extent is significantly less than the channel width. Alternate bars may move slowly downstream.

c.) Transverse bars may also occur in straight channels. They occupy nearly the full channel width. They occur both as isolated and periodic forms along a channel, and move slowly downstream.

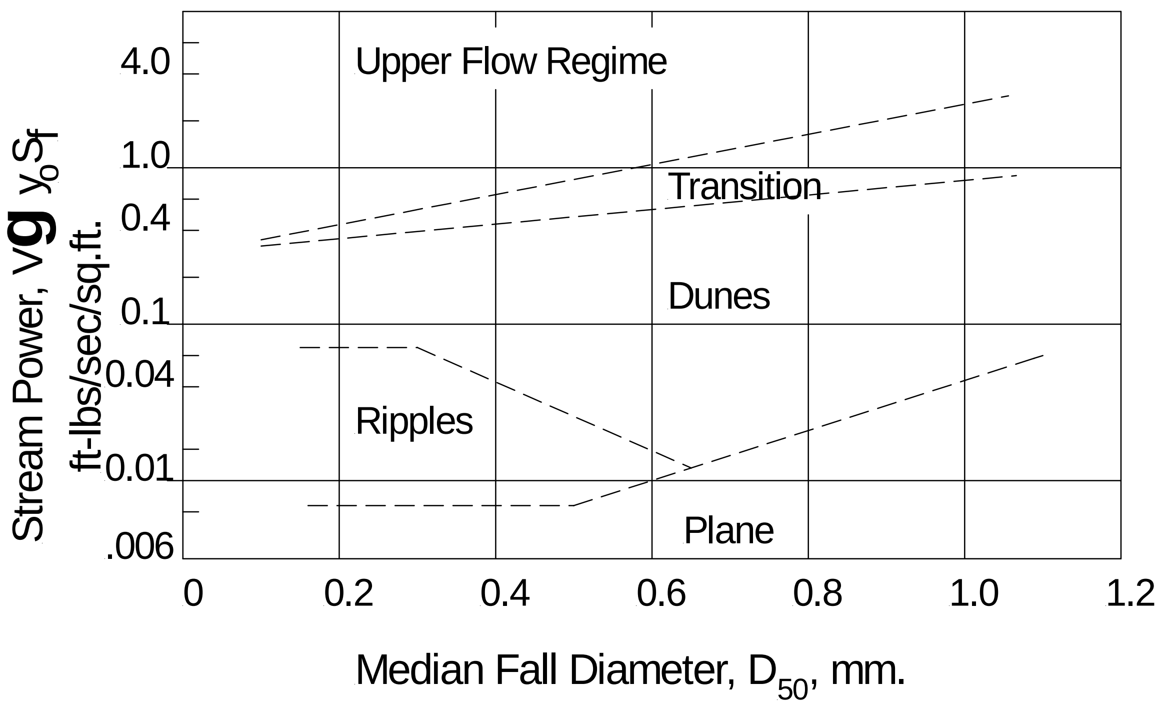
d.) Tributary bars occur immediately downstream from points of lateral inflow into a channel.

In longitudinal section, bar are approximately triangular, with very long gentle upstream slope and short downstream slopes of approximately the same as the angle of repose. Bars appear as small barren islands during low flows. Portions of upstream slope of bars are often covered with ripples or dunes.

7.3 Alluvial Streams (continued)

7.3.6 Resistance to Flow in Alluvial Channels

Resistance to flow in alluvial channels is a complex reaction affected by a large number of factors and by the interdependency of these factors. Some of the variables that describe alluvial channel flow are: velocity, depth, slope of the energy-grade line, density of water-sediment mixture, gravitational acceleration, fall diameter of the bed material, density of the sediment, shape factors of the particles, bed and fine material concentrations and the critical shear stress. The relation between stream power, median fall diameter of bed material, and form roughness is shown in figure 7.5. The relationship gives an indication of the form of the bed roughness one can anticipate if the stream power and fall diameter of bed material are known.



Lower Flow Regime	
Ripples	0.018<= n <= 0.028
Dunes	0.020<= n <= 0.040
Upper Flow Regime	
Plane Bed	0.010 <= n <= 0.013
Antidunes	
Standing Waves	0.010 <= n <= 0.015
Breaking Waves	0.012 <= n <= 0.020
Chute and Pools	0.018 <= n <= 0.035

Figure 7-5 Stream Power vs. Fall Diameter

7.3 Alluvial Streams (continued)

7.3.6 Resistance to Flow in Alluvial Channels (continued)

Bed form changes affect the impact of the stream on its boundaries. At high flows, most sandbed channel streams shift from a dune bed to a transition or a plane bed configuration. The resistance to flow is then decreased two or three fold. The corresponding increase in velocity can increase scour around bridge piers, abutments, spur dikes, or banks and increase the size of the scour protection required. Conversely an increase in resistance can result in an increase in flow depth, requiring an increase in elevation of the bridge crossing, the height of embankments, the height of any dikes, and the height of any channel control works.

A very important effect of bed forms and bars is the change in flow direction in channels. At low flows, the bars can be residual and cause high-velocity flow along or at a pier or abutment or any other structure in the streambed, causing deeper than anticipated scour. An effect of dunes on the bed is a fluctuating pattern of scour on the bed and around piers, abutments, guide banks, and spur dikes. The average height of dunes is 1/2 to 1/3 the average depth of flow, and the maximum height of a dune may approach the average depth of flow.

When analyzing the sandbed river environment, care must be used in analyzing the crossing in order to foresee possible changes that may occur in the bed form and what this change may do to the resistance coefficient, to sediment transport, and to the stability of the reach and its structures.

Alluvial processes and resistance to flow in coarse material streams

The behavior of coarse material channels is somewhat different from the sandbed channels. This includes all channels with non-cohesive bed material coarser than 2 mm size. In general, the coarse material channels are less active and slower in bank shifting than the sandbed channels. Often the bed material in the mobile bed channel is rearranged during flow resulting in armoring. The armoring phenomenon is a covering of the bed by a one particle thick layer of the coarser material underlain by the finer sizes. The absence of finer sizes from the surface layer is caused by the winnowing away of these sizes by the flows. As the range of particle sizes available in the bed of coarse-material is large, these channels can armor their beds and behave as rigid boundary channels for flows below the armoring event. The bed and bank forming activity in these channels is therefore limited to much smaller intervals of the annual hydrographs than the sandbed channels.

The general lack of mobility in coarse material channels also means the bed forms do not change as much or as rapidly as in sandbed channels. The roughness characteristics of coarse material channels are more consistent during the annual hydrographs than sandbed channels. The major component of the resistance to flow in coarse material channels comes from grain roughness and from bars; bed forms (dunes) are a lesser factor in flow resistance. Ripples never form and dunes are rare. The main type of form roughness is the pool and rifle configuration. With this type of configuration, the grain roughness is the main component of the channel roughness.

7.3 Alluvial Streams (continued)

7.3.7 Sediment Motion

The beginning and ceasing of sediment motion can be related to either the shear stress on the grains or to the fluid velocity in the vicinity of the grains. When the grains are at incipient motion, these values are called the critical stress or critical velocity.

The forces acting on an individual particle on the bed of an alluvial channel are:

1.) the body forces due to gravity,
2.) the external forces acting at the points of contact between the grain and its neighboring grains, and
3.) the fluid forces acting on the surface of the grain.

The fluid force varies with the velocity field and with the properties of the fluid. The relative magnitude of these forces determines whether the grain moves or not. The fluid forces may be divided into three components: 1.) form drag, 2.) viscous drag, and 3.) buoyant force. The form drag can be expressed in terms of the shear velocity. The viscous drag is related to the shear velocity for laminar flow. When flow over a grain is turbulent, the form drag is predominant; when the flow is laminar, the viscous shear force is predominant. In view of the bed consisting of particles of various sizes, each one having different shear stresses needed to dislodge, it is customary to consider the critical shear stress corresponding to the D50 size of the bed material as that required for beginning of motion.

7.3.8 Sediment Transport

The amount of material transported or deposited in the stream under a given set of conditions is the result of the interaction of two groups of variables. In the first group are those variables which influence the quantity and quality of the sediment brought down to that section of the stream. In the second group are variables that influence the capacity of the stream to transport that sediment.

Group I. - Sediment brought down to the stream depends on the geology and topography of the watershed; magnitude, intensity, duration, distribution, and season of rainfall, soil condition, vegetal cover, cultivation and grazing, surface erosion, and bank cutting.

Group II. - Capacity of the stream to transport sediment depends on hydraulic properties of the stream channel. These are fluid properties, slope, roughness, hydraulic radius, discharge, velocity, velocity distribution, turbulence, tractive effort, viscosity and density of the fluid, sediment mixture, and size and gradation of the sediment.

These variables are not all independent of each other, nor is their effect, in some cases, definitely known. The variables which control the amount of sediment brought down to the stream are subject to so much variation, not only between streams but at a given point of a single stream, that the analysis of any particular case in a quantitative way is extremely difficult. The variables that deal with the capacity of the stream to transport solids are subject to mathematical analysis. These variables are closely related to the hydraulic variables controlling the capacity of the streams to carry water.

7.3 Alluvial Streams (continued)

7.3.8 Sediment Transport (continued)

Sources of Sediment Transported

For engineering purposes, there are two sources of sediment transported by a stream: (1.) the bed material that make up the streambed; and (2) the fine material that comes from the banks and the watershed (wash load). Geologically both materials come from the watershed. But the distinction is important because the bed material is transported at the capacity of the stream and is functionally related to the measurable hydraulic variables. The wash load is not transported at the capacity of the stream, it is dependent on the availability and is not functionally related to measurable hydraulic variables.

Total Sediment Discharge

The total sediment discharge of a stream is the sum of the bed sediment discharge and the fine sediment (wash load) discharge, or the sum of the contact sediment discharge and suspended sediment discharge. In the former sum the total sediment discharge is based on the sources of the sediments and the latter sum is based on the mode of sediment transport. The suspended sediment load consists of both bed sediment and fine sediment (wash load), only the bed sediment discharge can be estimated by the various equations that have been developed.

There are many equations developed for the estimation of bed sediment transport. The variation between the magnitudes of the bed sediment discharge predicted by the different equations under the same conditions is tremendous. For the same discharge, the predicted sediment discharge can vary by a 100-fold difference between the smallest and the largest discharge. This can be expected given the number of variables, the interrelationships between them, the difficulty of measuring many of the variables and the statistical nature of bed material transport.

Suspended bed sediment discharge is usually computed by one of three methods. These are Meyer-Peter Muller (1948), Einstein (1950), and Colby (1961). The Meyer-Peter Muller equation is applicable to streams with little or no suspended sediment discharge and is thus used extensively for gravel and cobble bed streams. The other two methods, based to some degree on Einstein's work are used for sandbed channels.

7.3 Alluvial Streams (continued)

7.3.8 Sediment Transport (continued)

Sediment transport in coarse material channels.

The bed material load in coarse-bed channels is mostly transported as bed load and not as suspended load. For the bed-load transport, Einstein's bed-load function (without the suspended load component) and the Meyer-Peter Muller transport function may be found useful.

The time response of coarse-material channels is different from sandbed channels in the time scale of response. This time response is dominated by two factors: (1.) the difference in particle size between the surface (armor) layer of the bed and the bed material below it; and, (2) the wash load may extend to coarse sand sizes.

The formation of an armor layer on the bed may immobilize the bed for a large part of the hydrograph. However, if the conditions for incipient movement of this layer are exceeded, the underlying fine bed material will be readily picked up by the flow. Thus, extreme flow events in coarse-material channels are capable of inducing rapid and large bed-level fluctuations.

The coarse sand and larger particles may behave as wash load in coarse material channels; that is, although the flow may be transporting a large quantity of these particles, the boundary shear may be large, so that these particles are not found in appreciable quantities in the armor layer. If the boundary shear is reduced by afflux at a highway crossing, the flow may not sustain this material as wash load and rapid aggradation may occur.

When considerable constriction is imposed at a bridge crossing by the bridge approach or river training works sediment problems should be anticipated.

7.4 Principles of Open Channel Flow

7.4.1 General

Design analysis of both natural and artificial channels proceeds according to the basic principles of open channel flow (see Chow, 1970; Henderson, 1966). The basic principles of fluid mechanics -- continuity, momentum, and energy -- are applicable to open channel flow with the additional complication that the position of the free surface is usually one of the unknown variables. The determination of this unknown is one of the principle problems of open channel flow analysis and it depends on quantification of the flow resistance. Natural channels display a much wider range of roughness values than artificial channels. This section presents the application of Manning's equation and the energy equation for the computation of channel capacity.

7.4.2 Flow Classifications

7.4.2.1 Flow Classification and Determination

The classification of open channel flow can be summarized as follows.

Steady Flow

1. Uniform Flow
2. Nonuniform Flow
   a. Gradually Varied Flow
   b. Rapidly Varied Flow

Unsteady Flow

1. Unsteady Uniform Flow (rare)
2. Unsteady Nonuniform Flow
   a. Gradually Varied Unsteady Flow
   b. Rapidly Varied Unsteady Flow

The steady uniform flow case and the steady nonuniform flow case are the most fundamental types of flow treated in highway engineering hydraulics.

Steady and Unsteady Flow

Steady flow is one in which the discharge and other hydraulic properties passing a given cross-section are constant with respect to the time interval under consideration. At a section, the depth of flow does not change during the time interval under consideration. The maintenance of steady flow in any reach requires that the rates of inflow and outflow be constant and equal. The flow is unsteady when the discharge and therefore depth and velocity vary with time.

7.4 Open Channel Flow (continued)

7.4.2.1 Flow Classification and Determination (continued)

Uniform Flow and Non-uniform (Varied) Flow

For uniform flow, the depth and velocity remain constant along the length of a channel, while for non-uniform (varied) flow the velocity and depth vary along the direction of flow. Uniform flow can only occur in a prismatic channel, which is a channel of constant cross section, roughness, and slope along the flow direction; however, non-uniform flow can occur either in a prismatic channel or in a natural channel with variable properties.

Gradually-varied and Rapidly-Varied

Gradually varied, non-uniform flow is one in which the depth and velocity change gradually enough in the flow direction that vertical accelerations can be neglected. It is considered to be rapidly varied flow when the changes in velocity should not be neglected.

Steady, uniform flow is an idealized concept of open channel flow that seldom occurs in natural channels. However, for most practical highway drainage applications, the flow is steady and changes in width, depth, or direction are sufficiently small that flow can be considered uniform.

Continuity Equation

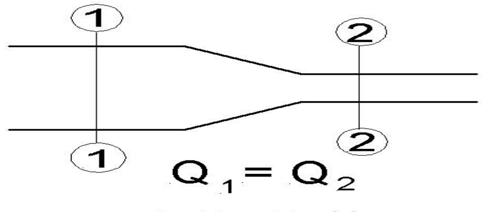
The continuity equation is the statement of conservation of mass in fluid mechanics. For the special case of steady flow of an incompressible fluid, it assumes the simple form:

$$\mathbf{Q = A_1V_1 = A_2V_2} \qquad \textbf{(7.4.1)}$$

Where:

Q = discharge, cfs
A = flow cross-sectional area, ft^2
V = mean cross-sectional velocity, ft/s (which is perpendicular to the cross section)

The subscripts 1 and 2 refer to successive cross sections along the flow path.



Continuity Equation

7.4 Open Channel Flow(continued)

7.4.2.1 Flow Classification and Determination (continued)

Manning's Equation

For a given channel geometry, slope, and roughness, and a specified value of discharge Q, a unique value of depth occurs in steady uniform flow. It is called the normal depth. It is computed indirectly from Manning's equation:

$$\mathbf{Q = (1.486/n)AR^{2/3}S^{1/2}} \qquad \textbf{(7.4.2)}$$

For a given depth of flow in a uniform channel, the mean velocity is computed using the Manning’s equation. Dividing the above equation by the area resulting in:

$$\mathbf{V = Q/A = (1.486/n)R^{2/3}S^{1/2}} \qquad \textbf{(7.4.3)}$$

Where:

Q = discharge, cfs
V = mean velocity, ft/sec
n = Manning's roughness coefficient
A = cross-sectional area of flow, ft^2
R = hydraulic radius = A/P, ft
P = wetted perimeter of flow area, ft
S = channel bed slope, ft/ft

The selection of Manning's “n” is generally based on observation; however, considerable experience is essential in selecting appropriate n values. The range of 'n' values for various types of channels and floodplains is given in Appendix 7-A, Table 7-1. R, the hydraulic radius is the ratio of flow area to the wetted perimeter. The wetted perimeter is the length along the channel cross-section where the water is in contact with the channel boundaries. It is a term that gives an indication of the hydraulic efficiency of the channel.

In channel analysis, it is often convenient to group the channel properties in a single term called the channel conveyance K:

$$\mathbf{K = (1.486/n)AR^{2/3}} \qquad \textbf{(7.4.4)}$$

then Manning's Equation can be written as:

$$\mathbf{Q = KS^{1/2}} \qquad \textbf{(7.4.5)}$$

The conveyance represents the carrying capacity of a stream cross-section based upon its geometry and roughness characteristics alone and is independent of the streambed slope. The concept of channel conveyance is useful when computing the distribution of overbank flood flows in the stream cross-section and the flow distribution through the opening in a proposed stream crossing.

7.4 Open Channel Flow (continued)

7.4.2.1 Flow Classification and Determination (continued)

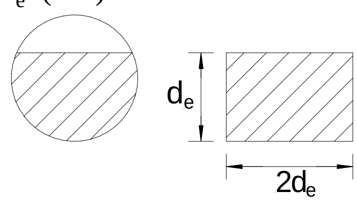
Normal Depth

In steady uniform flow for a given channel geometry, slope, and roughness, and discharge Q a unique value of depth occurs, it is called the normal depth.

Equivalent Depth

For sections that are not rectangular, it is often useful to determine hydraulic properties based on an equivalent depth of flow in a rectangular section that has a width equal to twice the depth.

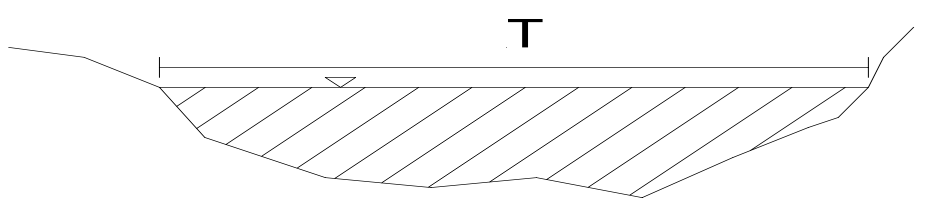
$$d_e=(A/2)^{0.5}$$



Mean Depth

The mean depth, sometimes referred to as hydraulic depth, is equal to the area of flow divided by the top width.

$$d_m={}^{A}/_{T}$$



7.4.3 Energy Principles

Total Energy Head, Energy Grade Line

Flowing water contains energy in two forms, potential and kinetic. The potential energy at a particular point is represented by the depth of the water plus the elevation of the channel bottom above a datum (elevation head). The plot of the potential energy head from one cross section to the next defines the hydraulic grade line. For open channel flow, the hydraulic grade line is coincident with the water surface. The kinetic energy is represented by the velocity head. The total energy head is the sum of potential energy head and kinetic energy head (velocity head). The plot of the total energy head from one cross section to the next defines the energy grade line.

7.4 Open Channel Flow (continued)

7.4.3 Energy Principles (continued)

Energy Equation

Written between an upstream open channel cross section designated 1 and a downstream cross section designated 2, the energy equation is:

$$\mathbf{h_1 + á_1(V_1^2/2g) = h_2 + á_2(V^2_2/2g) + h_L} \qquad \textbf{(7.4.6)}$$

Where:

h_1 and h_2 are the upstream and downstream stages, respectively, ft
á = kinetic energy correction coefficient
V = mean velocity, ft/s
h_L = head loss due to local cross-sectional changes (minor loss) as well as boundary resistance, ft

The stage h is the sum of the elevation head z at the channel bottom and the pressure head, or depth of flow y, i.e. h=z+y. The terms in the energy equation are illustrated graphically in Figure 7-6. The energy equation states that the total energy head at an upstream cross section is equal to the energy head at a downstream section plus the intervening energy head loss. The energy equation can only be applied between two cross sections at which the streamlines are nearly straight and parallel so that vertical accelerations can be neglected

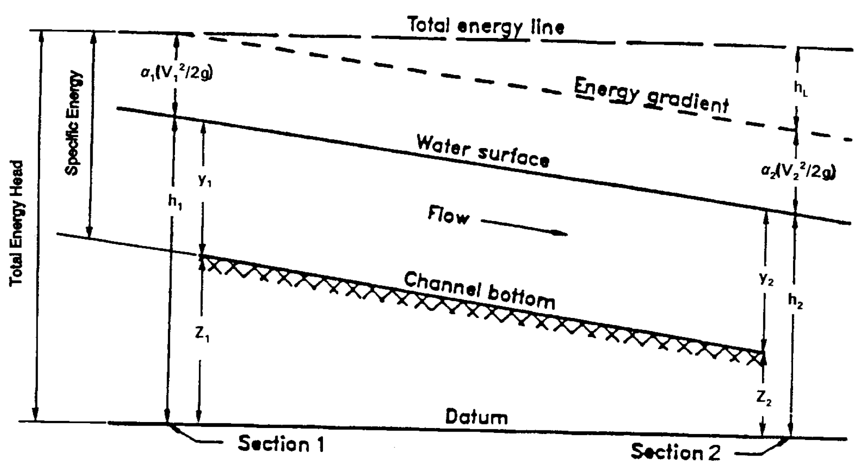


Figure 7-6 Terms In The Energy Equation

Source: FHWA, 1990

7.4 Open Channel Flow (continued)

7.4.3 Energy Principles (continued)

Specific Energy

In open channel flow, it is often desirable to consider the energy content with regards to the channel bottom. Specific energy E is defined as the energy head relative to the channel bottom. If the channel is not too steep (slope less than 10 percent) and the streamlines are nearly straight and parallel (so that the hydrostatic assumption holds), the specific energy E becomes the sum of the depth and velocity head:

$$\mathbf{E = y + á\,(V^2/2g)} \qquad \textbf{(7.4.7)}$$

Where:

y = depth, ft
á = kinetic energy correction coefficient
V = mean velocity, ft/s
g = gravitational acceleration, 32.2 ft/sec^2

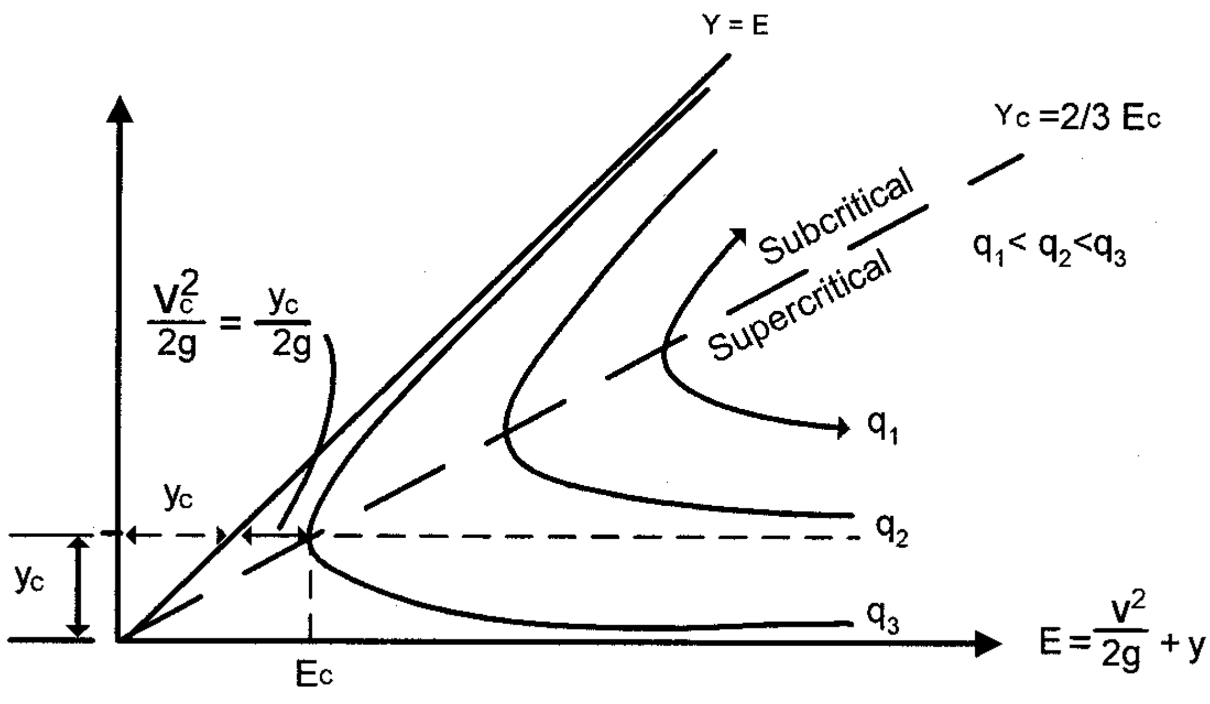


Figure 7-7 Specific Energy And Discharge Diagram For Rectangular Channels
(Adopted From Highways In The River Environment)

7.4 Open Channel Flow (continued)

7.4.3 Energy Principles (continued)

Kinetic Energy Coefficient

The kinetic energy correction coefficient is taken to have a value of one for turbulent flow in prismatic channels but may be significantly different than one in natural channels. As the velocity distribution in a river varies from a maximum at the design portion of the channel to essentially zero along the banks, the average velocity head, computed as $(Q/A)^2/2g$ for the stream at a section, does not give a true measure of the kinetic energy of the flow. A weighted average value of the kinetic energy is obtained by multiplying the average velocity head, above, by a kinetic energy coefficient, á, defined as:

$$\alpha = [\Sigma (qv^2)/(QV^2)] \qquad \textbf{(7.4.8)}$$

Where:

- v = average velocity in subsection, ft/sec
- q = discharge in same subsection, cfs
- Q = total discharge in river, cfs
- V = average velocity in river at section or Q/A, ft/sec

Froude Number

The Froude number is an important dimensionless parameter in open channel flow. It represents the ratio of inertia forces to gravity forces and is defined by:

$$F_r = V/(gd)^{0.5} \qquad \textbf{(7.4.9)}$$

Where:

- V = average velocity = Q/A, ft/sec
- g = acceleration of gravity, ft/sec^2
- d = hydraulic depth, also referred to as the mean depth, d=A/T
- A = cross-sectional area of flow, ft^2
- T = channel top width at the water surface, ft

This expression for Froude number applies to any single section channel of nonrectangular shape.

The Froude number may be used to distinguish between subcritical flow and supercritical flow.

F=1, Critical Flow
F<1, Subcritical Flow
F>1, Supercritical Flow

7.4 Open Channel Flow (continued)

7.4.3 Energy Principles (continued)

Critical Depth

The variation of specific energy with depth at a constant discharge shows a minimum in the specific energy at a depth called critical depth at which the Froude number has a value of one. Critical depth is also the depth of maximum discharge when the specific energy is held constant. If the normal depth computed from Manning's equation is greater than critical depth, the slope is classified as a mild slope, while on a steep slope, the normal depth is less than critical depth. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope. These relationships are illustrated in Figure 7-8.

Subcritical Flow

Depths greater than critical occur in subcritical flow and the Froude number is less than one. Slopes are considered mild. In this state of flow, small water surface disturbances can travel both upstream and downstream, and the control is always located downstream.

Supercritical Flow

Depths less than critical depth occur in supercritical flow and the Froude number is greater than one. Slopes are considered steep. Small water surface disturbances are always swept downstream in supercritical flow, and the location of the flow control is always upstream.

For the two types of flow, the depth, velocity, and slope characteristics vary as shown below.

	Subcritical Flow	**Supercritical Flow**
Depth	Relatively Deep	Shallow Flow
Velocity	Low Velocity	High Velocity
Slope	Mild Slope	Steep Slope

Changes in slope will result in water surface transitions. These changes can be smooth or abrupt depending on the magnitude and direction of change. The changing water surface can be classified based on the bed slope and direction of change in the depth. Figure 7-9 shows the classification of water surface profiles.

Control Section

Any cross-section for which the depth of flow can be uniquely predicted for a given discharge, most commonly at critical depth.

7.4 Open Channel Flow (continued)

7.4.3 Energy Principles (continued)

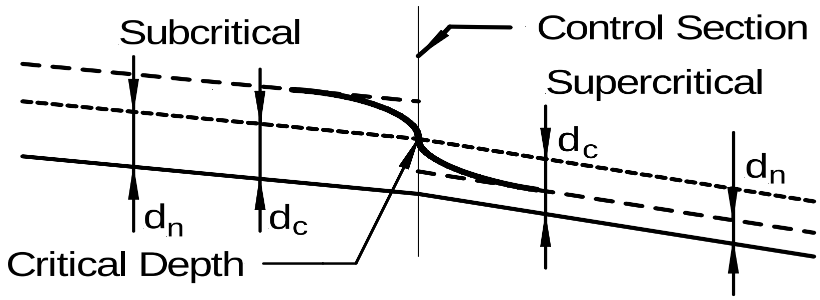


Fig. 7-8 Control Section

Table 7-6
Flow Profile Types

Classification	Class	Bed Slope	Depth	Type
M1	Mild	$S_0>0, S_0<S_c$	$Y > y_o > y_c$	1
M2	Mild	$S_0>0, S_0<S_c$	$y_o > y > y_c$	2
M3	Mild	$S_0>0, S_0<S_c$	$y > y_c > y_o$	3
C1	Critical	$S_0>0, S_0=S_c$	$y>y_o=y_c$	1
C3	Critical	$S_0>0, S_0=S_c$	$y<y_o=y_c$	3
S1	Steep	$S_0>0, S_0>S_c$	$y > y_c > y_o$	1
S2	Steep	$S_0>0, S_0>S_c$	$y_c > y > y_o$	2
S3	Steep	$S_0>0, S_0>S_c$	$y_c > y_o > y$	2
H2	Horizontal	$S_0=0$	$y=y_o>y_c$	2
H3	Horizontal	$S_0=0$	$y_c>y=y_o$	3
A2	Adverse	$S_0<0$	$y>y_o>y_c$	2
A3	Adverse	$S_0<0$	$y_c>y>y_o$	3

With a type 1 curve (M1,S1,C1), the actual depth of flow y is greater than both the normal depth y_o and the critical depth, y_c. Because flow is tranquil, control of the flow is downstream.

With a type 2 curve (M2,S2,A2, H2), the actual depth y is between the normal depth y_o and the critical depth y_c. The flow is tranquil for M2, A2, and H2 and thus the control is downstream. Flow is rapid for S2 and the control is upstream.

With a type 3 curve (M3, S3, C3, A3, and H3), the actual depth y is smaller than both the normal y_o depth and the critical depth y_c. Because the flow is rapid, the hydraulic control is upstream.

7.4 Open Channel Flow (continued)

7.4.3 Energy Principles (continued)

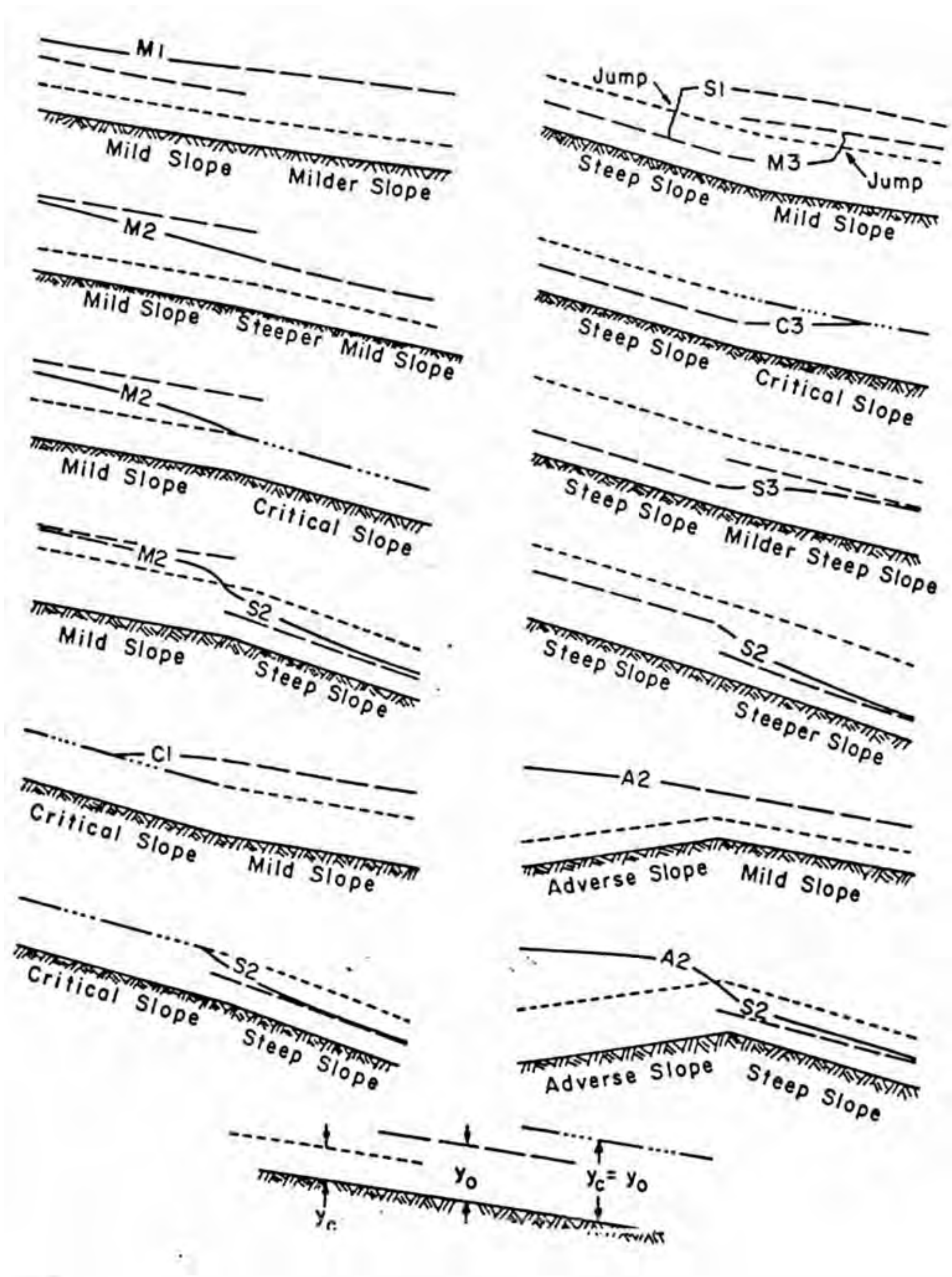


Figure 7-9 Water Surface Profiles

7.4 Open Channel Flow (continued)

7.4.3 Energy Principles (continued)

Hydraulic Jump

A hydraulic jump occurs as an abrupt transition from supercritical to subcritical flow in the flow direction. There are significant changes in depth and velocity in the jump, and energy is dissipated. For this reason, the hydraulic jump is often employed to dissipate energy and control erosion at highway drainage structures. See Figure 7-10 for a plot of the hydraulic jump diagram. The two depths associated with a hydraulic jump are related by the Momentum equation.

A hydraulic jump will not occur until the ratio of the flow depth d_1 in the approach channel to the depth d_2 in the downstream channel reaches a specific value that depends on the channel geometry. The depth before the jump is called the initial depth, d_1 and the depth after the jump is the sequent depth, d_2. When a hydraulic jump is used as an energy dissipator, controls to create sufficient tailwater depth are often necessary to control the location of the jump and to ensure that a jump will occur over the desired range of discharges. Sills can be used to control a hydraulic jump if the tailwater is less than the sequent depth. (See Chow,1970)

Momentum Equation

$$d_2/d_1=(\{1+8*F_r^2\}^{0.5}-1)^{1/2}$$

Figure 7-10 Hydraulic jump diagram.

7.5 Hydraulic Analysis

7.5.1 General

The hydraulic analysis of a channel determines the depth and velocity at which a given discharge will flow in a channel of known geometry, roughness, and slope. The depth and velocity of flow are necessary for the design or analysis of channel linings and highway drainage structures.

Two methods are commonly used in hydraulic analysis of open channels. The single-section method is a simple application of Manning's Equation to analyze situations in which uniform or nearly uniform flow conditions exist, such as tailwater rating curves for culverts. The step-backwater method is used to compute the complete water surface profile, such as for bridge hydraulics, in a stream reach to evaluate the unrestricted water surface elevations, or to analyze other gradually varied flow problems in streams.

The single-section method will generally yield less reliable results because it requires more judgment and assumptions than the step-backwater method. In many situations, however, the single-section method is all that is justified, e.g., a standard roadway ditch, culverts, storm drain outfalls, etc. The selection of the method of analysis is based on the cost of application and the risk and consequences associated with the feature/element under consideration.

7.5.2 Cross Sections

The step-backwater analysis should be used where evaluation of the impact of the project is needed upstream or downstream. Cross-sections shall extend sufficiently in either direction so that all impacts are identified.

Cross-sectional geometry of streams is defined by coordinates of lateral distance and ground elevation that locate individual ground points. The cross section is taken normal to the flow direction along a single straight line where possible, but in wide floodplains or bends it may be necessary to use a section along intersecting straight lines, i.e. a "dog-leg" section. It is especially important to make a plot of the cross section to reveal any inconsistencies or errors.

Cross sections should be located to be representative of the subreaches between them. Stream locations with major breaks in bed profile, abrupt changes in roughness or shape, control sections such as free overfalls, bends and contractions, or other abrupt changes in channel slope or conveyance will require cross-sections taken at shorter intervals in order to better model the change in conveyance.

Cross sections should be subdivided with vertical boundaries where there are abrupt lateral changes in geometry and/or roughness as in the case of overbank flows. The conveyances of each subsection are computed separately to determine the flow distribution and á, and are then added to determine the total flow conveyance. The subsection divisions must be chosen carefully so that the distribution of flow or conveyance is nearly uniform in each subsection (Davidian, 1984). Selection of cross sections and vertical subdivision of a cross section are shown in Figure 7-11.

7.5 Hydraulic Analysis (continued)

7.5.2 Cross Sections (continued)

7.5.2.1 Manning's n Value Selection

Manning's ‘n’ is affected by many factors and its selection in natural channels depends heavily on engineering experience. Resistance to flow depends on a number of factors such as bed material size, changes in channel geometry, bed forms (dunes, ripples, etc.), and vegetation type and density. Roughness will also vary with depth, decreasing as the water depth becomes much greater than the roughness elements (vegetation, bed material, bed forms, etc). Therefore the resistance to flow will vary from season to season and year to year. Because changes will occur over time, a range of roughness values should be considered and a sensitivity analysis is recommended to identify how uncertainty in roughness value affects the computed water surface elevation and/or velocity. Consideration needs to be given to the overall goal of the model. When velocity is a critical parameter (bank protection design), a roughness value on the lower end of the range should be used, and when the water surface elevation is more critical (levee design), a higher roughness value should be used.

Pictures of channels and flood plains for which the discharge has been measured and Manning's n has been calculated are very useful (see Arcement and Schneider(1984); Barnes, 1978, ADOT, MCFCD). For situations lying outside the engineer's experience, a more regimented approach is presented in Arcement and Schneider, (1984). Once the Manning's n values have been selected, it is highly recommended that they be verified with historical high-water marks and/or gaged streamflow data.

Manning's n values for artificial channels are more easily defined than for natural stream channels. See Table 7-1 in Appendix 7-A for typical n values of both artificial channels and natural stream channels.
The following publications provide information specific to Arizona streams:

- Estimated Manning’s Roughness Coefficients for Stream Channels and Floodplains in Maricopa County, Arizona (Thomsen and Hjalmarson, 1991)
- Roughness Coefficients for Stream Channels in Arizona (Albridge and Garret, 1973)
- Verification of Roughness Coefficients for Selected Natural and Constructed Stream Channels in Arizona (Philips and Ingersoll, 1998)

In dealing with vegetated flood control channels, the modeler must account for fully vegetated conditions. One reference specific to Arizona watercourses is “Method to Estimate Effects of Flow-Induced Vegetation Changes on Channel Conveyances of Streams in Central Arizona” (Phillips et al., 1998).

The designer must exercise caution when using Manning’s n based on field reconnaissance. When the modeler inspects a watercourse to judge its roughness, the physical state of the system when it is inspected is not necessarily the physical state the system would be under at design conditions. When the design condition is a rare event, the roughness of the watercourse during such an event may be drastically different than what is seen in the field. When roughness is based upon vegetative resistance, a shear stress analysis should be conducted to check if the vegetation critical

7.5 Hydraulic Analysis (continued)

7.5.2.1 Manning's n Value Selection (continued)

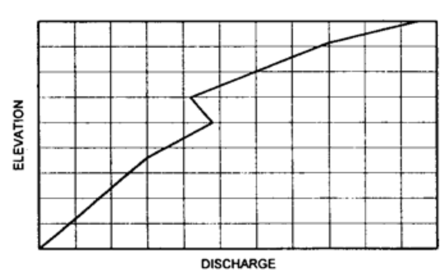
shear stress is exceeded by the actual shear stress. Conversely, if a plain streambed is observed and only the grain resistance is used to obtain the “n” value, the actual resistance may be higher under design floods because dunes or ripples may form and become important additional components in determining Manning’s n.

7.5.2.2 Calibration

The values used in the equations should be calibrated to ensure that they accurately represent local channel conditions. The following parameters, in order of preference, should be used for calibrations: Manning's n, slope, discharge, and cross-section. Proper calibration is essential if accurate results are to be obtained. Calibration is not easy, or always attainable.

7.5.2.3 Switchback Phenomenon

If the cross-section is improperly subdivided, the mathematics of the Manning's Equation causes a switchback. A switchback results when the calculated discharge decreases with an associated increase in elevation. This occurs when, with a minor increase in water depth, there is a large increase of wetted perimeter. Simultaneously, there is a corresponding small increase in cross-sectional area that causes a net decrease in the hydraulic radius from the value it had for a lesser water depth. With the combination of the lower hydraulic radius and the slightly larger cross-sectional area, a discharge is computed which is lower than the discharge based upon the lower water depth. More subdivisions within such cross-sections should be used in order to avoid the switchback.



SWITCHBACK

7.5 Hydraulic Analysis (continued)

7.5.2 Cross Sections (continued)

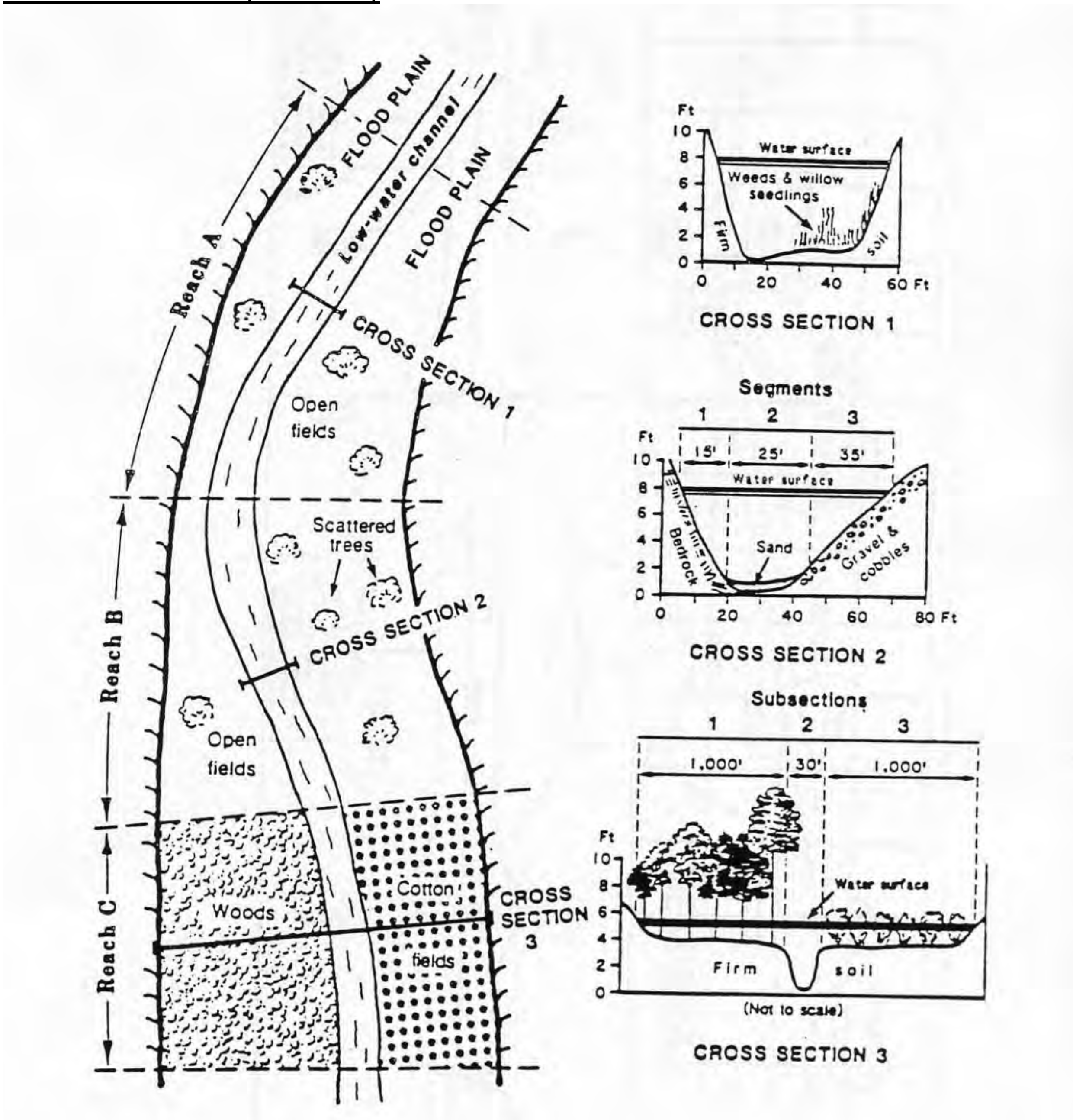


Figure 7-11 Hypothetical Cross Section Showing Reaches, Segments, And Subsections Used In Assigning n Values

Source: FHWA, 1984

7.5 Hydraulic Analysis (continued)

7.5.2.3 Switchback Phenomenon (continued)

This phenomenon can occur in any type of conveyance computation, including the step-backwater method. Computer logic can be seriously confused if a switchback were to occur in any cross-section being used in a step-backwater program. For this reason, the cross-section should always be subdivided with respect to both vegetation and geometric changes. Note that the actual n-value, itself, may be the same in adjacent subsections.

7.5.2.4 Bends

Bends are not usually directly considered in the analysis of water surface. For bends with a radius of curvature to active channel width less than 10, Cowan (1956) presents suggestions on adjusting the Manning's n. Changing the expansion and contraction coefficients as a means to account for bends is not appropriate. A superelevated water surface should be considered for artificial channels and levees and for determining increased shear stresses on stream bank protection measures.

$$\text{Delta } S = \frac{V^2 w}{g R_c}$$

where S = superelevation, ft.
V = mean channel velocity, ft/sec
W = top channel width, ft
G = gravity, 32,2 ft/sec^2
R_c = radius of centerline of channel, ft.

Curved alignment,

Subcritical: $R_c > 3w$, If $R_c > 10w$ consider as straight.

Supercritical: $R_c > \frac{4(V^2 w)}{gy}$

7.5.2.5 Modeling of depressions (Pit Areas and Sand & Gravel Mining)

Natural or man-made depressions can be situated in-channel or off-channel. One cause may be sand and gravel mining extraction operations. For modeling pits, an appropriate number of cross-sections should be selected to properly represent the change in pit dimensions. At a minimum, there should be a cross-section at the beginning and end of the pit with an additional cross-section located at the widest part of the pit. The pit will then be represented as a diamond shape in planform.

7.5 Hydraulic Analysis (continued)

7.5.2.5 Modeling of depressions, (Pit Areas and Sand & Gravel Mining) (continued)

In-channel pits

With regard to a hydraulic study involving in-channel pit, an important factor influencing the type of model is the size of the pit. If the pit is very large in comparison to the average channel size, storage and its effect on flow attenuation will be a major consideration. In that case, an unsteady flow analysis considering the volume of water in the hydrograph may be the best choice.

On the other hand, the purpose of the study may govern the kind of model selected. If the purpose is to delineate the floodplain, it might be of interest to compute the most conservative (maximum) water surface elevations. In such a case, a steady-flow model can be used and the water surface computed by ignoring the pit (filling in the pit) in the cross-section. The filled-in area should have a Manning's n similar to the areas upstream and downstream of the pit.

Off-channel pits

For off-channel pits, the major consideration is whether the stream flow will be diverted into the pit. This is affected by the presence of levees, the size of the pit, and the magnitude and duration of the flood. If levees separate the main channel for the pits and will hold during the flood under consideration, the area beyond the levees can be ignored or blocked off. If the levees would fail, or if the flood would overtop the levees, two situations need to be considered.

In one situation, the pit area will be storing water but not actively conveying flow downstream. In this case the pit should be considered as an ineffective flow area. In the second situation, the pit area is actively conveying flow and will have to be modeled as a part of the overbank flow path.

7.5.3 Single Section Analysis

The single-section analysis method (slope-area method) is simply a solution of Manning's Equation for the normal depth of flow given the discharge and cross-section properties including geometry, slope, and roughness. It implicitly assumes the existence of steady, uniform flow; however, uniform flow rarely exists in either artificial or stream channels. Nevertheless, the single-section method is often used to design artificial channels for uniform flow as a first approximation, and to develop a stage-discharge rating curve in a stream channel for tailwater determination at a culvert or storm drain outlet.

A stage-discharge curve is a graphical relationship of streamflow depth or elevation to discharge at a specific point on a stream. The stage-discharge curve can be determined as follows:

- Select the typical cross-section at or near the location where the stage-discharge curve is needed.
- Subdivide cross-section and assign n-values to subsections as described in Section 7.5.2.1

7.5 Hydraulic Analysis (continued)

7.5.3 Single Section Analysis (continued)

- Estimate water-surface slope. Since uniform flow is assumed, the average slope of the streambed can usually be used.

- Apply a range of incremental water surface elevations to the cross-section.

- Calculate the discharge using Manning's equation for each incremental elevation. A graphical technique such as that given in Figure 7-12 or a nomograph as in Figure 7-13 can be used for trapezoidal and prismatic channels. For non-prismatic channels, the channel is subdivided into sections; the total discharge at each elevation is the sum of the discharges from each subsection at that elevation. In determining hydraulic radius, the wetted perimeter should be measured only along the solid boundary of the cross-section and not along the vertical water interface between subsections.

- After the discharge has been calculated at several incremental elevations, a plot of stage versus discharge can be made. This plot is the stage-discharge curve and it can be used to determine the water-surface elevation corresponding to the design discharge or other discharge of interest.

An example application of the stage-discharge curve procedure is presented in Appendix B.

In stream channels the transverse variation of velocity in any cross-section is a function of subsection geometry and roughness and may vary considerably from one stage and discharge to another. It is important to know this variation for purposes of designing erosion control measures and locating relief openings in highway fills, for example. The best method of establishing transverse velocity variations is by current meter measurements. If this is not possible, the single-section method can be used by dividing the cross section into subsections of relatively uniform roughness and geometry. It is assumed that the energy grade line slope is the same across the cross section so that the total conveyance K_T of the cross section is the sum of the subsection conveyances. The total discharge is then $K_T S^{1/2}$ and the discharge in each subsection is proportional to its conveyance. The velocity in each subsection is obtained from the continuity equation, $V = Q/A$.

Alluvial channels present a more difficult problem in establishing stage-discharge relations by the single-section method because the bed itself is deformable and may generate bed forms such as ripples and dunes in lower regime flows. These bed forms are highly variable with the addition of form resistance, and selection of a value of Manning's n is not straightforward. Instead, several methods outlined in (Vanoni, 1977) have been developed for this case (Einstein-Barbarossa; Kennedy-Alam-Lovera; and Engelund) and should be followed unless it is possible to obtain a measured stage-discharge relation.

7.5 Hydraulic Analysis (continued)

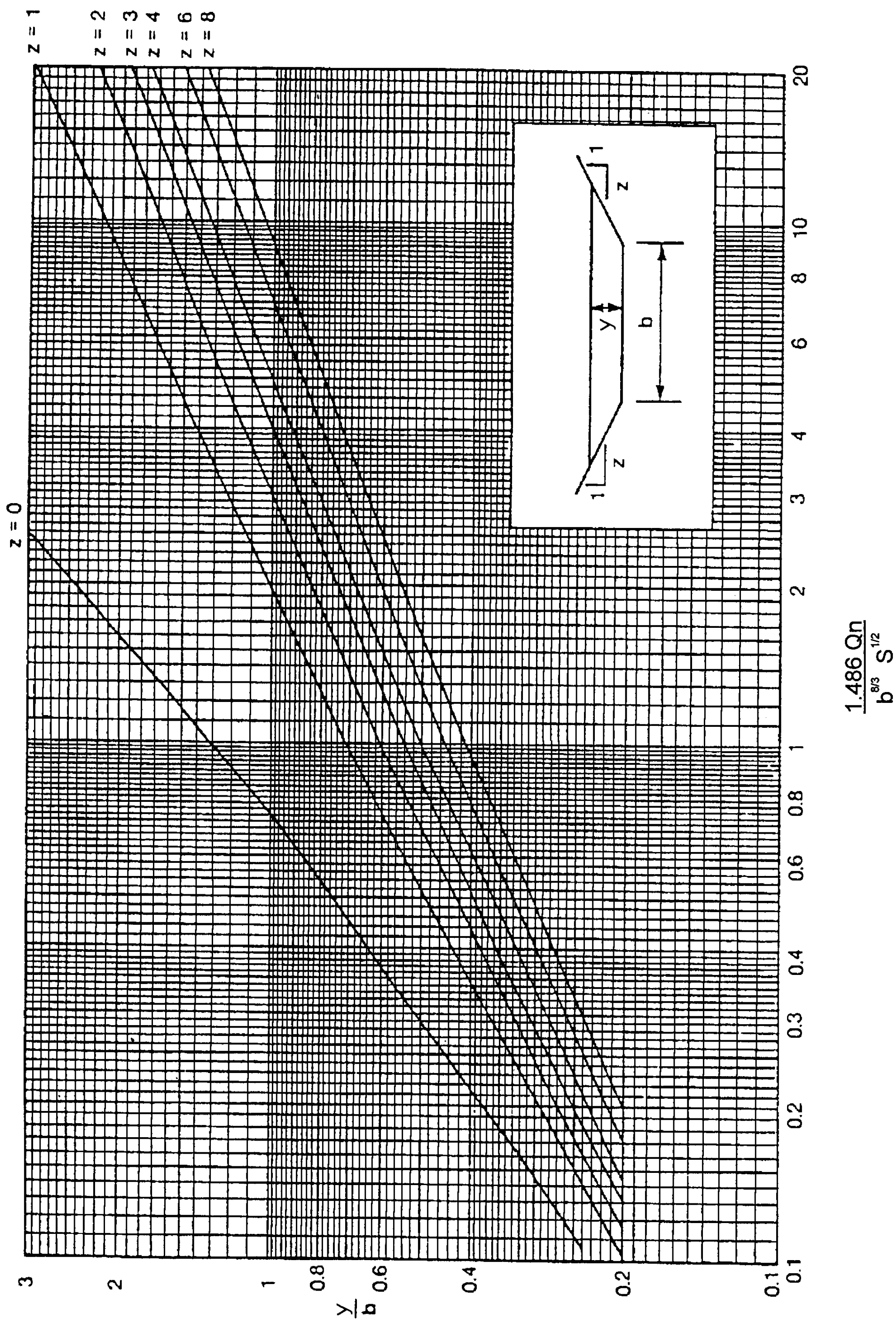


Figure 7-12 Trapezoidal Channel Capacity Chart

7.5 Hydraulic Analysis (continued)

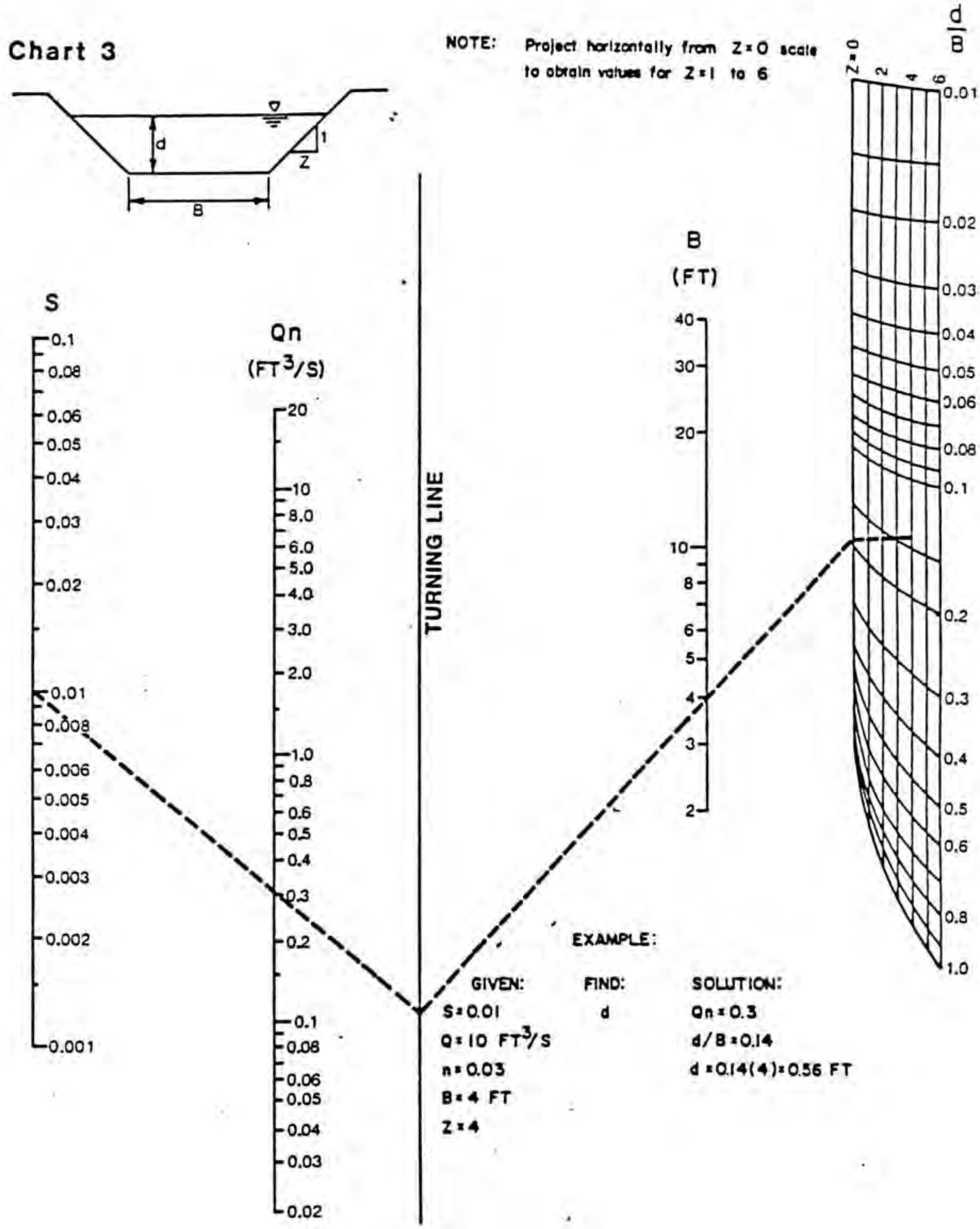


Figure 7-13 Nomograph For Normal Depth

Source: HEC-15

7.5 Hydraulic Analysis (continued)

7.5.4 Step-Backwater Analysis

Step-backwater analysis is useful for determining unrestricted water surface profiles where a highway crossing is planned, and for analyzing how far upstream the water surface elevations are affected by a culvert or bridge. Because the calculations involved in this analysis are tedious and repetitive, it is recommended that a computer program such as the US Army Corps of Engineers (USACOE) HEC-RAS be used.

7.5.4.1. Step-Backwater Models

The HEC-RAS program developed by the USA Corps of Engineers is widely used for calculating water surface profiles for steady gradually varied flow in a natural or man-made channel. This program is for fixed bed, one-dimensional flow. Both subcritical and supercritical flow profiles can be calculated. The effects of bridges, culverts, weirs, and structures in the floodplain may be also considered in the computations. This program is also designed for application in flood plain management and flood insurance studies.

7.5.4.2 Step-Backwater Methodology

The computation of water surface profiles by HEC-RAS is based on the standard step method in which the stream reach of interest is divided into a number of subreaches by cross-sections spaced such that the flow is gradually varied in each subreach. The energy equation is then solved in a step-wise fashion for the stage at one cross-section based on the stage at the previous cross-section.

The method requires definition of the geometry and roughness of each cross section as discussed in Section 7.5.1. Manning's n values can vary both horizontally across the section as well as vertically. Expansion and contraction head loss coefficients, variable main channel and overbank flow lengths, and the method of averaging the slope of the energy grade line can all be specified.

To amplify on the methodology, the energy equation is repeated from Section 7.4.4:

$$\mathbf{h_1 + á_1(V_1^2/2g) = h_2 + á_2(V_2^2/2g) + h_L} \qquad \mathbf{(7.5.1)}$$

Where:

h_1 and h_2 are the upstream and downstream stages, respectively, ft
á = kinetic energy correction coefficient
V = mean velocity, ft/s
h_L = head loss due to local cross-sectional changes (minor loss) as well as boundary resistance, ft

7.5 Hydraulic Analysis (continued)

7.5.4.2 Step-Backwater Methodology (continued)

The stage h is the sum of the elevation head z at the channel bottom and the pressure head, or depth of flow y, i.e., h = z+y. The energy equation is solved between successive stream reaches with nearly uniform roughness, slope, and cross-sectional properties.

The total head loss is calculated from:

$$h_L = K_m[(\acute{a}_1 V_1^2/2g) - (\acute{a}_2 V_2^2/2g)] + S_e L \qquad (7.5.2)$$

Where:

K_m = the minor loss coefficient
S_e = the mean slope of the energy grade line evaluated from Manning's equation and a selected averaging technique (Shearman, 1990 and HEC-RAS), ft/ft

These equations are solved numerically in a step-by-step procedure called the Standard Step Method from one cross-section to the next.

The default values of the minor loss coefficient K_m are 0.1 for contractions and 0.3 for expansions in HEC-RAS.

7.5.4.3 Profile Computation

Water surface profile computation requires a beginning value of elevation or depth (boundary condition) and proceeds upstream for subcritical flow and downstream for supercritical flow. In the case of supercritical flow, critical depth is often the boundary condition at the control section, but in subcritical flow, uniform flow and normal depth may be the boundary condition. The starting depth in this case can either be found by the single-section method (slope-area method) or by computing the water surface profile upstream to the desired location for several starting depths and the same discharge. These profiles should converge toward the desired normal depth at the control section to establish one point on the stage-discharge relation. If the several profiles do not converge, then the stream reach may need to be extended downstream, or a shorter cross-section interval should be used, or the range of starting water-surface elevations should be adjusted. In any case, a plot of the convergence profiles can be a very useful tool in such an analysis (see Figure 7-14). Given a long enough stream reach, the water surface profile computed by step-backwater will converge to normal depth at some point upstream for subcritical flow. Establishment of the upstream and downstream boundaries of the stream reach is required to define limits of data collection and subsequent analysis. Calculations must begin sufficiently far downstream to assure accurate results at the desired locations, and continued a sufficient distance upstream to accurately determine the impact of the conditions under considerations on upstream water surface profiles (see Figure 7-15).

7.5 Hydraulic Analysis (continued)

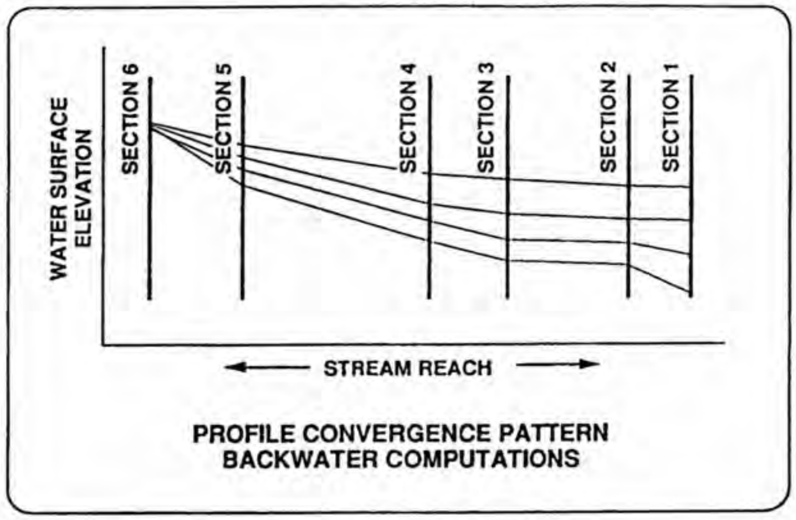


Figure 7-14 Profile Convergence Pattern Backwater Computation

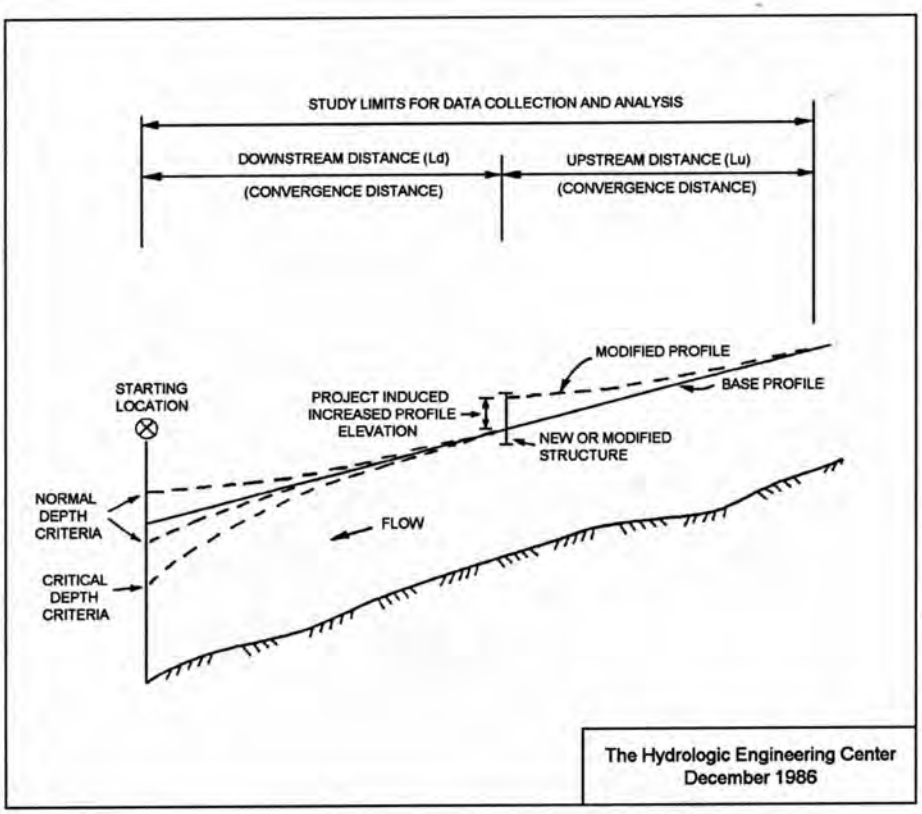


Figure 7-15 Profile Study Limits

Source: USACOE, 1986

7.5 Hydraulic Analysis (continued)

7.5.4.3 Profile Computation (continued)

The USA Corps of Engineers (USACOE, 1986) developed equations for determining upstream and downstream reach lengths as follows:

$$\mathbf{Ldn = 8{,}000\ (HD^{0.8}/S)} \tag{7.5.3}$$

$$\mathbf{Lu = 10{,}000\ [(HD^{0.6})(HL^{0.5})]/S} \tag{7.5.4}$$

Where:

Ldn = downstream study length (along main channel), ft (for normal depth starting conditions)

Lu = estimated upstream study length (along main channel), ft (required for convergence of the modified profile to within 0.1 feet of the base profile)

HD = average hydraulic depth (1-percent chance event flow area divided by the top width), ft

S = average reach slope, ft/mile

HL = headloss ranging between 0.5 and 5.0 feet at the channel crossing structure for the 1-percent chance flood, ft

References (Davidian, 1984 and USACOE, 1986) are very valuable sources of additional guidance on the practical application of the step-backwater method to highway drainage problems involving open-channels. These references contain more specific guidance on cross-section determination, location, and spacing and stream reach determination. Reference (USACOE, 1986) investigates the accuracy and reliability of water surface profiles related to n-value determination and the survey or mapping technology used to determine the cross-section coordinate geometry.

7.5.5 Modeling and Review Guidance

The following suggestions are to be considered in developing and reviewing HEC-RAS water surface profile models.

Review the warning messages, notes and cautions. Identify any need for additional cross sections. Check the graphical and tabular output. Look for unexpected results in water surface profile, velocity, flow distribution, top width, divided flow and length between cross sections including main channel and overbanks.

The cross-section spacing should be determined based on the change in slope of the ground profile of the watercourse as well as the change in width of the watercourse. In general the cross section spacing is determined by the need to adequately account for the energy losses (friction, flow expansion, and flow contraction) between consecutive cross sections.

7.5 Hydraulic Analysis (continued)

7.5.5 Modeling and Review Guidance (continued)

A review of the energy slope should be undertaken to see if rapid changes occur. If the slope increases or decreases rapidly between consecutive cross sections it usually indicates the need for decreasing the cross-section spacing by adding additional cross sections.

The use of critical depth at an isolated cross-section may indicate an error in geometry of the cross section. If no coding error is found, this may be due to a large change in energy slope. This may indicate a need for additional cross sections or that ineffective flow areas need to be defined. Several cross sections with critical flow may be an indication of supercritical flow. This may require a mixed flow run in HEC-RAS. If this is a FEMA model, this result may be computationally adequate, as FEMA does not permit the use of supercritical profiles in alluvial channels.

A drastic change in top width may require a check of flow paths. Again, additional cross sections may be necessary. The sudden change in top width may also indicate an error in the coding of levees, not specifying areas of ineffective flow, or blocked obstructions.

A large change in the distribution of flow in the channel and overbanks may indicate a need for additional sections.

Indications of divided flow should be checked for consistency with the topography. The divided flow should be hydraulically connected to be run in a single model.

Skewed cross sections should be corrected by using the projected length. The projected length is found by projecting the skewed length onto a plane perpendicular to the direction of flow. In some cross sections the skew may apply only to a part of the cross section such as the channel, and not the overbanks.

The output should be reviewed for warnings about vertical extensions of the ends of the cross section. These warnings occur when the computed water surface elevation exceeds the ends of the cross section. The cross section should be extended based on topographical data as appropriate.

Multiple profile runs should be checked to ensure that there are no conflicts in the modeling requirements for the various profiles. For example, the definition of ineffective flow areas and roughness values for modeling a low flow situation may not be applicable for a high flow situation.

7.6 Design Procedure

7.6.1 General

The design procedure for channels involves two parts. The first part involves the computation of the channel section that carries the design discharge. The second part involves evaluating the degree of protection required for a desirable maintenance and stability performance. The capacity analysis is performed using the principles of flow and Manning's equation. The stability is determined by comparison of the predicted velocity with the permissible velocity for the type of channel lining to be used.

The design procedure for all types of channels has some common elements as well as some substantial differences. This section will outline a process for assessing a natural stream channel and a more specific design procedure for roadside channels.

7.6.2 Stream Channels

The analysis of a stream channel in most cases is in conjunction with the design of a highway hydraulic structure such as a culvert or bridge. In general, the objective is to convey the water along or under the highway in such a manner that will not cause damage to the highway, stream, or adjacent property. An assessment of the existing channel is usually necessary to determine the potential for problems that might result from a proposed action. The detail of studies necessary should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream and adjoining flood plain (see Section 7.2).

Although the following step-by-step procedure may not be appropriate for all possible applications, it does outline a process that will usually apply.

<u>Step 1 Assemble site data and project file.</u>

A. Data Collection.
- Topographic, site, and location maps
- Roadway profile
- Photographs
- Field reviews
- Design data at nearby structures
- Gaging records

B. Studies by other agencies.
- Flood insurance studies
- Floodplain studies
- Watershed studies

7.6 Design Procedure (continued)

7.6.2 Stream Channel (continued)

C. Environmental constraints.
- Floodplain encroachment
- Floodway designation
- Habitat
- Commitments in review documents

D. Design criteria.
- See Section 7.3.

Step 2 Determine the project scope.

A. Determine level of assessment.
- Stability of existing channel
- Potential for damage
- Sensitivity of the stream

B. Determine type of hydraulic analysis.
- Qualitative assessment
- Single-section analysis
- Step-backwater analysis

C. Determine additional survey information.
- Extent of streambed profiles
- Locations of cross sections
- Elevations of flood-prone property
- Details of existing structures
- Properties of bed and bank materials

Step 3 Evaluate hydrologic variables.

A. Compute discharges for selected frequencies.

Step 4 Perform hydraulic analysis.

A. Single-section analysis (7.5.3).
- Select representative cross section (7.5.2)
- Select appropriate n values (Table 7-1)
- Compute stage-discharge relationship

7.6 Design Procedure (continued)

7.6.2 Stream Channel (continued)

Step 4 Perform hydraulic analysis. (continued)

B. Step-backwater analysis (7.5.4).

C. Calibrate with known high water.

Step 5 Perform stability analysis.

A. Geomorphic factors. (long-term degradation, low-flow incisement)

B. Hydraulic factors. (bed forms, bend scour)

C. Stream response to change.

Step 6 Design countermeasures.

A. Criteria for selection.
- Erosion mechanism
- Stream characteristics
- Construction and maintenance requirements
- Vandalism considerations
- Cost

B. Types of countermeasures.
- Meander migration countermeasures
- Bank stabilization (Bank Protection Chapter)
- Bend control countermeasures
- Channel braiding countermeasures
- Degradation countermeasures
- Aggradation countermeasures

C. For additional information.
- HEC-20 Stream Stability
- Highways in the River Environment
- See Reference List

Step 7 Documentation.
- Prepare report and file with background information.

7.6 Design Procedure (continued)

7.6.3 Roadside Channels

A roadside channel is defined as an open channel usually paralleling the highway embankment and within the limits of the highway right-of-way. It is normally trapezoidal or V-shaped in cross section and often lined with grass or a special protective lining.

The primary function of roadside channels is to collect surface runoff from the highway and areas that drain to the right-of-way and to convey the accumulated runoff to an acceptable outlet point.

A secondary function of a roadside channel may be to drain subsurface water from the base of the roadway to prevent saturation and loss of support for the pavement or to provide a positive outlet for subsurface drainage systems such as pipe underdrains.

The alignment, cross section, and grade of roadside channels are usually constrained to a large extent by the geometric and safety standards applicable to the project. These channels should accommodate the design runoff in a manner that considers the safety of motorists, minimizes future maintenance, damage to adjacent properties, and adverse environmental or aesthetic effects.

7.6.3.1 Step-By-Step Procedure

Each project is unique, but the following six basic design steps are normally applicable:
In order to obtain the optimum roadside channel system design, it may be necessary to make several trials of the procedure before a final design is achieved.

<u>Step 1 Establish a roadside plan.</u>

A. Collect available site data.

B. Obtain or prepare existing and proposed plan-profile layout including highway, culverts, bridges, and other elements which affect the design.

C. Determine and plot on the plan the locations of natural basin divides and roadside channel outlets. An example of a roadside channel plan/profile is shown in Figure 7.16.

D. Perform the layout of the proposed roadside channels to minimize diversion flow lengths.

7.6 Design Procedure (continued)

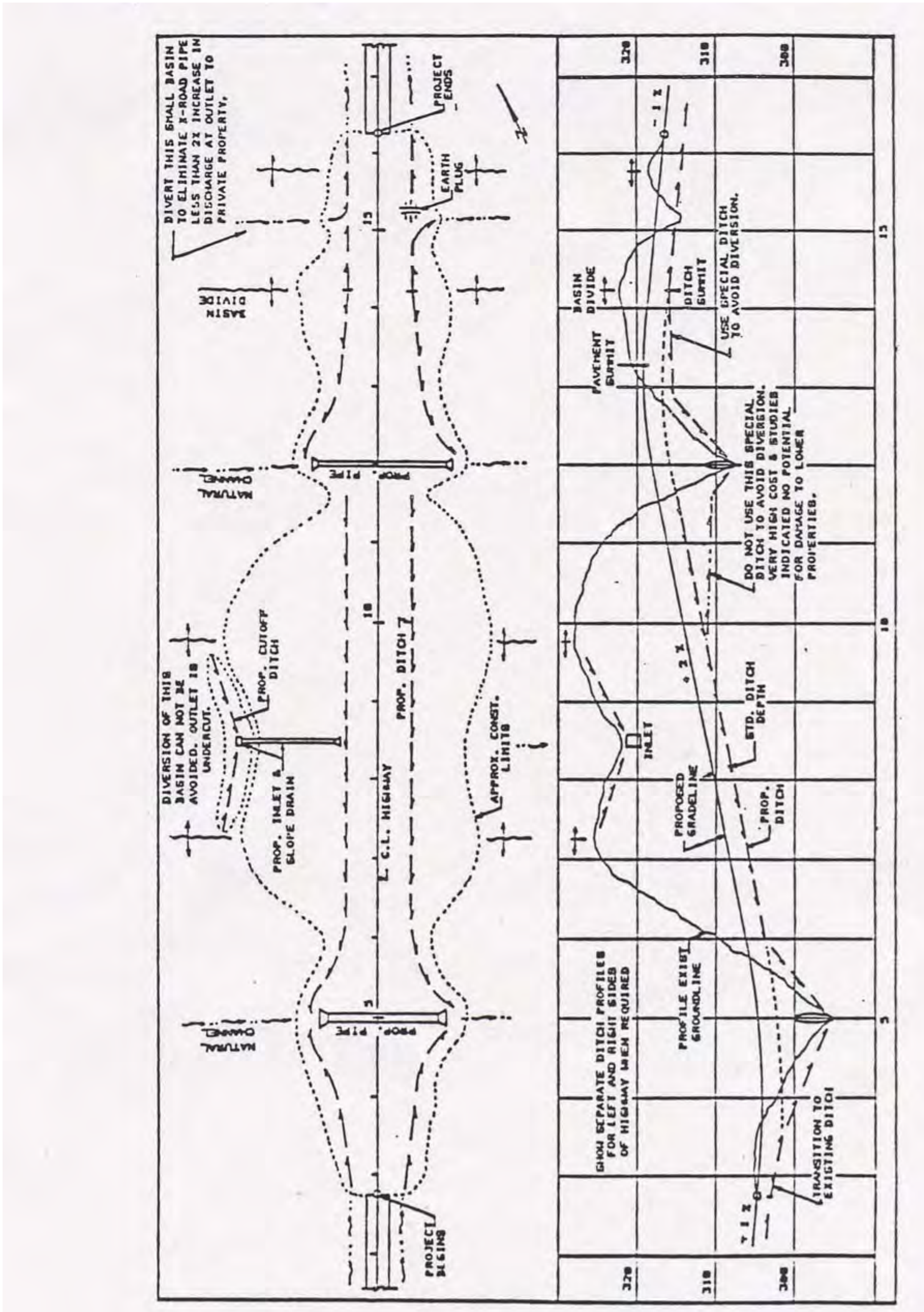


Figure 7-16 Sample Roadside Channel

7.6 Design Procedure (continued)

7.6.3.1 Step-By-Step Procedure (continued)

Step 2 Obtain or establish cross section data.

A. Provide channel depth adequate to drain the subbase and minimize freeze-thaw effects.

B. Choose channel side slopes based on geometric design criteria including safety, economics, soil, aesthetics, and access.

C. Establish bottom width of trapezoidal channel.

D. Identify features that may restrict cross section design:
- right-of-way limits,
- trees or environmentally sensitive areas,
- utilities, and
- existing drainage facilities.

Step 3 Determine initial channel grades.

A. Plot initial grades on plan-profile layout. (Slopes in roadside ditches in cut are usually controlled by highway grades.)

B. Provide minimum grade of 0.3% to minimize ponding and sediment accumulation.

C. Consider influence of type of lining on grade.

D. Where possible, avoid features that may influence or restrict grade, such as utility locations.

Step 4 Check flow capacities and adjust as necessary.

A. Compute the design discharge at the downstream end of a channel segment.

B. Set preliminary values of channel size, roughness coefficient, and slope.

C. Determine maximum allowable depth of channel including freeboard.

7.6 Design Procedure (continued)

7.6.3.1 Step-By-Step Procedure (continued)

Step 4 (continued)

D. Check flow capacity using Manning's Equation and single-section analysis.

E. If capacity is inadequate, possible adjustments are as follows:
- increase bottom width,
- make channel side slopes flatter,
- make channel slope steeper,
- provide smoother channel lining, and
- install drop inlets and a parallel storm drain pipe beneath the channel to supplement channel capacity.

F. Provide smooth transitions at changes in channel cross sections.

G. Provide extra channel storage where needed to replace floodplain storage and/or to reduce peak discharge.

Step 5 Determine channel lining/protection needed (HEC-15).

More details on channel lining design may be found in HEC-15 including consideration of channel bends, steep slopes, and composite linings.

A. Select a lining and determine the permissible shear stress $\hat{o}_p$ in lbs/ft^2 from Table 7C-1 and/or Table 7D-1.

B. Estimate the flow depth and choose an initial Manning's n value from Table 7A-1 or from Table 7B-1.

C. Calculate normal flow depth y_o (ft) at design discharge using Manning's Equation and compare with the estimated depth. If they do not agree, repeat steps 5B and 5C.

D. Compute maximum shear stress at normal depth as:

$$\hat{o}_d \text{ (lbs/ft}^2) = 62.4\, y_o\, S$$

7.6 Design Procedure (continued)

7.6.3.1 Step-By-Step Procedure (continued)

Step 5 (continued)

E. If $\hat{o}_d < \hat{o}_p$ then lining is acceptable. Otherwise consider the following options:

- decrease slope in combination with drop structures if necessary,
- increase channel width and/or flatten side slopes.
- choose a more resistant lining, such as concrete, gabions, or other more rigid lining either as full lining or composite,

Step 6 Analyze outlet points and downstream effects.

A. Identify any adverse impacts to downstream properties that may result from one of the following at the channel outlet:

- increase or decrease in discharge,
- increase in velocity of flow,
- confinement of sheet flow,
- change in outlet water quality, or
- diversion of flow from another watershed.

B. Mitigate any adverse impacts identified in 6A. Possibilities include:

- enlarge outlet channel and/or install control structures to provide detention of increased runoff in channel,
- install velocity control structures,
- increase capacity and/or improve lining of downstream channel,
- install sophisticated weirs or other outlet devices to redistribute concentrated channel flow, and
- install sedimentation/infiltration basins,
- eliminate diversions which result in downstream damage and which cannot be mitigated in a less expensive fashion.

7.6.3.2 Channel Layout Considerations

- A minimum bottom width with access for maintenance equipment of 8 feet may be necessary.

- Turnaround points may be required.

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Appendix 7-A Manning's n, Natural Channels

Table 7A-1

Manning's Roughness Coefficient, n
UNIFORM FLOW

UNLINED CHANNLES

Type Of Channel and Description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
a. Earth, straight and uniform	0.016	0.018	0.020
1. Clean, recently completed	0.018	0.022	0.025
2. Clean, after weathering	0.022	0.025	0.030
3. Gravel, uniform section, clean	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense Weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.025	0.030	0.035
5. Stony bottom and weedy sides	0.025	0.035	0.045
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140

Appendix 7-A Manning's n, Natural Channels

Table 7A-1 (continued)

Manning's Roughness Coefficient, n
UNIFORM FLOW

Type Of Channel and Description	Minimum	Normal	Maximum
NATURAL STREAMS			
1. Minor streams (top width at flood stage < 100 ft)			
a. Streams on Plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and some stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stands of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
2. Flood Plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160

Appendix 7-A Manning's n, Natural Channels

Table 7A-1 (continued)

Manning's Roughness Coefficient, n
UNIFORM FLOW

Type Of Channel and Description	Minimum	Normal	Maximum
NATURAL STREAMS			
2. Flood Plains			
d. Trees			
1. Dense Willows, summer, straight	1.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of spouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flooded stage reaching branches	0.100	0.120	0.160

Type Of Channel and Description	Minimum	Normal	Maximum
3. Major Streams (top width at flood stage > 100 ft). The n value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	.....	0.060
b. Irregular and rough section	0.035	.....	0.100

Appendix 7-B Manning's n, Channel Linings

Table 7B-1
Manning's Roughness Coefficient, n (Uniform Flow)

Lining Category	Lining Type	Depth Ranges 0 - 0.5 ft.	0.5-2.0 ft.	>2.0 ft
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Rock Riprap	6-inch D_{50}	0.104	0.069	0.035
	12-inch D_{50}	-	0.078	0.040
Gravel Riprap	1-inch D_{50}	0.044	0.033	0.030
	2-inch D_{50}	0.066	0.041	0.034
	Wire-tied			
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021

Note: Values listed are representative values for the respective depth ranges. Manning's roughness coefficients, n, vary with the flow depth.

*Some "temporary" linings become permanent when buried.

Appendix 7-C Permissible Velocities

Table 7C-1 Permissible Velocities

Grass and Earth-Lined Channels

Channel Slope	Lining	Permissible Velocity*
0-5%	Bermuda grass	6 ft/sec
	Reed canary grass	5 ft/sec
	Tall fescue	5 ft/sec
	Kentucky bluegrass	5 ft/sec
	Grass-legume mixture	4 ft/sec
	Red fescue	4 ft/sec
	Redtop	4 ft/sec
	Sericea lespedeza	4 ft/sec
	Annual lespedeza	4 ft/sec

Channel Slope	Lining	Permissible Velocity*
	Small grains (temp)	
5-10%	Bermuda grass	5 ft/sec
	Reed canary grass	4 ft/sec
	Tall fescue	4 ft/sec
	Kentucky bluegrass	4 ft/sec
	Grass-legume mixture	3 ft/sec
>10%	Bermuda grass	4 ft/sec
	Reed canary grass	3 ft/sec
	Tall fescue	3 ft/sec
	Kentucky bluegrass	3 ft/sec

- For highly erodible soils, decrease permissible velocities by 25%

Bare Soil

	Permissible Velocities for Water Flow conditions		
SOIL Types	Clear Water	w/ Fine Silts	w/ Sand & Gravel
Fine Sand (noncolloidal)	1.5 ft/sec	2.5 ft/sec	1.5 ft/sec
Sandy Loam (noncolloidal)	1.7 ft/sec	2.5 ft/sec	2.0 ft/sec
Silt Loam (noncolloidal)	2.0 ft/sec	3.0 ft/sec	2.0 ft/sec
Ordinary Firm Loam	2.5 ft/sec	3.5 ft/sec	2.2 ft/sec
Fine Gravel	2.5 ft/sec	5.0 ft/sec	3.7 ft/sec
Graded, Loam to Cobbles	3.7 ft/sec	5.0 ft/sec	5.0 ft/sec
Graded, Silt to Cobbles (noncolloidal)	4.0 ft/sec	5.5 ft/sec	5.0 ft/sec
Alluvial Silts (noncolloidal)	2.0 ft/sec	3.5 ft/sec	2.0 ft/sec
Alluvial Silts (colloidal)	3.7 ft/sec	5.0 ft/sec	3.0 ft/sec
Coarse Gravels (noncolloidal)	4.0 ft/sec	6.0 ft/sec	6.5 ft/sec
Cobbles and Shingles	4.0 ft/sec	6.0 ft/sec	6.5 ft/sec
Shales and Hard Pan	6.0 ft/sec	6.0 ft/sec	5.0 ft/sec

Source: Special Committee on Irrigation Research, American Society of American Civil Engineers, 1926

Appendix 7-D Shear Stress Capacity

Table 7D-1

Summary Of Shear Stress For Various Protection Measures

Protective Cover	Underlying Soil	t_C (lb/ft^2)
Gravel	D_{50} = 1 in	0.40
	D_{50} = 2 in	0.80
Rock	D_{50} = 6 in	2.50
	D_{50} = 12 in	5.00
6 in Gabions	Type I	35
4 in Geoweb	Type I	10
Soil Cement (8% cement)	Type I	>45
Concrete construction Blocks, granular filter underlayer	Type I	>20
Wedge-shaped blocks with drainage slot	Type I	>25

Source: FHWA-RD-89-110, HEC-15

Appendix 7-E Water Surface Profile Computation

Example Problem

A sample computation is taken from "Hydrologic Engineering Methods For Water Resources Development - Volume 6, Water Surface Profiles", The Hydrologic Engineering Center, Corps of Engineers, U.S. Army, Davis, California.

A convenient form for use in calculating water surface profiles is shown in Figure 7-E-1. In summary, columns 2 and 4 through 12 are devoted to solving Manning's Equation to obtain the energy loss due to friction, columns 13 and 14 contain calculations for the velocity distribution across the section, columns 15 through 17 contain the average kinetic energy, column 18 contains calculations for "other losses" (expansion and contraction losses due to interchanges between kinetic and potential energies as the water flows), and column 19 contains the computed change in water surface elevation. Conservation of energy is accounted for by proceeding from section to section down the computation form.

Column 1 - CROSS SECTION NO., is the cross-section identification number. Miles upstream from the mouth are recommended.

Column 2 - ASSUMED, is the assumed water surface elevation which must agree with the resulting computed water surface elevation within $\pm$ .05 feet, or some allowable tolerance, for trial calculations to be successful.

Column 3 - COMPUTED, is the rating curve value for the first section, but thereafter, is the value calculated by adding WS to the computed water surface elevation for the previous cross section.

Column 4 - A, is the cross section area. If the section is complex and has been subdivided into several parts (e.g., left overbank, channel and right overbank) use one line of the form for each subsection and sum to get A_t, the total area of cross section.

Column 5 - R, is the hydraulic radius. Use the same procedure as for column 4 if section is complex, but do not sum subsection values.

Column 6 - $R_{2/3}$, is 2/3 power of hydraulic radius.

Column 7 - n, is Manning coefficient of channel roughness.

Column 8 - K, is conveyance and is defined as ($C_mAR_{2/3}/n$) where C_m is 1.49 for English units. If the cross section is complex, sum subsection K values to get K_t.

Column 9 - K_t, is average conveyance for the reach, and is calculated by $0.5(K_{td} + K_{tu})$ where subscripts D and U refer to downstream and upstream ends of the reach, respectively.

Appendix 7-E Water Surface Profile Computation

Column 10 - S_f, is the average friction slope through the reach determined by $(Q/K_t)^2$.

Column 11 - L, is the distance between cross sections: different values may be used in each strip.

Column 12 - h_f, is energy loss due to friction through the reach and is calculated by $h_f = (Q/K)^2L$.

Column 13 - $K(K/A)^2$, is part of the expression relating distributed flow velocity to an average value. If the section is complex, calculate one of these values for each subsection and sum all subsection values to get a total. If one subsection is used, Column 13 is not needed and (Column 14) equals one.

Column 14 - α, is the velocity distribution coefficient and is calculated by $K(K/A)^2/(K_t/A_t)^2$ where the numerator is the sum of values in Column 13 and the denominator is calculated from K_t and A_t.

Column 15 - V, is the average velocity and is calculated by Q/A_t.

Column 16 - $V^2/2g$, is the average velocity head corrected for flow distribution.

Column 17 - $(\Delta\alpha V^2/2g)$, is the difference between velocity heads at the downstream and upstream sections. A positive value indicates velocity is increasing, therefore, use a contraction coefficient for "other losses". A negative value indicates the expansion coefficient should be used in calculating "other losses".

Column 18 - h_o, is "other losses", and calculated with either C_e or C_c.

Column 19 - WS, is the change in water surface elevation from the previous cross section. It is the algebraic sum of columns 12, 17 and 18.

Appendix 7-E Water Surface Profile Computation

Cross Section No.	Water Surface Elevation		Area	Hydraulic Radius R	$R^{2/3}$	n	K	$\bar{K}_f$	1000 $\bar{S}_f$	L	h_f	K^3/A^2	α	V	$\alpha V^2/2g$	$\Delta(\alpha V^2/2g)$	h_e	Δ Water Surface Elevation
	Assumed	Computed																
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)

Figure 7-E-1 Water Surface Profile Form

Appendix 7-F HEC-RAS Checklist

INPUT CHECKLIST

1. GEOMETRIC DATA

A. Review the Project Limits (limits of data collection). Is enough information (cross-sections) gathered both upstream and downstream of the study reach? For subcritical regime, make sure that the upstream project limit is at a distance where the water surface profile resulting from a channel modification converges with the existing conditions profile (to evaluate any upstream impacts due to project alternatives). The downstream limit should be far enough to prevent any user identified boundary condition from affecting the results within the study reach. For supercritical regime, the roles of the upstream and downstream limits are reversed.

B. Check the river system schematic. Are the various reaches (for a dendritic river system) properly connected? Inspect the location of junctions. Are the flow directions correct? Check the location of flow splits and flow combinations in looped networks (if any).

C. Review the cross section geometry. Does it characterize locations of changes in discharge, slope, shape or roughness, locations where levees begin or end, at bridges, culverts, weirs, or other control structures? Are the cross sections properly oriented (perpendicular to the anticipated flow lines, i.e. approximately perpendicular to the ground contour lines)? Review individual cross-section plots. Does a cross section extend across the entire floodplain? Is each end of the cross section higher than the anticipated maximum water surface elevation? Is the topography of the channel (bank elevation) and floodplain accurately reflected in the geometry of the cross sections?

D. Review the reach lengths (distances between cross sections). Check that the channel reach lengths are correctly determined along the thalweg and the overbank reach lengths are measured along the anticipated path of the center of mass of the overbank flow. Make sure that the cross section properly reflects the stream size, slope, uniformity of cross-section shape, and the purpose of the study.

E. Review the profile plots of channel bed elevations and top of bank elevations for abrupt changes, adverse grade, or other anomalies.

2. FLOW DATA

A. What is the design discharge and how is it derived?

B. Is there existing discharge data (hydrologic record) that may be more appropriate or required for regulatory purposes?

C. Are there any tributaries at which a change in discharge might be expected?

D. Are there multiple discharges (multiple profile runs)? What return interval (event) does each discharge represent?

Appendix 7-F HEC-RAS Checklist

2. FLOW DATA(continued)

E. What is the expect flow regime? Is there a possibility for mixed flow regime?

3. BOUNDARY CONDITIONS

A. What method is used to establish the staring water surface elevation: observed, slope-area, critical, or other? Is this method appropriate based on available information on flow regime and topography (for subcritical flow, boundary conditions are necessary at the downstream project limit; for supercritical flow, boundary conditions are necessary at the upstream project limit.; for mixed flow conditions, boundary conditions are necessary at both project limits.)?

B. If there is not a known starting water surface elevation, prepare a range of user-defined starting elevations to check the sensitivity of the results in the study reach.

4. ENERGY LOSS COEFFICIENTS

A. What are Manning's roughness coefficients are used for the channel and overbank areas? Review available aerial and/or ground photography. Conduct a field reconnaissance. Are the coefficients realistic and representative of vegetation, season change, channel irregularities, channel alignment, channel slope, stage and discharge, and bedforms? Is there a need to model more than three distinct zones within each cross section (left overbank, channel, and right overbank)? Does aerial photography or field review indicate braided channels or other areas with the horizontal variability or roughness? Check if the observed water surface profile information (gaged data and high water marks) is available for the roughness calibration. Compare the adopted Manning's coefficients to those used in other studies for similar stream conditions and/or those obtained from experimental data.

B. What expansion and contraction coefficients are used to evaluate transition losses? Are they representative of the changes in geometry between successive cross sections and flow regime? Do they include energy losses at bridges, culverts, weirs, and other control structures? Make sure that the coefficients applied between two cross sections are specified as part of the data for the upstream cross sections.

5. INEFFECTIVE FLOW AREAS

A. Are non-conveying, flow separation areas where the velocity in the downstream direction is close to zero (e.g. "shadow areas" outside the main flow conveyance zone approaching or exiting a bridge, culvert, or other flow obstacle) modeled as ineffective?

B. Are depressions such as overbank excavations or low grounds where water ponds but is not actively being conveyed, represented as ineffective flow areas?

Appendix 7-F HEC-RAS Checklist

INPUT CHECKLIST (continued)

6. SPECIAL CONDITIONS

Based on review of the input data, note the existence or any indication of the possible existence of the following conditions for further investigation after reviewing the output:

A. Bridges, culverts, weirs, and other control structures
B. Levees
C. Blocked obstructions
D. Distributary or alluvial fan conditions
E. Split and/or divided flow
F. Islands

OUTPUT CHECKLIST

1. KEY HYDRAULIC PARAMETERS

Check the following parameters for consistency are reasonableness:

A. Flow depth

B. Critical flow depth

C. Velocity

D. Velocity Head

E. Area

F. Top width

G. Invert slope

H. Energy slope

These parameters should vary gradually between cross sections. Note any unusual variations and any extreme values that do not seem realistic or are inconsistent with known conditions regarding the stream reach.

2. FLOW CONSISTENCY

A. Check the streamwise variation (from cross section to cross section) in the flow distribution between the channel and the left/right overbank. Does the amount of flow (discharge) in any one area vary from one cross section to the next?

Appendix 7-F HEC-RAS Checklist

OUTPUT CHECKLIST (continued)

2. FLOW CONSISTENCY (continued)

B. Check the later distribution of flow between the channel and the overbanks for each cross section. Does it seem reasonable (e.g. if majority of flow is in one overbank, is this what is expected based on the input review)?

3. ERROR AND WARNING MESSAGES

Review summary of errors, warnings, and notes generated after each run. It is important to note that the user does not have to eliminate all the warning messages. However, it is up to the user to determine whether or not these warnings require additional actions for the analysis.

Some common messages to look for include:

A. If there are consistent warning messages indicating profile defaulting to critical depth, consider modeling the alternative flow regime (subcritical vs. supercritical). Mixed flow regime should be attempted.

B. Each cross section with the energy equation could not be balanced message (so that critical depth was assumed) requires further examinations. This message is often an indication of unstable modeling (due to insufficient number of cross sections and data points, inconsistent flows, inaccuracy of roughness coefficients and energy losses), rather than critical depth flows depths. However, it is up to the user to determine whether critical depth is a legitimate answer which indicates minimum specific energy of the flow (e.g. at a transition from subcritical to supercritical flow, at a sudden constriction in subcritical flow, etc.).

C. If there are any extended cross sections messages (these messages indicate the computed vertical floodplain limits exceed the limits of the cross section), consider obtaining additional ground points from the available topography to “close” the cross section and account for additional flow area. Consider if flow would actually leave the main channel at this location (split flow situation).

D. In case of divided flow messages, it should be first determined whether the water can actually flow on both sides of the dividing land feature at the specified flow rate. If so, this usually requires separate modeling of divided reaches.

E. Messages indicating change in velocity head and/or conveyance ration exceeding allowable limits imply that the flow area are changing abruptly between cross sections and may call for additional cross sections or specification of ineffective flow areas.

F. Any other messages should be examined and either eliminated or justified.

Appendix 7-F HEC-RAS Checklist

OUTPUT CHECKLIST (continued)

4. SPECIAL CONDITIONS

Based on the foregoing review, determine whether the model input or output suggest any of the following special flow conditions:

A. Bridges and/or culverts? Are boundary cross sections properly located? Has the correct computational method been used (energy, momentum, Yarnell, pressure and/or weir)? Are ineffective flow limits properly specified? Check pressure/low flow distributions in the output?

B. Levees? Is flow confined within levees allowing overbank flow only above the levee crest stage?

C. Blocked obstruction? Note that these elements decrease flow area and add wetted perimeter when water comes in contact with them (unlike ineffective flow limits).

D. Distributary or alluvial fan conditions? Does the output indicate consistent occurrence of flows diverging from a common path without rejoining downstream or do flow characteristics indicate a gradually expanding pattern of flow with little or no boundary definition? If so, a distributary type flow pattern may predominate, making one-dimensional modeling impractical or impossible.

E. Split and/or divided flow? Flow overtopping a divide as side weir flow? Have these areas been accounted for using split flow modeling or other approximation to account for lost flow?

F. Islands? Do the model results indicate isolated flood free areas within the floodplain? The occurrence of islands may indicate a flow pattern similar to split or divided flow where the two (or more) separate flow paths around the island must be modeled separately to accurately determine flow profiles.

Appendix 7-G Equilibrium Slopes & Drop Structures

Equilibrium Slopes & Drop Structures

Given a fixed distribution of sediments, the sediment-transport capacity of a stream is dependent on flow velocity and depth. For most streams, transport of all particle sizes of bed material increases, as flow velocity increases, at a rate proportional to the third to fifth power of the velocity. Correspondingly, transport of sediment particles composed of bed material generally decreases as depth increases, while transport increases with decreased depth.

For the purpose of analysis and design, most natural, undisturbed channels can be assumed to be at or near a state of dynamic equilibrium with regard to sediment transport. This means that over a given reach of channel, the sediment-transport capacity of the channel, over the long term, is more or less equal to the sediment supply. The channel bed is therefore stable".

The equilibrium slope for a channel that has an upstream sediment supply that is essential zero can be computed by:

$$S_{eq} = (1.45n/q^{0.11})^2 \qquad \textbf{(7G-1)}$$

Where:

- S_{eq} = Equilibrium slope with no sediment supply, ft./ft.
- n = Manning's roughness coefficient,
- q = Channel unit discharge, cfs/ft.

For channels that have some sediment-transport capacity, the equilibrium slope can be computed by

$$S_{eq} = [\{(n_u/n_n)^2*(Q_{u,10}/Q_{n,10})^{-1.1} * (b_u/b_n)^{0.4}*(1-R_s)^{0.7}]*S_n \qquad \textbf{(7G-2)}$$

Where:

- S_{eq} = Equilibrium slope
- n_u, n_n = Manning's roughness
- $Q_{u,10}$, $Q_{n,10}$= Ten year discharge,
- b_u, b_n= Channel bottom width, ft
- R_s = Reduction factor for sediment supply. This factor is usually assumed to be equal to the ratio of impervious area to the total area of the watershed.
- S_n = Natural or existing slope, ft./ft.

The subscripts n or u relate to the natural or urbanized condition.

Appendix 7-G Equilibrium Slopes & Drop Structures

Equilibrium Slopes (continued)

Spacing and depth of grade control structures

If the equilibrium slope of a channel, as determined by either equation above, is flatter than the existing or design slope, a grade-control structure may be needed to limit degradation from exceeding a certain depth at any point along the channel. Grade-control structures are barriers in a channel that prevent the channel bed from degrading at a point located immediately upstream of where they are located. After the channel has reached equilibrium, the bed elevation immediately upstream of the grade control structure is at the design elevation. Downstream of the grade-control structure, the bed is at an “equilibrium” elevation that is lower than the design elevation. For most channels the design process is iterative involving drop height, reach length, and depth of scour downstream of the drop.

Once a drop height is chosen, the reach length, or spacing, between adjacent structures can be computed from

$$L_r = h/(S_{ib}-S_{eq}) \qquad \textbf{(7G-3)}$$

Where:
L_r = Reach length, or spacing, between adjacent grade-control structures, in ft.
H = Drop height, measured at the downstream face, in ft.
S_{ib}= Initial channel bed slope, in ft/ft
S_{eq} = Equilibrium channel bed slope, in ft/ft.

If the initial and equilibrium slopes are approximately the same, the distance between structures will be large. Under such circumstances, grade-control structures may not be necessary.

For economical and technical reasons, grade-control structures should be spaced no closer than twelve times the local scour depth as computed by equation 7G-4 or equation 7G-5 as discussed in Chapter 9, energy dissipators.

$$Z_{lsf} = 1.32(q)^{0.54}\,(H_t)^{0.225}-TW \qquad \textbf{(7G-4)}$$

$$Z_{lss} = 0.581(q)^{0.667}\,(h/Y)^{0.411}[1-(h/Y)]^{-0.118} \qquad \textbf{(7G-5)}$$

Grade-control structures may be constructed of un-reinforced concrete walls of no more than two feet high, to very large energy dissipators as discussed in Chapter 11.

Appendix 7-G Equilibrium Slopes & Drop Structures

Equilibrium Slopes (continued)

Example: Spacing and depth of grade-control structures.

A channel is to be constructed in an urbanized area to contain the 100-year discharge. The banks of the channel are to be of shotcrete, the bottom is earth.

Channel characteristics are as follows:

Bottom width = 20 feet
Design slope = 0.006 ft/ft
Side slope 1:1
Manning's n = 0.022

Hydraulic data is

Q_{10} = 350 cfs	Q_{100} = 700 cfs
Y_{10} = 2.1 ft.	Y_{100} = 3.1
V_{10} = 7.7 fps.	V_{100} = 9.7 fps.
Unit discharge, q = 350/20 = 17.5 cfs	Unit discharge, q = 700/20 = 35.0 cfs

As the watershed is considered to be urbanized the equilibrium slope is calculated with equation 6.25.

$S_{eq} = (1.45(n)/q^{0.11}) = (1.45(0.022)/(17.5^{0.11}) = 0.0005$, try a 2-foot drop.

The spacing between drops by equation 7G-3 is

$$L_f = \frac{2.0}{(0.006-0.0005)} = 364 \text{ feet.}$$

Since the drop is 2 feet, and Y_{10} = 2.1 ft. the drop is submerged, the scour is estimated by equation 6.14 as

$$Z_{lss} = 0.581(q_{100})^{0.667}\ (h/Y_{100})^{0.411}[1-(h/Y_{100})]^{-0.118}$$

$$Z_{lss} = 0.581(35)^{0.667}\ (2/3.1)^{0.411}[1-(2/3.1)]^{-0.118}$$

$$Z_{lss} = 5.9 \text{ feet.}$$

Therefore, the total height of the drop structure from the top to the toe should be 5.9 feet plus the two-foot drop height; or 7.9 feet. This dimension does not include any protective depth below the scour.

Appendix 7-G Equilibrium Slopes & Drop Structures

Equilibrium Slopes (continued)

Example: Spacing and depth of grade-control structures. (continued)

If it is desired to keep the depth to less than 6 feet, a one-foot drop results in

$$L_f = \frac{1.0}{(0.006-0.0005)} = 182 \text{ feet.}$$

and

$$Z_{lss} = 0.581(q_{100})^{0.667} (h/Y_{100})^{0.411}[1-(h/Y_{100})]^{-0.118}$$

$$Z_{lss} = 0.581(35)^{0.667} (1/3.1)^{0.411}[1-(1/3.1)]^{-0.118}$$

Z_{lss} = 4.1 feet. For a total height of 5.1 feet.

This would result in 5-foot high drop structures with one-foot exposed at 180-foot spacing.

Appendix 7-H Concrete Lined Channels

Presented are some general guidelines for concrete channel lining design including minimums where appropriate. The channel lining designer should consult with the appropriate materials engineer regarding lining thicknesses, side slopes, concrete mix design, concrete shrinkage criteria, weephole requirements, and required subgrade treatment.

Thickness:
The channel lining thickness and reinforcement shall be designed for the soil conditions at the project site and should consider any negative pressures that might occur and any collapsing or expansive soils. The minimum thickness for trapezoidal channels should be as shown in the table below.

Minimum Channel Lining Thickness

Mean Water	**Slab Thickness (in.)**	
Velocity (ft/sec)	**Bottom Slab**	**Side Slope Slab**
Less than 10	**5**	**5**
10 to 15	**6**	**5.5**
15 to 20	**7.5**	**6**
more than 20	**8**	

Note: 6-inch minimum with tied reinforcement or when bottom slab width is 8 or more feet (required to support vehicles).

Joints:
Concrete channel linings should be continuously reinforced Portland cement without expansion or tooled joints. Longitudinal construction joints should be located as required for construction but within the low flow area of the bottom slab. The bottom slab pours should extend a minimum of 1 foot up the side slope. Transverse construction joints should be provided only when concrete placement stops for more than 45 minutes. The reinforcing shall be continuous through the construction joints and through joints with culverts and other hydraulic structures. **Sealed expansion joints with load transfer devices shall be provided at bridge piers, abutments, and other fixed structures.**

Side slopes:
Side slopes on main channels should be designed for soil conditions at the site but should not exceed a desirable slope of 2:1 with an absolute minimum of 1.5:1. If steeper slopes are required, the lining shall be designed as a retaining wall for appropriate lateral earth pressures.

Subgrade treatment should be on a site-specific basis as recommend by the geotechnical engineer. Pressure relief of channel linings should be provided by geotextile or geocomposite drainage strips and 4-inch diameter PVC weepholes through the lining 12 inches above the channel. The spacing of weepholes should be based on subsurface investigation; potential future changes in ground water levels, any structural backfill and any parallel or crossing futilities.

Appendix 7-H Concrete Lined Channels (continued)

Reinforcing:
Reinforcing steel shall be mild steel of either bars or welded wire fabric uniformly distributed. Longitudinal reinforcement should be a minimum of 0.3 percent of the concrete area. The minimum percentage of transverse reinforcement is dependent upon the top width of the channel with 0.20 percent for widths less than 65 feet, 0.25 percent for widths from 65 to 100 feet and 0.30 percent for widths greater than 100 feet.

Clearance:
Reinforcing shall have a minimum of 3-inch clearance to grade and a minimum of 2-inch clearance to exposed surfaces.

Cutoff walls:
Cutoff walls will be required:

- Where new lining abuts an existing concrete or other type of lining which is not continuously reinforced; (a sealed expansion joint should be provided between the new and existing linings)
- At the beginning (upstream end) of a transition section
- At breaks in the channel profile of more than 0.9 percent; and
- At existing structures where the new lining cannot realistically be made continuous with the existing lining.

Cutoff walls are not required to prevent progressive failure in continuously reinforced channels; however, stability of sloped wall lining at transitions where the cross-section shape changes, or at locations where channel slopes change must be evaluated. The change in directions will result in unbalanced force action away from the supporting earth. To prevent local buckling at these locations, cutoff walls rigidly attached to the paving, should be installed to stiffen the lining.

Appendix 7-I Example Computations

Rectangular Channel

1. Normal depth
2. Critical depth

Trapezoidal Channel

1. Normal depth
2. Channel capacity

Circular Channel

1. Normal depth
2. Critical depth
3. Equivalent depth

Appendix 7-I Example Computations (continued)

Rectangular Channel

1. Normal depth

Given: 5 ft. wide channel,
Q= 200 cfs.
n= 0.012
So = 0.03

Find: Normal depth of flow: d_n

For use of chart need following parameter; $Qn/(b^{2.67}\ So^{0.5})$

$Qn/(b^{2.67}\ So^{0.5}) = (200*0.012) / (5^{2.67}\ 0.03^{0.5}) = 2.4 / (73.49*0.1732) = 2.4/12.73 = 0.188$

From channel chart; d/b = 0.365, therefore depth =0.365*5 = 1.83 ft.

2. Critical depth

Find Critical depth:

For a rectangular channel,
$d_c = 0.315((Q/b)^2)^{0.333}$

$d_c = 0.315*((200/5)^2)^{0.333} = 0.315*(1600)^{0.333} = 3.68$ ft.

Appendix 7-I Example Computations (continued)

Trapezoidal Channel

1. Normal depth

Given:

$Q = 800$ cfs
$B = 8$ ft.
$Z = 4$ (Side Slope 4:1)
$n = 0.03$
$S_o = 0.05$

Find normal depth and velocity.

For use of chart need following parameter; $Qn/(b^{2.67}\ So^{0.5})$

$Qn/(b^{2.67}\ So^{0.5}) = (800*0.03) / (8^{2.67}\ 0.05^{0.5}) = 24 / (257.8*0.2236) = 24/57.65 = 0.416$

From channel chart; d/b = 0.34, therefore depth =0.34*8 = 2.72 ft.

For a depth of 2.72, top width = 8.0 + 2.72*2*4 = 29.76 ft.

Area = d(b1 +b2)/2 = 2.72*(8 + 29.76)/2 = 51.35 sq. ft.

Velocity = Q/A = 800/51.35 = 15.6 ft/sec.

2. Channel capacity

Given : $B = 4$ ft.
$d = 2$ ft
$Z = 4$ (Side Slope 4:1)
$n = 0.035$
$S_o = 0.05$

Find velocity and discharge.

$V = (1.486/n)\ R^{0.67}S^{0.5}$

Where R = A/P.
$A= Bd + Zd^2 = 4(2) + 4(2)^2 = 24.0$

$P = b + 2d(Z^2+1)^{0.5} = 4 +2(2)(4^2 +1)^{0.5} = 4 +2(2)(4^2 +1)^{0.5} = 20.49$ ft.

$V = (1.486/0.035)*(1.171)^{0.67}\ (0.05)^{0.5}$

Appendix 7-I Example Computations (continued)

2. Channel capacity (continued)

R=A/P = 24.0/20.49 = 1.171

V = 42.46*(1.112)*(0.2236) = 10.55

Q = VA

Q = 10.55*24.0 = 253.2 cfs

Circular channel

1. Normal depth

Given:
Q = 400 cfs
D = 60" = 5.0 ft.
n = 0.012
S_o = 0.05

For use of chart 47 need following parameter; $Qn/(D^{2.67}\ S_o^{0.5})$

$Qn/(D^{2.67}\ S_o^{0.5})$ = (300*0.012) /($5^{2.67}$ $0.05^{0.5}$) = 0.22

From chart d/D = 0.57, d_n = 0.57*5.0 = 2.85

2. Critical depth

Given:
Q= 300 cfs.
D = 60" = 5.0 ft.

Using chart critical depth chart, d_c = 4.7

3. Equivalent depth

Q= 400 cfs
n = 0.012
S_o = 0.05

Appendix 7-I Example Computations (continued)

3. Equivalent depth (continued)

Find equivalent depth, d_e

$d_e = (A/2)^{0.5}$

$$d/D = \frac{\text{depth of Flow}}{\text{Diameter of Pipe}} = \frac{2.85}{5.0} = 0.57$$

From d/d vs A, $A = 0.4625D^2$

$A = 0.4625*(25) = 11.56$ sq.ft.

$d_e = (11.56/2)^{0.5}$

$d_e = 2.40$ ft.

CHAPTER 8

CULVERTS

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8.1 Introduction

8.1.1 Purpose

This chapter provides general design procedures for the hydraulic design of highway culverts. For more in-depth discussion of the hydraulics of culverts see the FHWA Hydraulic Design Series Number 5 (HDS 5), Hydraulic Design of Highway Culverts. This chapter also presents results of culvert analysis using microcomputers that demonstrates the use of the HY8 culvert analysis software.

Culverts are structures designed to convey water through embankments, the hydraulics are for free surface flow approaching the culvert, with no consideration of the approach velocity (i.e. ponded conditions).

8.1.2 Definitions

Critical Depth

Critical depth is the depth at which the specific energy of a given flow rate is at a minimum. For a given discharge and cross-section geometry there is only one critical depth. Appendix D at the end of this chapter contains critical depth charts for different shapes.

Crown (Soffit)

The crown is the inside top of the culvert.

Flow Type

There are seven culvert flow types (USGS), but for highway culvert design a simplified approach with only two types of flow (inlet control and outlet control) are used. Diagrams of these flow types are provided in the design methods section.

Free Outlet

A free outlet has a tailwater equal to or lower than critical depth. For culverts having free outlets, lowering of the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.

Headwater

Headwater is the depth of water that is be ponded at the upstream end of the culvert during the flood event.

Improved Inlet

An improved inlet has an entrance geometry which decreases the flow contraction at the inlet and thus increases the capacity of culverts. These inlets are referred to as either side- or slope-tapered (walls or bottom tapered).

8.1 Introduction (continued)

8.1.2 Definitions (continued)

Invert

The invert is the flowline of the culvert (inside bottom).

Normal Depth Flow

Normal depth flow occurs in a channel reach when the discharge, velocity and depth of flow do not change throughout the reach. The water surface and channel bottom will be parallel. This type of flow will exist in a culvert operating on a constant slope provided the culvert is sufficiently long so that normal depth is achieved.

Slope

- A steep slope occurs where normal depth is less than critical depth.
- A mild slope occurs where normal depth is greater than critical depth.
- Critical slope occurs where the specific energy of a given flow rate is at a minimum. For a given discharge and cross-section geometry there is only one critical slope.

Submerged

- A submerged outlet occurs when the tailwater elevation is higher than the crown of the culvert at the outlet end.
- A submerged inlet occurs when the headwater is greater than 1.2D where D is the culvert diameter or barrel height.

Tailwater

Tailwater is the depth of water that is at the downstream end of the culvert during the flood event.

8.1 Introduction (continued)

8.1.3 Symbols

To provide consistency within this chapter as well as throughout this manual the symbols given in Table 8-1 will be used. These symbols were selected because of their wide use in culvert publications.

Table 8-1 Symbols

Symbol	Definition	Units
A	Area of cross section of flow	Ft^2
AHW	Allowable High Water	Ft
B	Barrel width	Ft
D	Culvert diameter or barrel height	Ft. or in.
DHW	Design Highwater	Ft
d	Depth of flow	Ft
d_c	Critical depth of flow	Ft
g	Acceleration due to gravity	$Ft./sec^2$
H	Sum of $H_E + H_f + H_o$	Ft
H_b	Bend headloss	Ft
H_E	Entrance headloss	Ft
H_f	Friction headloss	Ft
H_L	Total energy losses	Ft
H_o	Outlet or exit headloss	Ft
H_v	Velocity head	Ft
h_o	Hydraulic grade line height above outlet invert	Ft
HW	Headwater depth (subscript indicates section)	Ft
K_E	Entrance loss coefficient	-
L	Length of culvert	Ft
n	Manning's roughness coefficient	-
P	Wetted perimeter	Ft
Q	Rate of discharge	Ft^3/sec
R	Hydraulic radius (A/P)	Ft
S	Slope of culvert	Ft/Ft
TW	Tailwater depth above invert of culvert	Ft
V	Mean velocity of flow with barrel full	Ft/sec
V_d	Mean velocity in downstream channel	Ft/sec
V_o	Mean velocity of flow at culvert outlet	Ft/sec
V_u	Mean velocity in upstream channel	Ft/sec
γ	Unit weight of water	Lb/Ft^3
τ	Tractive force	Lb/Ft^2

8.2 Design Goals & Guidelines

8.2.1 Design Goals

The following design goals are specific to culverts.

- All culverts that are to convey a quantifiable discharge (design flow) shall be hydraulically designed.
- The design flood selected shall be consistent with the class of highway and commensurate with the risk at the site.
- Culvert location in both plan and profile shall be investigated to avoid sediment build-up in culvert barrels.
- Culverts shall be designed with consideration of debris by either providing debris screens or by using an enlarged opening.
- Culverts in urban situations may need to have access barriers.
- Where practicable, some means shall be provided for personnel and equipment access to facilitate maintenance.

Length and Slope

The culvert length and slope shall be chosen to approximate existing topography, and to the degree practicable:
- the culvert invert shall be aligned with the channel bottom and the skew angle of the stream, and
- the culvert entrance shall match the geometry of the roadway embankment.

Debris Control

Debris control shall be considered:
- where experience or physical evidence indicates the watercourse will transport a heavy volume of controllable debris,
- for culverts located in mountainous or steep regions,
- for culverts that are under high fills, and
- where clean out access is limited. However, access must be available to clean out the debris control device.

Screens are usually provided upstream of the culvert inlet, see Hydraulic Engineering Circular No. 9, "Debris-Control Structures" for design information.

Allowable Headwater

Allowable headwater is the depth of water that can be ponded at the upstream end of the culvert during the design flood that is limited by one or more of the following:

- non-damaging to upstream property,
- 3 inches below the edge of the pavements,
- equal to the elevation above which flow diverts around the culvert to another watercourse,
- May be limited to an elevation that does not adversely affect the performance of upstream culverts.

8.2 Design Goals & Guidelines (continued)

8.2.1 Design Goals (continued)

Tailwater Relationship - Channel

- Evaluate the hydraulic conditions of the downstream channel to determine a tailwater depth for a range of discharges.
- Calculate backwater curves at sensitive locations or use a single cross section analysis. (Backwater curves yield the most accurate tailwater.)
- Use the critical depth and equivalent hydraulic grade line if the culvert outlet is operating with a free outfall.
- Use the headwater elevation of any nearby, downstream culvert if it is greater than the channel depth. A backwater surface profile may be appropriate if the tailwater significantly impacts the culvert under consideration.

Tailwater Relationship - Confluence or Large Water Body

- Use the high water elevation that has the same frequency as the design flood if events are known to occur concurrently (statistically dependent).
- If statistically independent, the tailwater should be evaluated for two conditions; 1.) that from the design peak flow of the culvert concurrent with the 10-year peak flow in the main watercourse, 2.) that from the 10-year peak flow in the culvert with the design peak flow in the main watercourse.

Maximum Velocity

At the design flow, the maximum velocity at the culvert exit shall be compared with the velocity in the natural channel as described in Section 612 2.C of the Roadway Design Guidelines (RDG).

- No protection is generally required in the natural stream if the outlet velocity is less than 1.5 times the natural stream velocity.
- Dumped rock riprap is generally sufficient for ratios between 1.5 and 2.0 with an outlet velocity less than 10 fps.
- Wire-tied rock riprap should generally be used where the ratio is 1.5 to 2.5 with an outlet velocity between 10 and 15 fps.
- Energy dissipators are required when the ratio between outlet and natural stream velocities is greater than 2.5 or the outlet velocity is greater than 15 fps.

Minimum Velocity

The minimum velocity in the culvert barrel shall result in a tractive force ($\tau = \gamma dS$) greater than critical τ of the transported streambed material at low flow rates.

- Use 2.5 ft/sec. when streambed material size is not known.
- If clogging is probable, consider installation of a sediment trap or size culvert to facilitate cleaning.

8.2 Design Goals & Guidelines (continued)

8.2.1 Design Goals (continued)

Flood Frequency

The flood frequency used to design the culvert shall be based on:

- the roadway classification, (Chapter 600, Roadway Design Guidelines)
- existence of FEMA mapped floodplains, and
- the level of risk associated with the 100-year event to adjacent property.

8.2.2 Design Features

Alternative Analysis

Culvert alternatives shall be selected that satisfy:
- topography (fit the site)
- design policies and criteria

Alternatives shall be analyzed for:
- hydraulic equivalency,
- environmental impact (changes in velocity, flow distribution, and alignment), and
- risk and cost.

The selected alternative should best integrate engineering, economic and social considerations. The chosen culvert shall meet the selected structural and hydraulic criteria and shall be based on:

- construction and maintenance costs,
- risk of failure or property damage,
- traffic safety,
- environmental or aesthetic considerations,
- social or nuisance considerations, and
- land use requirements.

Culvert Sizes and Shape

The culvert size and shape selected shall be based on engineering and economic criteria related to site conditions.

- The following minimum sizes shall be used to minimize maintenance problems and clogging: 24” pipe culvert, 6’x6’ box culvert, (a smaller box culvert may be used as required with approval of the Drainage Section Supervisor and District representative).
- Land use requirements, such as need for an equipment or animal pass, may dictate a larger or different barrel geometry than required for hydraulic considerations.
- Use arch or oval shapes only if required by hydraulic limitations, site characteristics, structural criteria, or environmental criteria.

8.2 Design Goals & Guidelines (continued)

8.2.2 Design Features (continued)

Multiple Barrels

Multiple barrel culverts shall fit within the natural dominant channel with minor widening of the channel so as to avoid conveyance loss through sediment deposition in some of the barrels. They are to be avoided where:

- the approach flow is high velocity, particularly if supercritical, (These sites require either a single barrel or special inlet treatment to avoid adverse hydraulic jump effects.)
- irrigation canals or ditches are present unless approved by the canal or ditch owner,

Culvert Skew

The culvert skew shall not exceed 45° as measured from a line perpendicular to the roadway centerline without the approval of the Drainage Engineer. Culvert skews shall be specified to no greater precision than 1 degree and should be constructible with standard designs of 15, 30, and 45 degrees skew.

Broken-back Culverts

Broken-back culverts may be necessary where the topography does not allow for a continuous grade. Broken-back culverts shall only be used with the approval of the Drainage Section.

End Treatment (Inlet or Outlet)

Inlet and Outlet end treatments are shown in the ADOT B- and C- standards. Culverts 48” and larger shall have a headwall. Culverts less than 48” shall have an end section.

Culvert ends may be protected from traffic impacts as follows:

- Small culverts, 30 in. in diameter or less, shall use an end section or slope paving.
- Culverts greater than 30-in. in diameter may receive one of the following:
    a. be extended to the appropriate "clear zone" distance as shown in the AASHTO Roadside Design Guide.
    b. safety treated with a grate if the consequences of clogging and causing a potential flooding hazard is less than the hazard of vehicles impacting an unprotected end. If a grate is used, the net area of the grate (excluding the bars) shall be 1.5 to 3.0 times the culvert entrance area.
    c. shielded with a traffic barrier if the culvert is very large, cannot be extended, has a channel that cannot be safely traversed by a vehicle, or has a significant flooding hazard with a grate.

Performance Curves

Performance curves may be used for evaluating the hydraulic capacity of a culvert. These curves will display the effects of different flow rates at the site and provide a basis for evaluating flood hazards.

8.2 Design Goals & Guidelines (continued)

8.2.2 Design Features (continued)

Outlet Protection

Outlet protection shall be provided where necessary due to excessive outlet velocities or a significant difference between the outlet velocity and the downstream channel velocity. See Energy Dissipator Chapter.

8.2.3 Design Methods

Hydrology Methods

A. Constant Discharge
- Is assumed for most culvert designs.
- Is usually the peak discharge.
- Will yield a conservatively sized structure where temporary storage is available, but not used.

B. Hydrograph and Routing
- Storage capacity behind a highway embankment attenuates a flood hydrograph and reduces the peak discharge.
- Significant storage will reduce the required culvert size. However, the storage should not create a state class dam.
- Is checked by routing the design hydrographs through the culvert site to determine the outflow hydrograph and stage (backwater) behind the culvert.
- Procedures are in HDS 5, Section V.
- Must be of a permanent nature, and
- May only be used by approval of the Drainage Section.

Computational Methods

Nomographs

- Require a trial and error solution that is quite easy and provides reliable designs for many applications.
- Require additional computations for tailwater, outlet velocity, hydrographs, routing and roadway overtopping.
- Circular and box shapes are included in the appendix of this chapter. Other shapes and improved inlets are found in HDS 5.

8.2 Design Goals & Guidelines (continued)

Computational Methods (continued)

Computer Software

HY8 (FHWA Culvert Analysis Software)
- Uses the theoretical basis for the nomographs.
- Can compute tailwater, improved inlets, road overtopping, hydrographs, routing and multiple independent barrels.
- Develops and plots tailwater rating curves.
- Develops and plots performance curves.

Other computer programs may be used, they must use the methodology presented in HDS 5. The designer shall check with the ADOT Drainage section prior to use. Comparison of computer program results with HDS 5 nomographs may be required.

8.3 Design Equations

8.3.1 General

An exact theoretical analysis of culvert flow is extremely complex because the following is required:
- analyzing nonuniform flow with regions of both gradually varying and rapidly varying flow,
- determining how the flow type changes as the flow rate and tailwater elevations change,
- applying backwater and drawdown calculations, energy and momentum balance,
- applying the results of hydraulic model studies, and
- determining if hydraulic jumps occur and if they are inside or downstream of the culvert barrel

8.3.2 Approach

The procedures in this chapter use the following:

Control Section

The control section is the location where there is a unique relationship between the flow rate and the upstream water surface elevation. Inlet control is governed by the inlet geometry. Outlet control is governed by a combination of the culvert inlet geometry, the barrel characteristics and the tailwater.

Minimum Performance

Minimum performance is assumed by analyzing both inlet and outlet control and using the highest headwater. The culvert may operate more efficiently at times (more flow for a given headwater level), but it will not operate at a lower level of performance than calculated.

8.3 Design Equations (continued)

8.3.3 Inlet Control

For inlet control, the control section is at the upstream end of the barrel (the inlet). The flow passes through critical depth near the inlet and becomes shallow, high velocity (supercritical) flow in the culvert barrel. Depending on the tailwater, a hydraulic jump may occur downstream of the inlet.

Headwater Factors

- Headwater depth is measured from the inlet invert of the inlet control section to the surface of the upstream pool.
- Inlet area is the cross-sectional area of the face of the culvert. Generally, the inlet face area is the same as the barrel area.
- Inlet edge configuration describes the entrance type. Some typical inlet edge configurations are thin edge projecting, mitered, square edges in a headwall and beveled edge.
- Inlet shape is usually the same as the shape of the culvert barrel. Typical shapes are rectangular, circular, elliptical and arch. Check for an additional control section, if different than the barrel.

Hydraulics

Three regions of flow are shown in the Figure 8-1: unsubmerged, transition and submerged:

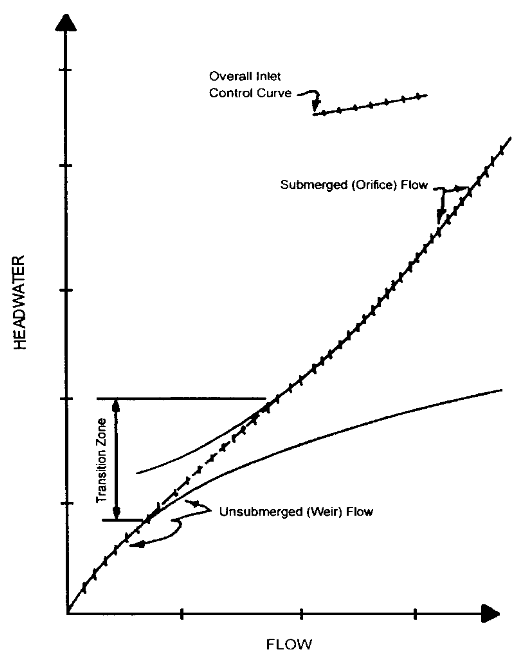


Figure 8-1 Unsubmerged, Transition And Submerged

8.3 Design Equations (continued)

8.3.3 Inlet Control (continued)

Unsubmerged

For headwater below the inlet crown, the entrance operates as a weir.

- A weir is a flow control section where the upstream water surface elevation can be predicted for a given flow rate.
- The relationship between flow and water surface elevation must be determined by model tests of the weir geometry or by measuring prototype discharges.
- These tests are then used to develop equations. Appendix A of HDS 5 contains the equations which were developed from model test data, see Figure 8-2, Unsubmerged:

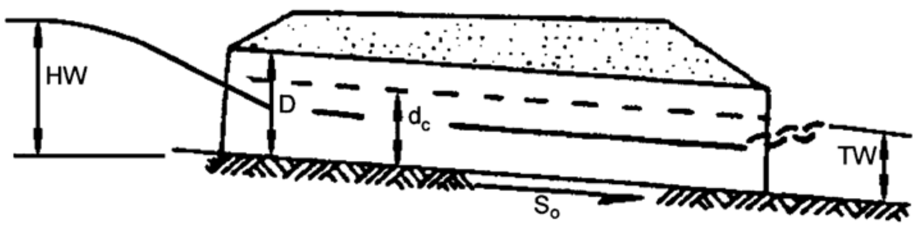


Figure 8-2 Unsubmerged

Submerged

For headwaters above the inlet, the culvert operates as an orifice.

- An orifice is an opening, submerged on the upstream side and flowing freely on the downstream side, which functions as a control section.
- The relationship between flow and headwater can be defined based on results from model tests. Appendix A of HDS 5 contains flow equations that were developed from model test data. See Figure 8-3, Submerged.

Transition Zone

The transition zone is located between the unsubmerged and the submerged flow conditions where the flow is poorly defined. This zone is approximated by plotting the unsubmerged and submerged flow equations and connecting them with a line tangent to both curves.

8.3 Design Equations (continued)

8.3.3 Inlet Control (continued)

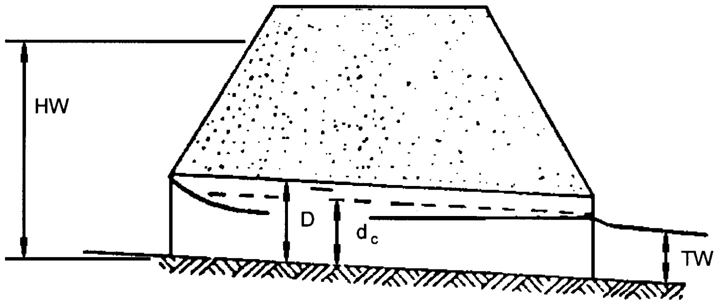


Figure 8-3 Submerged

Nomographs

The inlet control flow versus headwater curves which are established using the above procedure are the basis for constructing the inlet control design nomographs. Note that in the inlet control nomographs, HW is measured to the total upstream energy grade line including the approach velocity head.

8.3.4 Outlet Control

Outlet control has depths and velocity that are subcritical. The control of the flow is at the downstream end of the culvert (the outlet). The tailwater depth is either assumed to be critical depth near the culvert outlet or the downstream channel depth, whichever is higher. In a given culvert, the type of flow is dependent on all of the barrel factors. All of the inlet control factors also influence culverts in outlet control.

Barrel Roughness

Barrel roughness is a function of the material used to fabricate the barrel. Typical materials include concrete and corrugated metal. The roughness is represented by a hydraulic resistance coefficient such as the Manning n value. Typical Manning n values for pipe materials are presented in Appendix B.

Barrel Area

Barrel area is measured perpendicular to the flow.

8.3 Design Equations (continued)

8.3.4 Outlet Control (continued)

Barrel Length

Barrel length is the total culvert length from the entrance crown to the exit crown of the culvert. Because the design height of the barrel and the slope influence the actual length, an approximation of barrel length is usually necessary to begin the design process.

Barrel Slope

Barrel slope is the actual slope of the culvert barrel, and is often the same as the natural stream slope. However, when the culvert inlet or outlet is raised or lowered, the barrel slope is different from the stream slope.

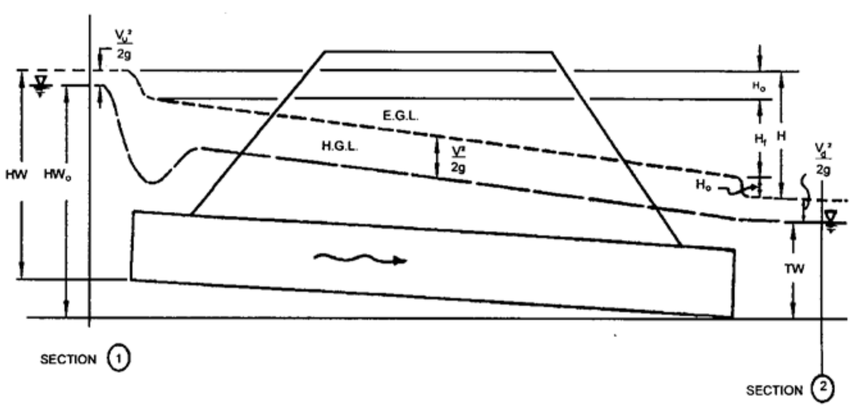
Tailwater Elevation

Tailwater is based on the downstream water surface elevation. Backwater calculations from a downstream control, a normal depth approximation, or field observations are used to define the tailwater elevation (see Section 8.3.3).

Hydraulics

Full flow in the culvert barrel is assumed for the analysis of outlet control hydraulics. Outlet control flow conditions can be calculated based on an energy balance from the tailwater pool to the headwater pool

Figure 8-4 Outlet Control Flow, TW ≥ D



8.3 Design Equations (continued)

8.3.4 Outlet Control (continued)

Losses: $$H_L = H_E + H_f + H_o + H_b + H_j + H_g \quad (8.1)$$

Where:
- H_L = total energy loss, ft.
- H_E = entrance loss, ft.
- H_f = friction losses, ft.
- H_o = exit loss (velocity head), ft.
- H_b = bend losses, ft. (see HDS 5)
- H_j = losses at junctions, ft. (see HDS 5)
- H_g = losses at grates, ft. (see HDS 5)

Velocity: $$V = Q/A \quad (8.2)$$

Where:
- V = average barrel velocity, ft/sec
- Q = flow rate, ft^3/sec
- A = cross sectional area of flow with the barrel full, ft^2

Velocity head: $$H_v = V^2/2g \quad (8.3)$$

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

Entrance loss: $$H_E = K_E\,(V^2/2g) \quad (8.4a)$$

Where: K_E = entrance loss coefficient, see table in Appendix B

Friction loss: $$H_f = [(29n^2L)/R^{1.33}]\,[V^2/2g] \quad (8.4b)$$

Where:
- n = Manning's roughness coefficient, see table in Appendix B
- L = length of the culvert barrel, ft.
- R = hydraulic radius of the full culvert barrel = A/P, ft.
- P = wetted perimeter of the barrel, ft.

Exit loss: $$H_o = 1.0\,[(V^2/2g) - (V_d^2/2g)] \quad (8.4c)$$

Where: V_d = channel velocity downstream of the culvert, ft./sec (usually neglected, see equation 8.4d).

$$H_o = H_v = V^2/2g \quad (8.4d)$$

Barrel losses: $$H = H_E + H_o + H_f$$

$$H = [1 + K_e + (19.63n^2L/R^{1.33})]\,[V^2/2g] \quad (8.5)$$

8.3 Design Equations (continued)

8.3.4 Outlet Control (continued)

Energy Grade Line

The energy grade line represents the total energy at any point along the culvert barrel. Equating the total energy at sections 1 and 2, upstream and downstream of the culvert barrel in Figure 8-4, the following relationship results:

$$\mathbf{HW_o+(V_u^2/2g) = TW+(V_d^2/2g)+H_L} \tag{8.6}$$

Where:

HW_o = headwater depth above the outlet invert, ft.
V_u = approach velocity, ft./sec
TW = tailwater depth above the outlet invert, ft.
V_d = downstream velocity, ft/sec
H_L = sum of all losses (equation 8.1)

Hydraulic Grade Line

The hydraulic grade line is the depth to which water would rise in vertical tubes connected to the sides of the culvert barrel. In full flow, the energy grade line and the hydraulic grade line are parallel lines separated by the velocity head except at the inlet and the outlet.

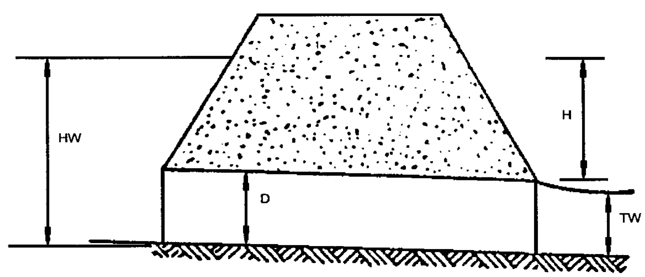


Figure 8-5 Outlet Control, TW = D

8.3 Design Equations (continued)

8.3.4 Outlet Control (continued)

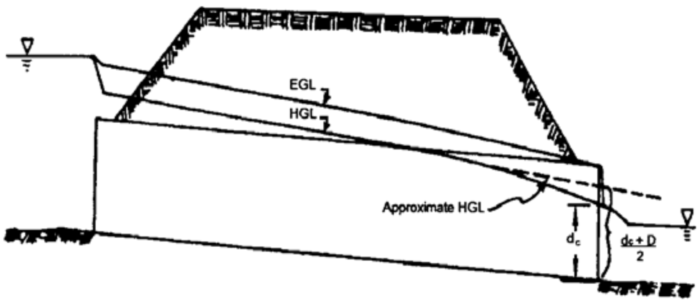


Figure 8-6 Outlet Control, TW < d_c

Nomographs (full flow)

The nomographs were developed assuming that the culvert barrel is flowing full and:
- TW ≥ D, (see Figure 8-4) or
- d_c ≥ D, (see Figure 8-5)
- V_u is small and its velocity head can be considered to be a part of the available headwater (HW) used to convey the flow through the culvert.
- V_d is small and its velocity head can be neglected.

Equation (8.6) becomes:

$$\mathbf{HW = TW + H - S_oL} \quad \textbf{(8.7)}$$

Where: HW = depth from the inlet invert to the energy grade line, ft.
H = is the value read from the nomographs (equation 8.5), ft.
S_oL = drop from inlet to outlet invert, ft.

Nomographs (Partly full flow)

Equations (8.1) through (8.7) were developed for full barrel flow. The equations also apply to the flow situations which are effectively full flow conditions, if TW < d_c, Figure 8-6.

- Backwater calculations may be required which begin at the downstream water surface and proceed upstream. If the depth intersects the top of the barrel, a full flow extends from that point upstream to the culvert entrance.

8.3 Design Equations (continued)

8.3.4 Outlet Control (continued)

Nomographs (Partly full flow) - Approximate method

Based on numerous backwater calculations performed by the FHWA staff, it was found that the hydraulic grade line pierces the plane of the culvert outlet at a point approximately one-half way between critical depth and the top of the barrel or $(d_c + D)/2$ above the outlet invert. TW should be used if higher than $(d_c + D)/2$. The following equation should be used:

$$\mathbf{HW = h_o + H - S_oL} \qquad \mathbf{(8.8)}$$

Where: h_o = the larger of TW or $(d_c + D)/2$, ft.

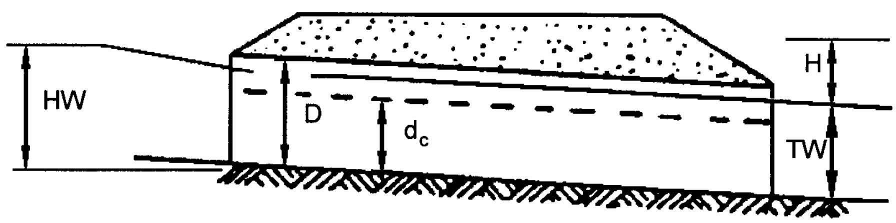
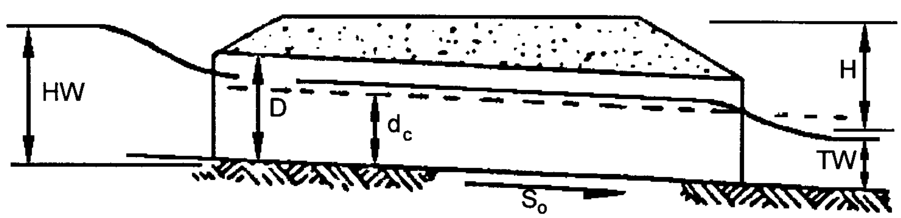


Figure 8-7 Outlet Control, TW < d_c

Adequate results are obtained down to a HW = 0.75D. For lower headwaters, backwater calculations are required. (See Figure 8-7 if TW < d_c and Figure 8-8 if TW > d_c)

Figure 8-8 Outlet Control, TW > d_c



8.3 Design Equations (continued)

8.3.5 Outlet Velocity

Culvert outlet velocities shall be calculated to determine need for erosion protection at the culvert exit. Culverts usually result in outlet velocities that are higher than the natural stream velocities. These outlet velocities may require flow readjustment or energy dissipation to prevent downstream erosion. If outlet erosion protection is necessary, the flow depths and Froude number may also be needed (see Chapter 9, Energy Dissipators).

Inlet Control

The velocity is calculated from equation 8.2 after determining the outlet depth. Either of the following methods may be used to determine the outlet depth.

- Calculate the water surface profile through the culvert. Begin the computation at d_c at the entrance and proceed downstream to the exit. Determine at the exit the depth and flow area.
- Assume normal depth and velocity. This approximation may be used if the culvert is of adequate length so that the water surface profile converges towards normal depth. This outlet velocity may be slightly higher than the actual velocity at the outlet. Normal depths may be obtained from design aids in Chapter 7. If the culvert is steep and sufficiently long that normal depth occurs, high tailwater may force a hydraulic jump. It may be necessary to evaluate the location of the hydraulic jump.

Outlet Control

The cross sectional area of the flow is defined by the geometry of the outlet and either critical depth, tailwater depth, or the height of the conduit.

- Critical depth is used when the tailwater is less than critical depth.
- Tailwater depth is used when tailwater is greater than critical depth, but below the top of the barrel.
- The total barrel area is used when the tailwater exceeds the top of the barrel.

8.3.6 Roadway Overtopping

Roadway overtopping will begin when the headwater rises to the elevation of the roadway. The overtopping will usually occur at the low point of a sag vertical curve on the roadway. The flow will be similar to flow over a broad crested weir. Flow coefficients for flow overtopping roadway embankments are found in Hydraulic Design of Highway Culverts, HDS No. 5**.** For flow overtopping a median barrier, the weir will no longer function as a broad crested weir. H must be measured from the top of barriers.

8.3 Design Equations (continued)

8.3.6 Roadway Overtopping (continued)

$$Q_r = C_d\ L\ HW_r^{1.5} \tag{8.9}$$

Where: Q_r = overtopping flow rate, ft^3/sec
C_d = overtopping discharge coefficient (weir coefficient) = $k_t\ C_r$
k_t = submergence coefficient
C_r = discharge coefficient
L = length of the roadway crest, ft.
HW_r = the upstream depth, measured above the roadway crest, ft.

Height

The height is measured above the point where the flow "crests" over the "dam". It may be the top of guardrail or barrier, if these are present.

Length

The length is difficult to determine when the crest is defined by a roadway sag vertical curve.

- Recommend subdividing into a series of segments. The flow over each segment is calculated for a given headwater. The flows for each segment are added together to determine the total flow.
- The length can be represented by a single horizontal line (one segment). The length of the weir is the horizontal length of this segment. The depth is the average depth (area/length) of the upstream pool above the roadway.

Total Flow

- Roadway overflow plus culvert flow must equal total design flow.
- Roadway overflow is calculated for a given upstream water surface elevation using equation 8.9
- A trial and error process is necessary to determine the flow passing through the culvert and the amount flowing across the roadway.

8.3.7 Performance Curves

Performance curves are plots of flow rate versus headwater depth or elevation, velocity, or outlet scour. The culvert performance curve is made up of the controlling portions of the individual performance curves for each of the following control sections. Performance curves for the culvert and the road overflow may be summed to yield an overall performance. (See Figure 8-9):

Inlet

The inlet performance curve is developed using the inlet control nomographs. (see Appendix A).

Outlet

The outlet performance curve is developed using equations 8.1 through 8.7, the outlet control nomographs (see Appendix A), or backwater calculations.

8.3 Design Equations (continued)

8.3.7 Performance Curves (continued)

Roadway

Roadway performance curve is developed using equation 8.9.

Overall

Overall performance curve is the sum of the flow through the culvert and the flow across the roadway and 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 may need to be calculated.

2. Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.

3. When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the upstream water surface depth above the roadway for each selected flow rate. Use these water surface depths and equation 8.9 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 as shown in Figure 8-9.

8.3 Design Equations (continued)

8.3.7 Performance Curves (continued)

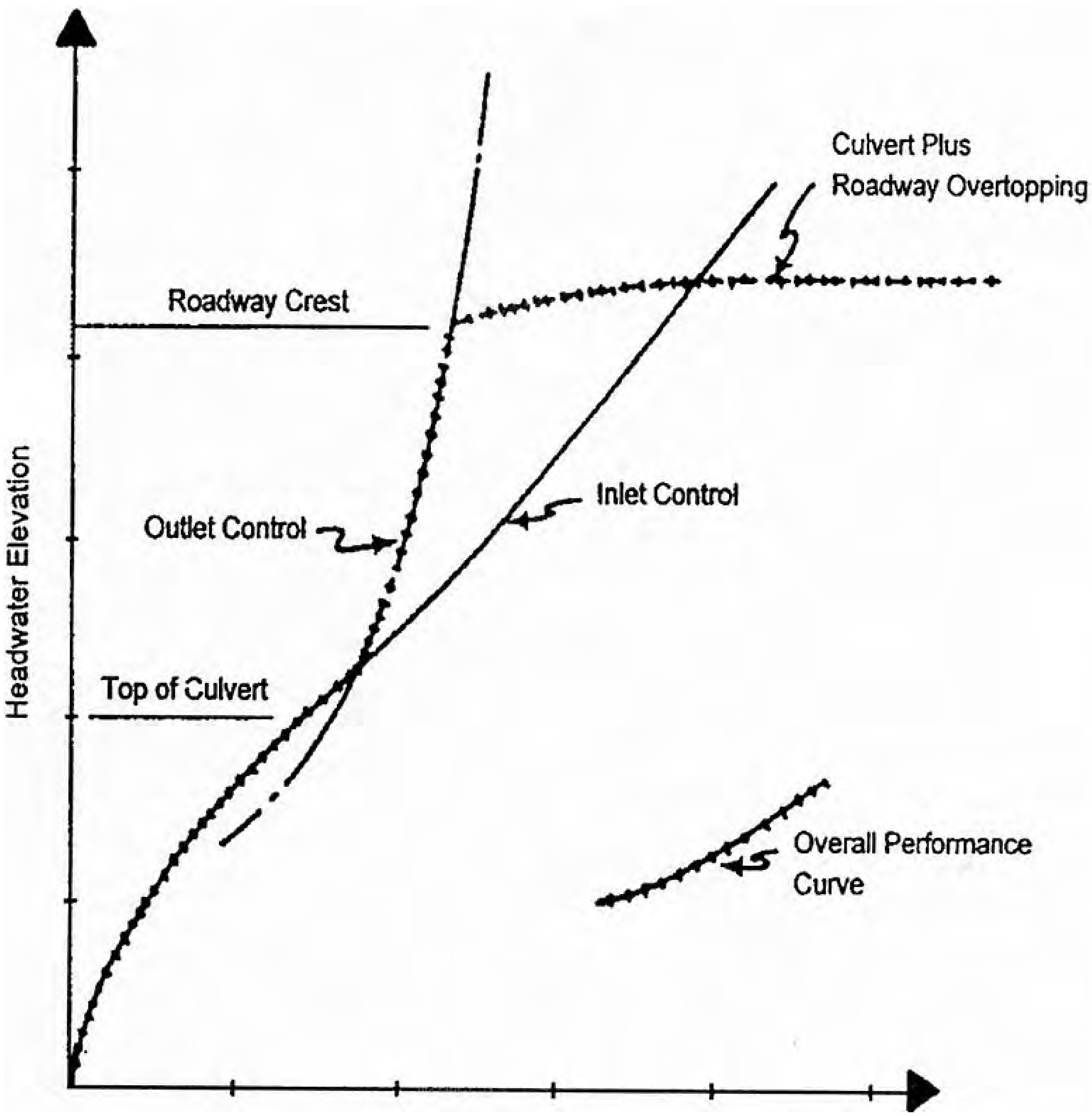


Figure 8-9 Overall Performance Curve

8.4 Design Procedure

The following design procedure provides a convenient and organized method for designing culverts for a constant discharge, considering inlet and outlet control. The procedure does not address the affect of storage which is discussed in the Storage Chapter.

- The designer should be familiar with all the equations in Section 8.3 before using these procedures.
- Following the design method without an understanding of culvert hydraulics can result in an inadequate, unsafe, or costly structure.
- The computation form has been provided in Appendix B to guide the user. It contains blocks for the project description, designer's identification, hydrologic data, culvert dimensions and elevations, trial culvert description, inlet and outlet control HW, culvert barrel selected and comments.

Step 1 Assemble Site Data And Project File

a. See Data Chapter — The minimum data are:
- site and location maps, USGS,
- embankment cross section,
- roadway profile,
- field visit (sediment, debris)

Desirable Data includes
- photographs, and
- design data at nearby structures.

b. Studies by other agencies including:
- small dams -- NRCS, USCE, BLM,
- canals -- NRCS, USCE, USBR, SRP
- floodplain -- FEMA, NRCS, USCE, USGS, and Flood Control Districts, and
- storm drain -- local or private.

c. Environmental constraints including:
- commitments contained in project and environmental documents,

Design criteria
- allowable headwater elevation
- allowable outlet velocity.

Step 2 Determine Hydrology

a. Determine flood frequency from criteria.
b. Identify discharges to be used in determining culvert size.

8.4 Design Procedure (continued)

Step 3 Evaluate Downstream Channel

a. Determine tailwater depth/elevation information
b. Minimum data are cross section of channel and the rating curve for channel.

Step 4 Summarize Data On Design Form

a. See Chart in Appendix B.
b. Data from steps 1-3.

Step 5 Identify Design Alternative

a. Choose culvert material, shape, size and entrance type.

Step 6 Select Design Discharge Q_d

a. Divide design Q by the number of barrels.

Step 7 Determine Inlet Control Headwater Depth (HW_i)

Use the inlet control nomograph (Appendix A).

a. Locate the size or height on the scale.
b. Locate the discharge.
- For a circular shape use discharge.
- For a box shape use Q per foot of width.

c. Locate HW/D ratio.
- Use a straight edge.
- Extend a straight line from the culvert size through the flow rate.
- Mark the first HW/D scale. Extend a horizontal line to the desired scale and read HW/D and note on Culvert Design Form (Appendix B).
- If the design case falls above the nomograph, the trial culvert size is too small.
- If the line falls below the nomograph, the trial culvert size is too large. Sometimes the minimum culvert size is controlled by factors other than hydraulics. If a calculated headwater surface is needed, see discussion in Appendix D.

d. Calculate headwater depth (HW_i).
- Multiply HW/D by D to obtain HW to energy gradeline.
- Neglecting the approach velocity HW_i = HW.
- Including the approach velocity HW_i = HW - approach velocity head.

8.4 Design Procedure (continued)

Step 8 Determine Outlet Control Headwater Depth At Inlet (HW_{oi})

a. Calculate the tailwater depth (TW) using the design flow rate and normal depth (single section) or using a water surface profile.
b. Calculate critical depth (d_c) using appropriate chart in Appendix A.

- Locate flow rate and read d_c.
- d_c cannot exceed D.
- If d_c > 0.9D, consult Handbook of Hydraulics (King and Brater) for a more accurate d_c, if needed, since curves are truncated where they converge.

c. Calculate (d_c + D)/2.

d. Determine (h_o).
h_o = the larger of TW or (d_c + D/2).
e. Determine (K_E).

- Entrance loss coefficient from Table 2 in the Appendix B.

f. Determine losses through the culvert barrel (H).

- Use nomograph (Appendix A) or equation 8.5 or 8.6 if outside range.
- Locate appropriate K_E scale.
- Locate culvert length (L) or (L_1):
  - use (L) if Manning's n matches the n value of the culvert and
  - use (L_1) to adjust for a different culvert n value.

$$\mathbf{L_1 = L(n_1/n)^2} \qquad \textbf{(8.10)}$$

Where: L_1= adjusted culvert length, ft
L = actual culvert length, ft
n_1 = desired Manning n value
n = Manning n value on chart

Mark point on turning line:
- use a straight edge and
- connect size with the length.

- Read (H):
  - use a straight edge,
  - connect Q and turning point and
  - read (H) on Head Loss scale.

g. Calculate outlet control headwater (HW_{oi}).

- Use equation 8.11, if V_u and V_d are neglected:

$$\mathbf{HW_{oi} = H + h_o - S_oL} \qquad \textbf{(8.11)}$$

- Use equation 8.1, 8.4c and 8.6 to include V_u and V_d.

8.4 Design Procedure (continued)

Step 8 Determine Outlet Control Headwater Depth At Inlet (HW_{oi}) (continued)

g. Calculate outlet control headwater (HW_{oi}). (continued)

- If HW_{oi} is less than 1.2D and control is outlet control:
  - the barrel may flow partly full,
  - the approximate method of using the greater of tailwater or $(d_c + D)/2$ may not be applicable,
  - backwater calculations should be used to check the result and
  - if the headwater depth falls below 0.75D, the approximate nomograph method shall not be used. A backwater analysis can be performed from the outlet to just inside the inlet and the ponded Headwater can be directly computed. See Appendix D.

Step 9 Determine Controlling Headwater (HW_c)

- Compare HW_i and HW_{oi}, use the higher as HW_c.

Step 10 If appropriate, Compute Discharge Over The Roadway (Q_r)

a. Calculate depth above the roadway (HW_r).

$HWr = HW_c - HW_{ov}$

HW_{ov} = height of road above inlet invert

b. If $HW_r <= 0$, $Q_r = 0$

If $HW_r > 0$, determine C_d from Appendix D

c. Determine length of roadway crest (L).

d. Calculate Q_r using equation 8.12.

$$\mathbf{Q_r = C_d\, L\, HW_r^{1.5}} \qquad \textbf{(8.12)}$$

Step 11 Compute Total Discharge (Q_t)

$$\mathbf{Q_t = Q_d + Q_r} \qquad \textbf{(8.13)}$$

Step 12 Calculate Outlet Velocity (V_o) And Depth (d_n)

If inlet control is the controlling headwater:

a. Calculate flow depth at culvert exit.
- use normal depth (d_n) or
- use water surface profile

b. Calculate flow area (A).

8.4 Design Procedure (continued)

Step 12 Calculate Outlet Velocity (V_o) And Depth (d_n) (continued)

c. Calculate exit velocity (V_o) = Q/A.

If outlet control is the controlling headwater:

a. Calculate flow depth at culvert exit.
   - use (d_c) if d_c > TW
   - use (TW) if d_c < TW < D
   - use (D) if D < TW

b. Calculate flow area (A).

c. Calculate exit velocity (V_o) = Q/A.

Step 13 Review Results

Compare alternative design with constraints and assumptions. If any of the following are exceeded, repeat steps 5 through 12:

- the barrel must have adequate cover,
- the length shall be close to the approximate length,
- the headwalls and wingwalls must fit site,
- the allowable headwater shall not be exceeded, and
- the allowable overtopping flood frequency shall not be exceeded.

Step 14 If needed, Plot Performance Curve

a. Repeat steps 6 through 12 with a range of discharges.
b. Use the following upper limit for discharge:
   - Q_{100} if $Q_o <= Q_{100}$
   - Q_{500} if $Q_o > Q_{100}$

Step 15 Related Designs

Consider the following options (See Sections 8.3.4 and 8.3.5).

- Energy dissipators if V_o is larger than the normal V in the downstream channel (See Energy Dissipator Chapter).
- Sediment control storage for sites with sediment concerns such as alluvial fans (See Erosion and Sediment Control Chapter and Appendix C).

Step 16 Documentation

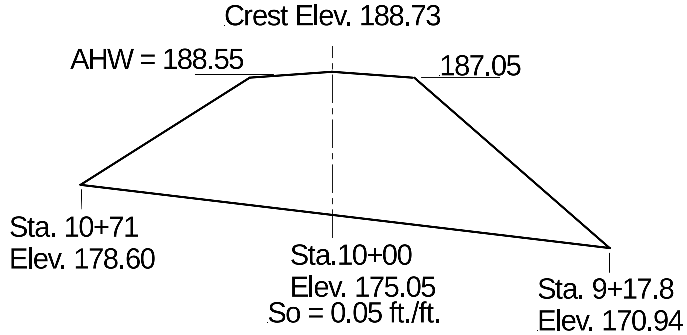
- See Documentation Chapter.
- Prepare report and file with background information.

8.5 Nomograph Design Example

The following example problem follows the Design Procedure Steps described in Section 8.4

Step 1 Assemble Site Data And Project File

a. Site survey Project file contains:
   - USGS, site, and location maps,
   - roadway profile, and
   - embankment cross-section



   Site visit notes indicate:
   - no sediment or debris problems, and
   - no nearby structures.

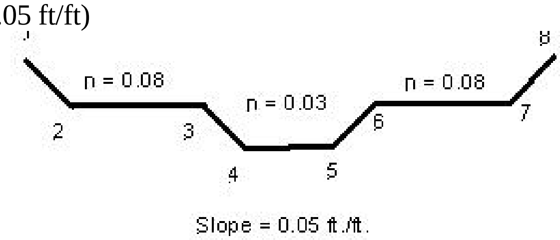
b. Studies by other agencies - none
c. Environmental, risk assessment shows:
   - no buildings near floodplain,
   - no sensitive floodplain values,
   - no FEMA involvement, and
   - convenient detours exist.
d. Design criteria:
   - 50-year frequency for design, and
   - 100-year frequency for overtopping check.

Step 2 Determine Hydrology

From file:
- Q_{50} = 600 cfs
- Q_{100} = 750 cfs

Step 3 Design Downstream Channel

Cross section of channel (Slope = .05 ft/ft)



8.5 Nomograph Design Example (continued)

Step 3 Design Downstream Channel (continued)

Point	Station, ft	Elevation, ft
1	12	180
2	22	175
3	32	174.5
4	34	172.5
5	39	172.5
6	41	174.5
7	51	175
8	61	180

The rating curve for the channel calculated by normal depth yields:

Q (cfs)	Elev. (ft.)	TW (ft)	V (ft/s)
0	172.50	0	0
75	173.70	1.7	10.23
150	174.28	1.78	12.52
225	174.69	2.19	14.35
300	174.99	2.49	15.94
375	175.23	2.73	17.17
450	175.43	2.93	18.18
525	175.63	3.13	19.10
600	175.80	3.30	19.92
675	175.97	3.47	20.68
750	176.12	3.62	21.39

Step 4 Summarize Data On Design Form

See Figure 8-10

Step 5 Select Design Alternative

Shape-box
Size – Minimum height = 6 ft
Material - concrete, n = .012
Entrance –beveled

Use Chart 9-2

Step 6 Select Design Discharge ($Q_d = Q_{50} = 600$ cfs)

8.5 Nomograph Design Example (continued)

Step 7 Determine Inlet Control Headwater Depth (HW_i)

Use inlet control nomograph - Chart 9 column 2

a. D = 6 ft

b. Allowable headwater = 188.55 – 177.19= 11.36’
trial H_W/D = 11.36/6.0 = 1.89
from Chart 9-2. q/B = 96 cfs.
therefore, B_{min}= 600/96 = 6.25’, use 8’
q/B = 600/8 = 75 cfs. OK.
H_W/D = 1.45. H_W = 1.45*6 = 8.7’

Step 8 Determine Outlet Control Headwater Depth At Inlet (HW_{oi})

a. TW = 3.3 ft for Q_{50} = 600 cfs
b. for q = 75 cfs, d_c = 5.6 ft from Chart 14, 4.8’ < 6’ OK.
c. (d_c + D)/2 = (5.6′ + 6′)/2 = 5.8 ft.
d. h_o = the larger of TW or (d_c + D/2)
h_o = (d_c + D)/2 = 5.80 ft
e. K_E = 0.2 from Table 2
f. Determine (H) - use Chart 15
- K_E scale = 0.2
- culvert length (L) = 93.8 ft, see page 8-33
- n = .012 same as on chart
- area = 48 ft^2
- H = 3.4 ft

g. $HW_{oi} = H + h_o - S_oL$ = 3.4′ + 5.80′ - (.05)*93.8’ = 4.41’
HW_{oi} is less than 1.2D, but control is inlet control.
Outlet control computations are for comparison only.

Step 9 Determine Controlling Headwater (HW_c)

- $HW_c = HW_i$ = 8.7 ft > HW_{oi} = 4.41 ft
- The culvert is in inlet control.

Step 10 Compute Discharge Over The Roadway (Q_r)

a. Calculate depth above the roadway:
HWr = $HW_c - HW_{ov}$ = 8.7′ - 11.36′ = - 2.66 ft
b. If $HW_r \leq 0$, $Q_r = 0$

Step 11 Compute Total Discharge (Q_t)

$Q_t = Q_d + Q_r$ = 600 cfs + 0 = 600 cfs

8.5 Nomograph Design Example (continued)

Step 12 Calculate Outlet Velocity (V_o) And Depth (d_n)

INLET CONTROL

a. Calculate normal depth (d_n):
   $Q = (1.49/n)A\ R^{2/3}\ S^{1/2} = 600$ cfs
   $= 124.2(8*d_n)[8*d_n/(8+2d_n)]^{2/3}(.05)^{.5}$
   $= (8*d_n)[8*d_n/(8+2d_n)]^{2/3} = 600/(124.2*(.05)^{.5}) = 21.60$
   try $d_n = 2.0$ ft, 19.4 < 21.6
   use $d_n = 2.16$ ft, 21.65 ≈ 21.6
b. $A = 2.16*8 = 17.28\ ft^2$
c. $V_o = Q/A = 600/17.28 = 34.7$ ft/s

Step 13 Review Results

Compare alternative design with constraints and assumptions. If any of the following are exceeded repeat, steps 5 through 12:

- headwalls and wingwalls fit site,
- allowable headwater, 11.36 ft > 8.7 ft is OK, and
- overtopping flood frequency > 50-year.

Step 14 Plot Performance Curve

Use Q_{100} for the upper limit. Steps 6 through 12 should be repeated for each discharge used to plot the performance curve. These computations are provided on the computation form that follows this example (see Figure 8-10).

Step 15 Related Designs

Consider the following options (see sections 9.3.4 and 9.3.5).

- Consider tapered inlets, culvert is in inlet control and has limited available headwater.
- No flow routing, a small upstream headwater pool exists.
- Consider energy dissipators since V_o = 35 ft/s > 18 ft/s in the downstream channel.
- No sediment problem.
- No fishery.

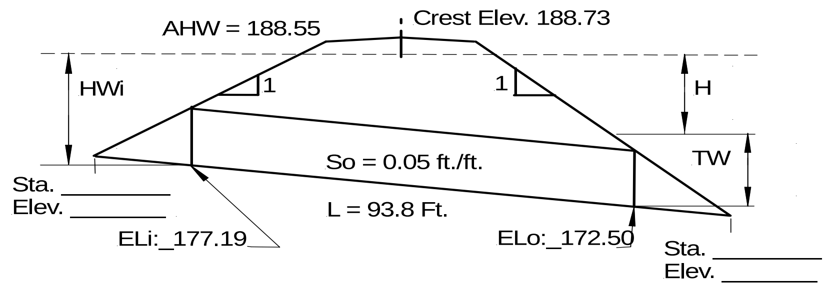
Step 16 Documentation

Prepare Report and file with background information

8.5 Nomograph Design Example (continued)

Project Name ____ADOT Hydraulics Manual______________________ Proj. No. __ADT064__________
Station/Location __Example 1______________________ Designer_G. Lopez-Cepero___ Date__2/11/04__
Subject ______________________ Checker__________ Date______

$El_{ho} = El_o + h_o + H$



Design Flows:

R.I (Years)	Flow (cfs)	T.W. (ft)
__50_____	_600____	_3.30___
__100____	_750____	_3.62___
________	________	________
________	________	________
________	________	________

For Table: $EL_{hi} = EL_i + Hw_i$

h_o = greater of TW or $(d_c+D)/2$

H from chart 15

Culvert Description:	1-10x6 with Bevel Entrance														
Total Flow (cfs)	Flow/Ft: (cfs)	HEADWATER CALCULATIONS											Control HW:	Vo Ft/sec:	Comments
		Inlet Control: Chart 9				Outlet Control: Chart 15									
		Hw_i/D	Hw_i	Fall	EL_{hi}	TW	d_c	$(d_c+D)/2$:	h_o	k_e	H	EL_{ho}			
600	60	1.26	7.56	--	184.75	3.30	4.8	5.6	5.6	0.2	2.8	180.90	184.75	33	184.75<188.55 OK
750	75	1.50	9.0	--	186.19	3.62	5.6	5.8	5.8	0.2	4.4	182.70	186.19	36	186.19<188.73 OT does not occur
150	15	0.5	3.0	--	180.19										
300	30	0.78	4.68	--	181.87										
450	45	1.04	6.24	--	183.43										

Figure 8-10 Culvert Chart for Design Example

8.6 Microcomputer Solution

8.6.1 Overview

Culvert hydraulic analysis can also be accomplished with the aid of the microcomputer software. The following example has been produced using the HY-8 Culvert Analysis Microcomputer Program. This is a computer solution of the data provided in Section 8.5.

8.6.2 Data Input

In creating a data input file, the user will be prompted for the discharge range, site data and culvert shape, size, material and inlet type. The discharge range for this example will be from 0 to 750 cfs.

As each group of data is entered the user is allowed to edit any incorrect entries. The following is the summary of the culvert information given the program.

Culvert Data

Let us use a 6 ft x 6 ft concrete box culvert as our initial trial size. For the culvert use a conventional inlet with 1:1 bevels and 45-degree wingwalls. The site data is entered by providing culvert invert data. If embankment data points are input, the program will fit the culvert in the fill and subtract the appropriate length.

CULVERT FILE: MAN_6x6	FHWA CULVERT ANALYSIS	DATE:01-06-2004
TAILWATER FILE: MAN_6x6	HY-8, VERSION 6.0	CULVERT NO. 1 OF 1

ITEM	SELECTED CULVERT
(1) BARREL SHAPE:	BOX
(2) BARREL SIZE	6.00 FT X 6.00 FT
(3) BARREL MATERIAL:	CONCRETE
(4) MANNING'S n:	.012
(5) INLET TYPE:	CONVENTIONAL
(6) INLET EDGE AND WALL:	BEVELED EDGE (1.5:1)
(7) INLET DEPRESSION:	NONE

8.6 Microcomputer Solution (continued)

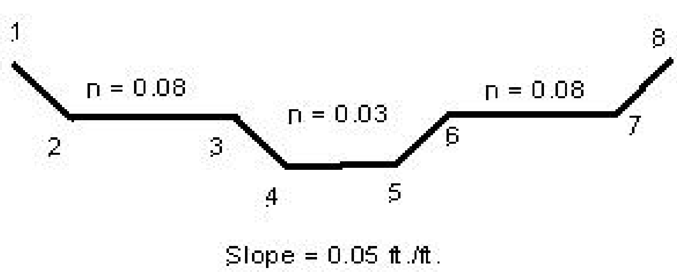
8.6.2 Data Input (continued)

Channel Data

Next the program will prompt for data pertaining to the channel so that tailwater elevations can be determined. Referring to the problem statement, the channel is irregularly shaped and can be described by the 8 coordinates listed. After opening the irregular channel file the user will be prompted for channel slope (.05), number of cross-section coordinates (8) and subchannel option. The subchannel option in this case would be option (2), left and right overbanks (n =.08) and main channel (n =.03).

The next prompt, for channel boundaries, refers to the number of the coordinate pair defining the left subchannel boundary and the number of the coordinate pair defining the right subchannel boundary. The boundaries for this example are the 3rd and 6th coordinates. After this is input, the program prompts for channel coordinates. Once these are entered, pressing (P) will cause the computer to display the channel cross-section shown below. The user can easily identify any input errors by glancing at the plot. To return to the data input screens, press any key. If data are correct press (return). The user can then enter the roughness data for the main channel and overbanks.

IRREGULAR CHANNEL CROSS-SECTION		
CROSS-SECTION	X	Y
COORD. NO	(FT.)	(FT.)
1	12	180
2	22	175
3	32	174.5
4	34	172.5
5	39	172.5
6	41	174.5
7	51	175
8	61	180



8.6 Microcomputer Solution (continued)

8.6.3 Rating Curve

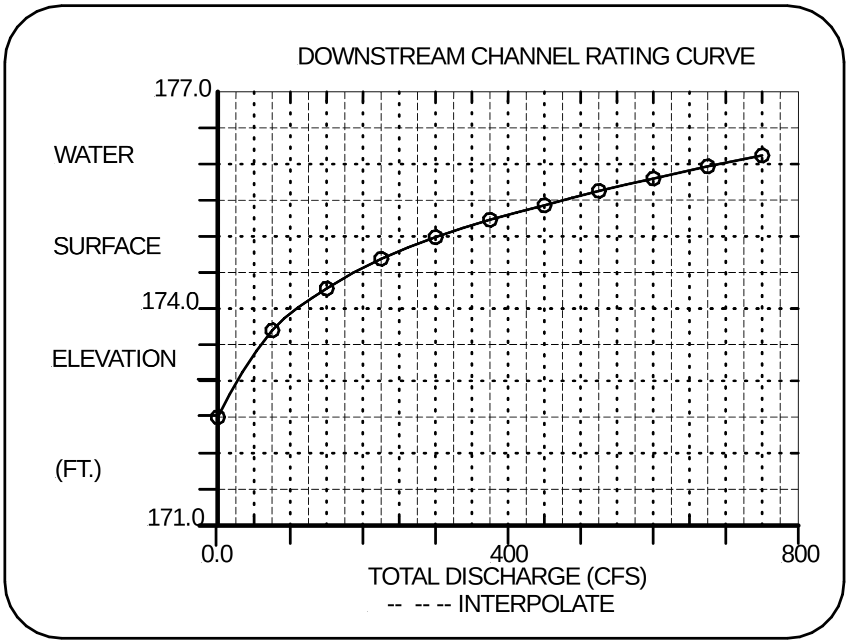
The program now has enough information to develop a uniform flow rating curve for the channel and provide the user with a list of options. Selecting option (T) on the Irregular Channel Data Menu will make the program compute the rating curve data and display the following table. Selecting option (I) will permit the user to interpolate data between calculated points.

[HY-8 screen]
TAILWATER RATING CURVE
IRREGULAR CHANNEL FILE: MAN_6

NO.	FLOW(CFS) (cfs)	W.S.E. (ft)	DEPTH (ft)	VEL. (f/s)	SHEAR (using R) (psf)
1	0.00	172.50	0.0	0.00	0.00
2	75.00	173.70	1.20	10.23	2.77
3	150.00	174.28	1.78	12.52	3.75
4	225.00	174.69	2.19	14.36	4.60
5	300.00	174.99	2.49	15.94	5.39
6	375.00	175.23	2.73	17.17	6.02
7	450.00	175.43	2.93	18.18	6.56
8	525.00	175.63	3.13	19.10	7.06
9	600.00	175.80	3.30	19.92	7.53
10	675.00	175.97	3.47	20.68	7.96
11	750.00	176.12	3.62	21.39	8.37

The Tailwater Rating Curve Table consists of tailwater elevation (W.S.E.), normal depth, natural channel velocity (Vel.) in feet per second, and the shear stress in pounds per square foot at the bottom of the channel for various flow rates. At the design flow rate of 600 cfs, the tailwater elevation will be 175.80 feet. The channel velocity will be 19.92 ft/s, and the shear will be 7.53 psf. This information is useful in the design of channel linings if they are needed. Entering (P) will cause the computer to display the rating curve for the channel. This curve, shown on the next page, is a plot of tailwater elevation vs. flow rate at the exit of the culvert.

8.6 Microcomputer Solution (continued)



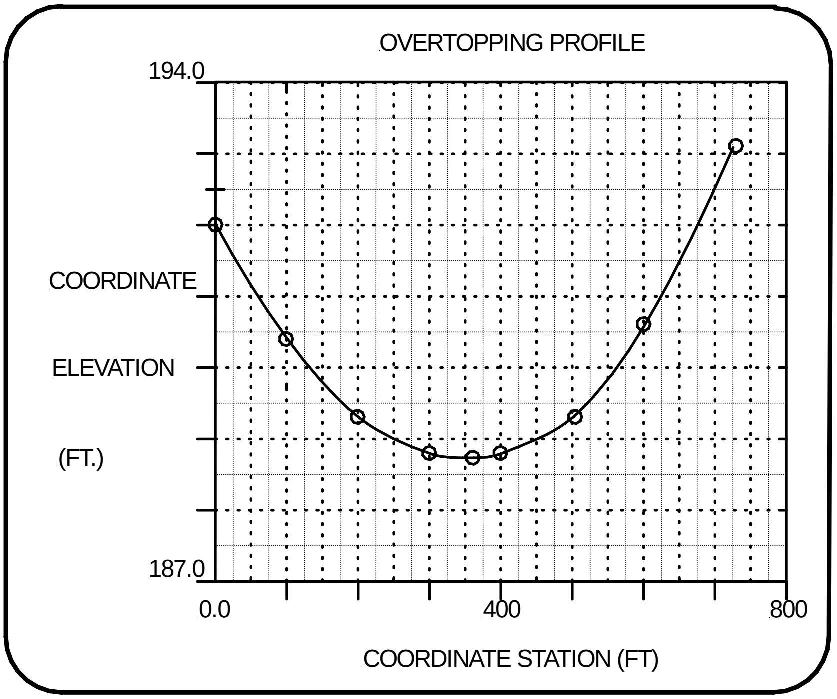
HY-8 Rating Curve

8.6.4 Roadway Data

The next prompts are for the roadway profile, so that an overtopping analysis can be performed. Referring to the problem statement, the roadway profile is a sag vertical curve, which will require nine coordinates to define. Once these coordinates are input, the profile will be displayed when (P) is entered, as illustrated below. The other data required for overtopping analysis are roadway surface or weir coefficient and the embankment top width. For this example, the roadway is paved with an embankment width of 50 feet.

ROADWAY PROFILE		
	STATION	ELEVATION
COORD. NO	(FT.)	(FT.)
1	0	192.0
2	100	190.4
3	200	189.3
4	300	188.8
5	360	188.73
6	400	188.8
7	500	189.3
8	600	190.6
9	720	193.1

8.6 Microcomputer Solution (continued)



HY-8 Overtopping Profile

8.6.5 Data Summary

All the data has now been entered and the summary table is displayed as shown below. At this point any of the data can be changed or the user can continue by pressing (Enter), which will bring up the Culvert Program Options Menu.

[HY-8 screen]

CULVERT FILE: MAN_6x6 FHWA CULVERT ANALYSIS DATE:01-06-2004
TAILWATER FILE: MAN_6x6 HY-8, VERSION 6.0 CULVERT NO. 1 OF 1
SUMMARY TABLE

C U L V NO.	A - SITE DATA INLET ELEV. (FT)	OUTLET ELEV. (FT)	CULVERT LENGTH (FT)	B - CULVERT SHAPE, MATERIAL, INLET BARRELS SHAPE MATERIAL	SPAN (FT)	RISE (FT)	MANN. n	INLET TYPE
1	177.19	172.50	93.93	1 - RCB	6.00	6.00	.012	CONV.
2								
3								

8.6 Microcomputer Solution (continued)

At this point, the program returns to the main menu. One may choose <S> Calculate or <M> Minimize. This feature, "Minimize Culvert Width" is intended to allow the designer to use HY-8 as a tool to perform culvert design for circular, box, elliptical, and arch shape culverts based on a user's defined allowable headwater elevation. This feature is activated by selecting letter "M". Once this letter is selected, the user inputs the allowable headwater elevation. That elevation will be the basis for adjusting the user's defined culvert size for the design discharge. The program will adjust the culvert span by increasing or decreasing by 0.5-foot increments. THE PROGRAM USES THE HEIGHT GIVEN; IT DOES NOT ADJUST THE HEIGHT OF THE CULVERT. It will compute the headwater elevation for the span, and compare it with the user's defined allowable headwater. If the computed headwater elevation is lower than or equal to the defined allowable headwater elevation the minimization routine will stop, and the adjusted culvert can be used for the remainder of the program.

8.6.6 Performance Curve (6 X 6)

At this point the data file can be saved or renamed by selecting option (S). The culvert performance curve table can be obtained by selecting option (N). If (N) is selected before (S) and an error occurs, the file can be retrieved by loading "current". When option (N) is selected, the program will compute the performance curve table without considering overtopping in the analysis. Since this 6 ft x 6 ft culvert is a preliminary estimate, the performance without considering overtopping is calculated and is shown below:

[HY-8 screen]

PERFORMANCE CURVE FOR CULVERT 1 - 1 (6.00 (ft) BY 6.00 (ft)) RCB

Q (cfs)	HWE (ft)	TWE (ft)	ICH (ft)	OCH (ft)		CCE (ft)	FCE (ft)	TCE (ft)	VO (ft/s)
0	177.19	172.50	0.00	-4.69	0.00	177.19	0.00	0.00	
75	179.61	173.70	2.42	-0.75	0.00	0.00	0.00	16.74	
150	181.13	174.28	3.94	0.04	0.00	0.00	0.00	19.33	
225	182.52	174.69	5.33	0.94	0.00	0.00	0.00	20.61	
300	183.87	174.99	6.68	1.99	0.00	0.00	0.00	21.98	
375	185.30	175.23	8.11	3.20	0.00	0.00	0.00	22.64	
450	186.90	175.43	9.71	4.58	0.00	0.00	0.00	23.42	
525	188.72	175.63	11.53	6.03		0.00	0.00	0.00	34.39
600	190.80	175.80	13.61	7.47		0.00	0.00	0.00	35.61
675	193.13	175.97	15.94	9.11		0.00	0.00	0.00	36.65
750	195.82	176.12	18.63	10.94		0.00	0.00	0.00	37.62

El. inlet face invert 177.19 ft El. outlet invert 172.50 ft
El. inlet throat invert 0.00 ft El. inlet crest 0.00 ft

This table indicates the controlling headwater elevation (HW), the tailwater elevation and the headwater elevations associated with all the possible control sections of the culvert. It is apparent from the table that at 600 cfs the headwater (HW) is 190.80 ft, which exceeds the design headwater of 188.73. Consequently, the 6 ft x 6 ft box culvert is inadequate to pass 600 cfs at the allowable headwater for the site conditions

8.6 Microcomputer Solution (continued)

8.6.7 Minimize Culvert

The user has the choice of using the minimize culvert or to select a new size by independent judgment. If minimize culvert is chosen, the program will request the allowable headwater elevation. The output is shown below. Several hydraulic parameters are also computed while performing the minimization routine. These hydraulic parameters which are part of the output of the minimization routine table, as shown below, must be printed from this screen because they are not printed with the output listing routine.

[HY-8 screen]

SUMMARY TABLE

CULV NO.	A - SITE DATA INLET ELEV. (FT)	OUTLET ELEV. (FT)	CULVERT LENGTH (FT)	B - CULVERT SHAPE, MATERIAL, INLET BARRELS SHAPE MATERIAL	SPAN (FT)	RISE (FT)	MANN. n	INLET TYPE
1	177.190	172.50	93.93	1 - RCB	7.00	6.00	.012	CONV.

HEADWATER ELEVATION		FLOW VELOCITY		FLOW DEPTHS	
ENTER ALLOWABLE	= 188.73	CULVERT	= 35.29	CULVERT	= 2.43
CONTROLLING	= 188.45	CHANNEL	= 19.92	CHANNEL	= 3.30
INLET CONTROL	= 188.45	DISCHARGE	= 600.00	NORMAL	= 2.43
OUTLET CONTROL	= 182.96	SLOPE	= 0.0500	CRITICAL	= 6.00

MAX. HEADWATER <ENTER> TO CHANGE HEADWATER <ANY KEY> TO CONT.

This feature may be a time saver for designers because it avoids the need for repetitively editing a culvert size to obtain a controlling headwater elevation.

8.6 Microcomputer Solution (continued)

8.6.8 Trial 2, 8 x 6 Culvert

Since the design headwater criterion has not been met, another size must be selected. Based on the results of the minimize culvert, try an 8 ft x 6 ft culvert, and modify the file accordingly. The resulting performance table shown below indicates that the design headwater will not be exceeded at 600 cfs. However, the headwater elevation of 188.98 feet at 750 cfs indicates that some overtopping will occur due to the 100-year storm.

[HY-8 screen]

PERFORMANCE CURVE FOR CULVERT 1 - 1 (8.00 (ft) BY 6.00 (ft)) RCB

Q (cfs)	HWE (ft)	TWE (ft)	ICH (ft)	OCH (ft)	CCE (ft)	FCE (ft)	TCE (ft)	VO (ft/s)
0	177.19	172.50	0.00	-4.69	0.00	177.19	0.00	0.00
75	179.18	173.70	1.99	-0.94	0.00	0.00	0.00	15.88
150	180.38	174.28	3.19	-0.37	0.00	0.00	0.00	18.51
225	181.49	174.69	4.30	0.24	0.00	0.00	0.00	19.90
300	182.52	174.99	5.33	0.92	0.00	0.00	0.00	20.96
375	183.53	175.23	6.34	1.68	0.00	0.00	0.00	21.75
450	184.57	175.43	7.38	2.52	0.00	0.00	0.00	22.46
525	185.68	175.63	8.49	3.46	0.00	0.00	0.00	23.43
600	186.90	175.80	9.71	4.49	0.00	0.00	0.00	23.64
675	188.25	175.97	11.06	5.58	0.00	0.00	0.00	36.03
750	189.73	176.12	12.54	6.58	0.00	0.00	0.00	37.18

El. inlet face invert 177.19 ft   El. outlet invert 172.50 ft
El. inlet throat invert 0.00 ft   El. inlet crest 0.00 ft

8.6 Microcomputer Solution (continued)

8.6.9 Overtopping Performance Curve

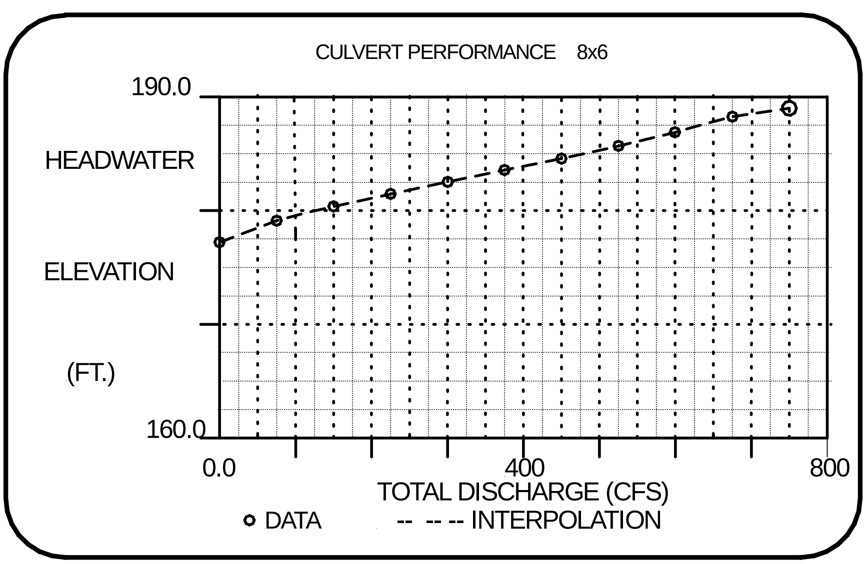
When overtopping occurs, the performance of the culvert will differ from that without overtopping. By selecting option (2), the culvert performance data can be obtained. The user also has the option to plot these data. To determine the amount of overtopping and the actual headwater, press (return), and then select (O) for overtopping. A Summary of Culvert Flows will appear on the screen, as shown below:

[HY-8 screen]

SUMMARY OF CULVERT FLOWS (CFS) FILE: MAN_8X6 DATE: 01-06-2004

ELEV(FT)	TOTAL	1	2	3	4	5	6	OT	ITER
177.19	0	0	0	0	0	0	0	0	0
179.18	75	75	0	0	0	0	0	0	1
180.38	150	150	0	0	0	0	0	0	1
181.49	225	225	0	0	0	0	0	0	1
182.52	300	300	0	0	0	0	0	0	1
183.53	375	375	0	0	0	0	0	0	1
184.57	450	450	0	0	0	0	0	0	1
185.68	525	525	0	0	0	0	0	0	1
186.90	600	600	0	0	0	0	0	0	1
188.25	675	675	0	0	0	0	0	0	1
188.98	750	713	0	0	0	0	0	34	6

This computation table is used when overtopping and/or multiple culvert barrels are used. It shows the headwater, total flow rate, the flow through each barrel and overtopping flow, and the number of iterations it took to balance the flows. From this information a total (culvert and overtopping) performance curve can be obtained by selecting option (1). This curve is a plot of the headwater elevation vs. the total flow rate which indicates how the culvert or group of culverts will perform over the selected range of discharges.



Performance curve

8.6 Microcomputer Solution (continued)

8.6.10 Review

From the Summary table, when the total flow is 750 cfs, 716 cfs passes through the culvert and 34 cfs flows over the road. The headwater elevation will be 188.98 feet. Assume that in this case overtopping at 100-year frequency can be tolerated, and the 8 ft x 6 ft culvert will be used. Referring back to the performance curve data, the outlet velocity at 600 cfs is 23.6 ft/s.

[HY-8 screen]

CULVERT # 1 PERFORMANCE CURVE NUMBER 1 BARREL(S)

Q (cfs)	HWE (ft)	TWE (ft)	ICH (ft)	OCH (ft)	CCE (ft)	FCE (ft)	TCE (ft)	VO (ft/s)
0	177.19	172.50	0.00	-4.69	0.00	177.19	0.00	0.00
75	179.18	173.70	1.99	-0.94	0.00	0.00	0.00	15.88
150	180.38	174.28	3.19	-0.37	0.00	0.00	0.00	18.51
225	181.49	174.69	4.30	0.24	0.00	0.00	0.00	19.90
300	182.52	174.99	5.33	0.92	0.00	0.00	0.00	20.96
375	183.53	175.23	6.34	1.68	0.00	0.00	0.00	21.75
450	184.57	175.43	7.38	2.52	0.00	0.00	0.00	22.46
525	185.68	175.63	8.49	3.46	0.00	0.00	0.00	23.43
600	186.90	175.80	9.71	4.49	0.00	0.00	0.00	23.64
675	188.25	175.97	11.06	5.58	0.00	0.00	0.00	36.03
712.85	188.98	176.12	11.79	6.07	0.00	0.00	0.00	36.60

El. inlet face invert	187.50 ft	El. outlet invert	172.50 ft
El. inlet throat invert	0.00 ft	El. inlet crest	0.00 ft

8.7 References

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Appendix 8.A Design Charts

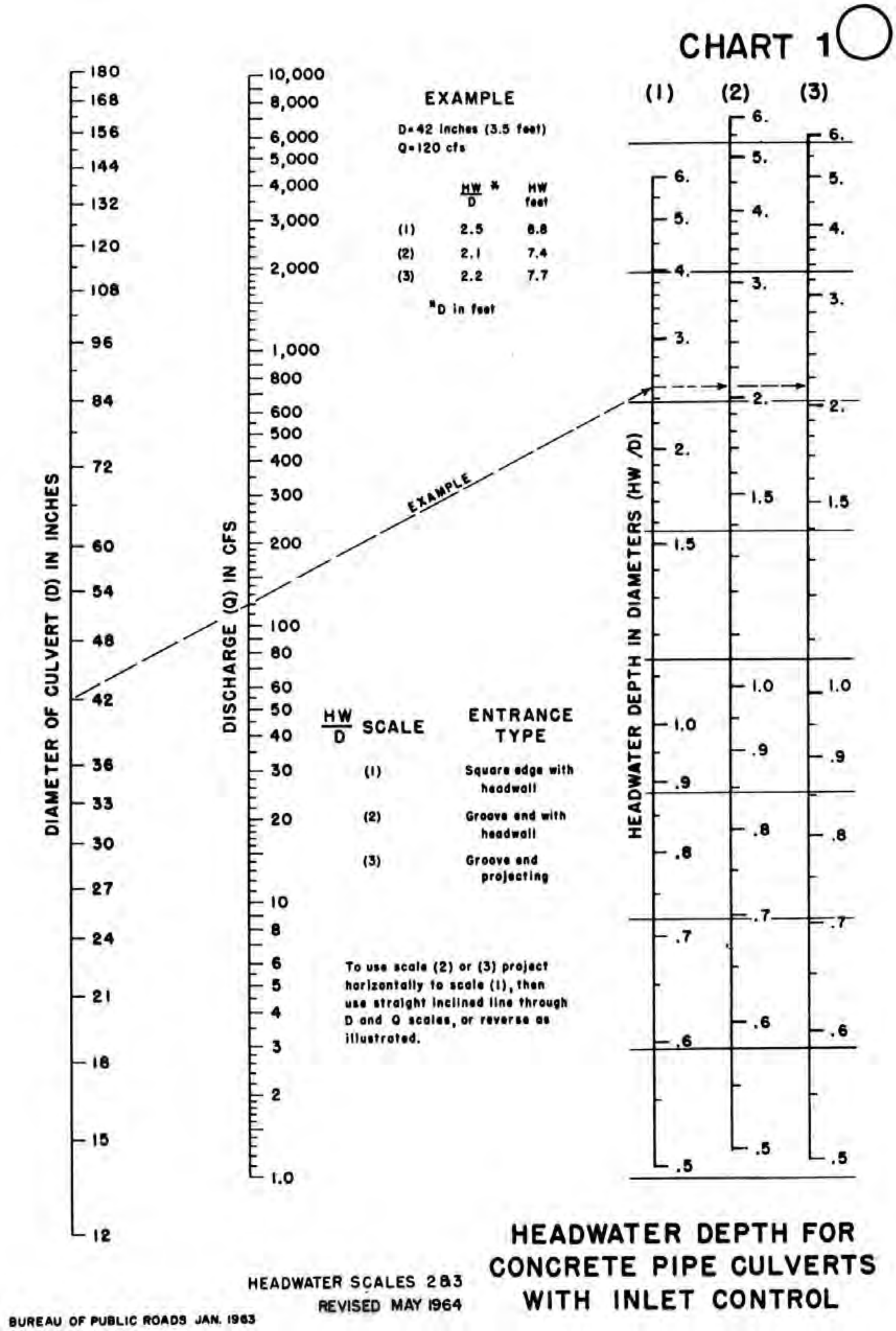
Culverts constructed in accordance with ADOT Standard Drawings should use the following charts.

Concrete Pipe -- Inlet Control Chart 1
- Groove end with headwall -Line 2
- Groove end Projecting -Line 3

Corrugated Metal Pipe Chart – Inlet Control Chart 2
- Mitered to conform to slope -Line 2
- End projecting -Line 3

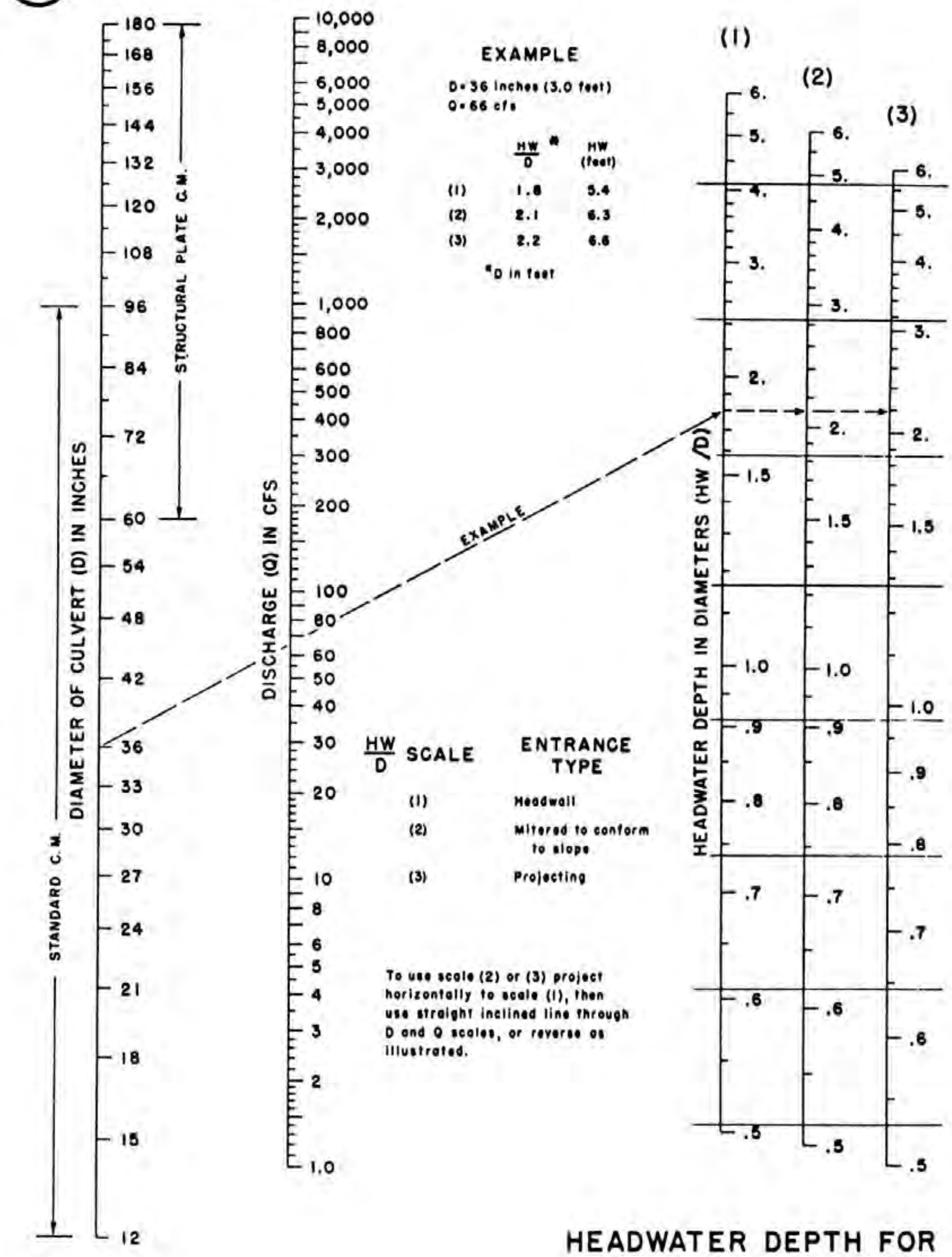
Corrugated Metal Pipe Chart – Inlet Control Chart 3
- Beveled with Headwall -Line B

Concrete Box Culvert Chart – Inlet Control Chart 9
- Beveled with Headwall -Line 2



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

CHART 2

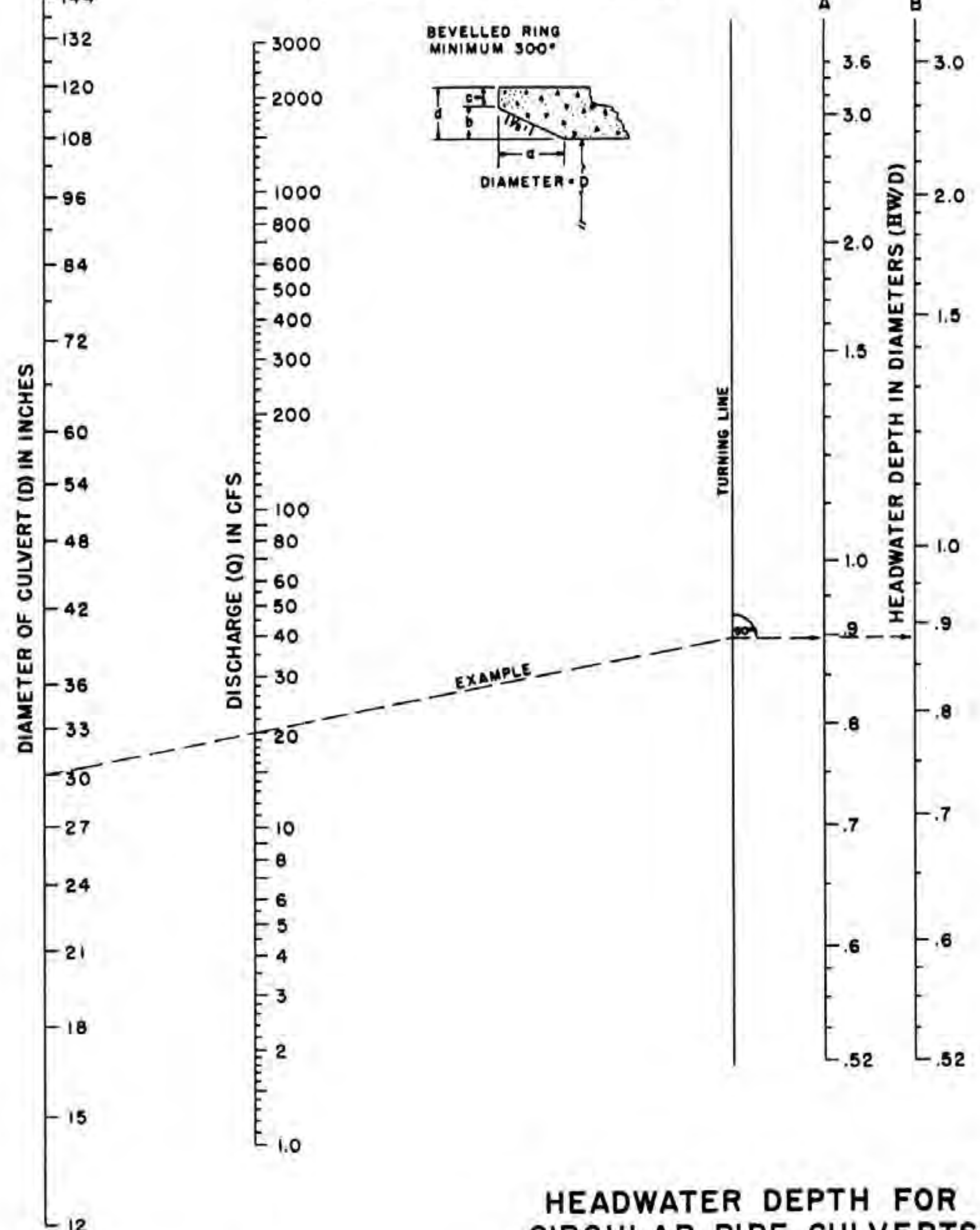


HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 3

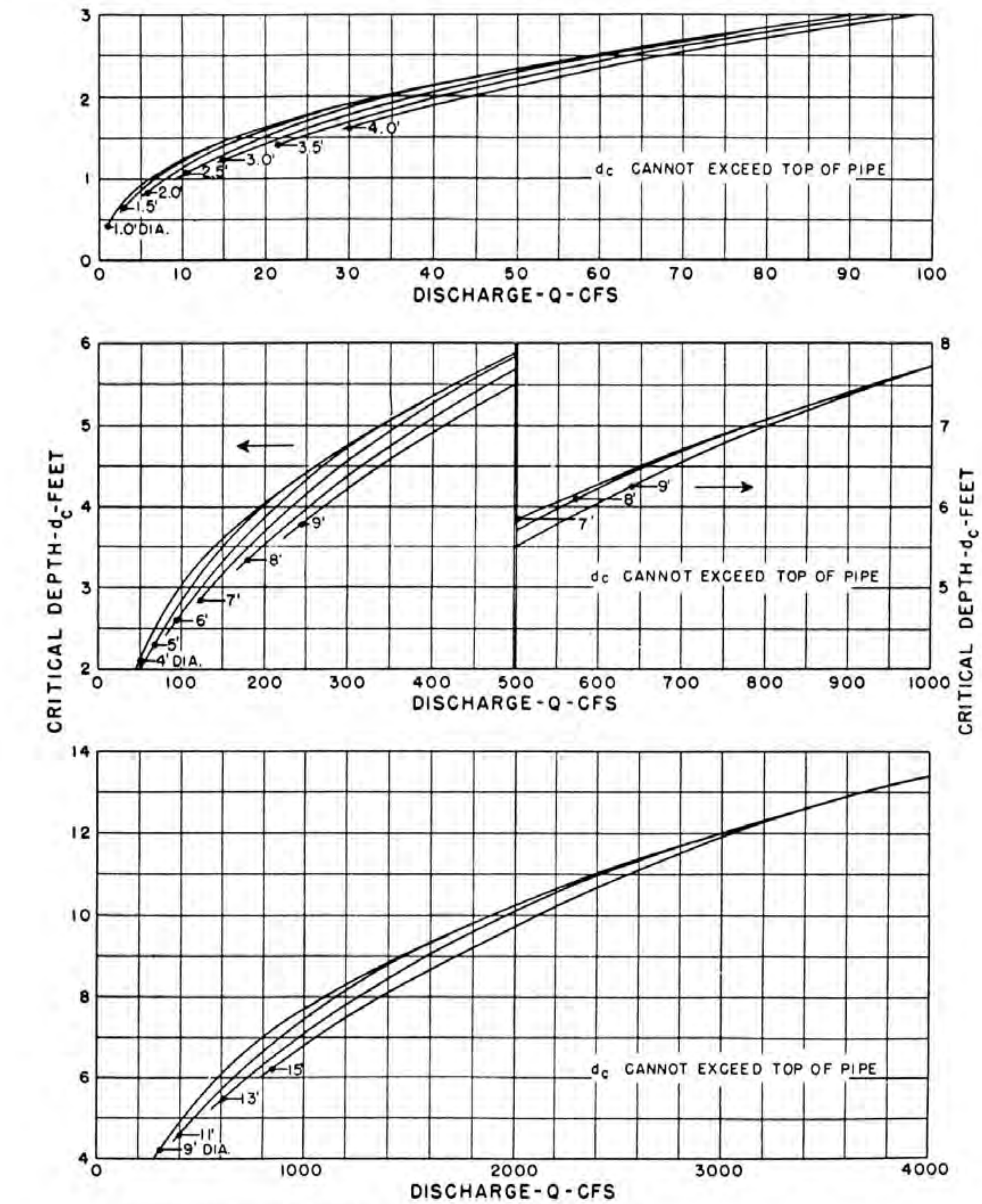
b/D	a/D	c/D	d/D	ENTRANCE TYPE
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B



)ERAL HIGHWAY ADMINISTRATION
MAY 1973

HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL

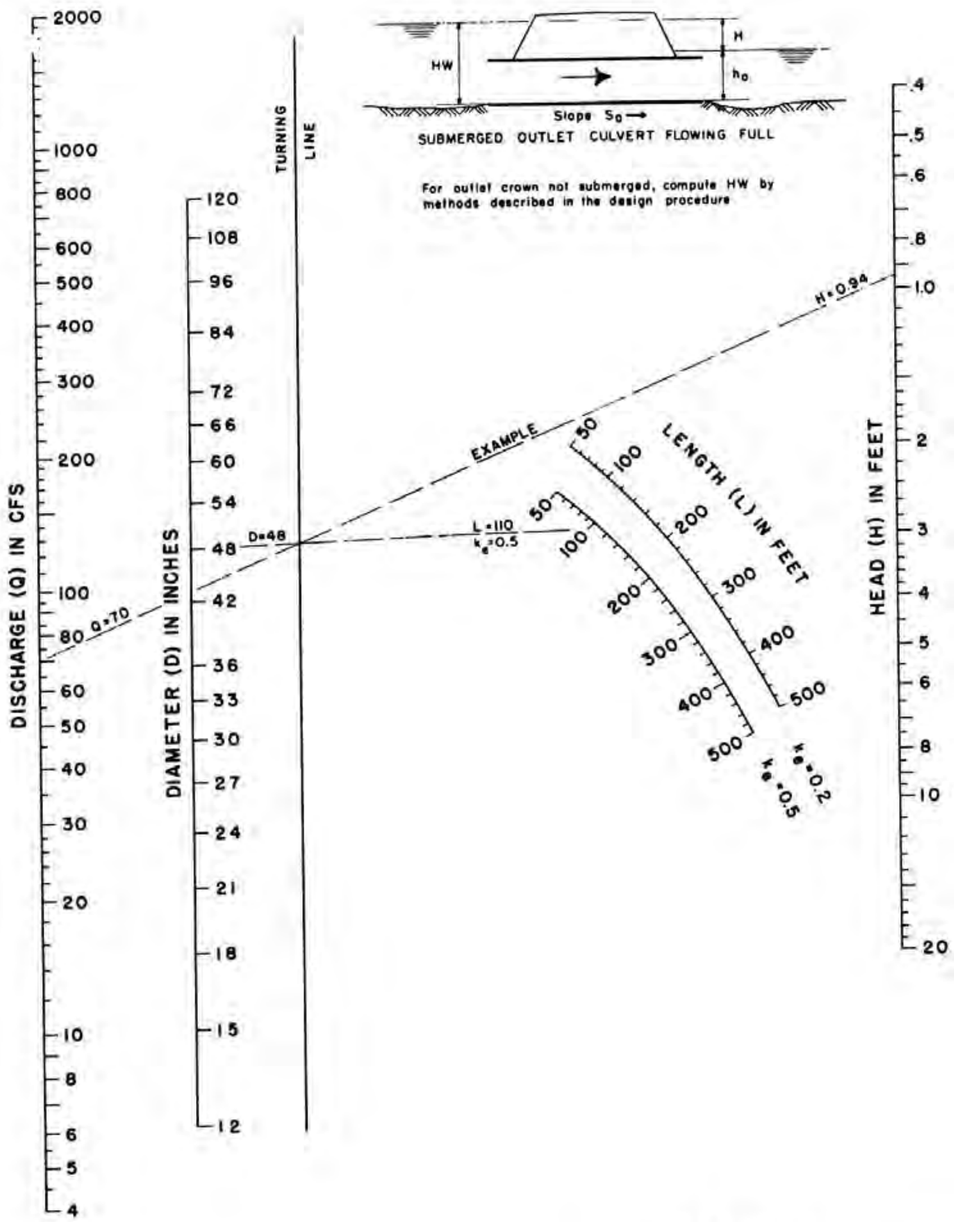
CHART 4



BUREAU OF PUBLIC ROADS
JAN. 1964

CRITICAL DEPTH CIRCULAR PIPE

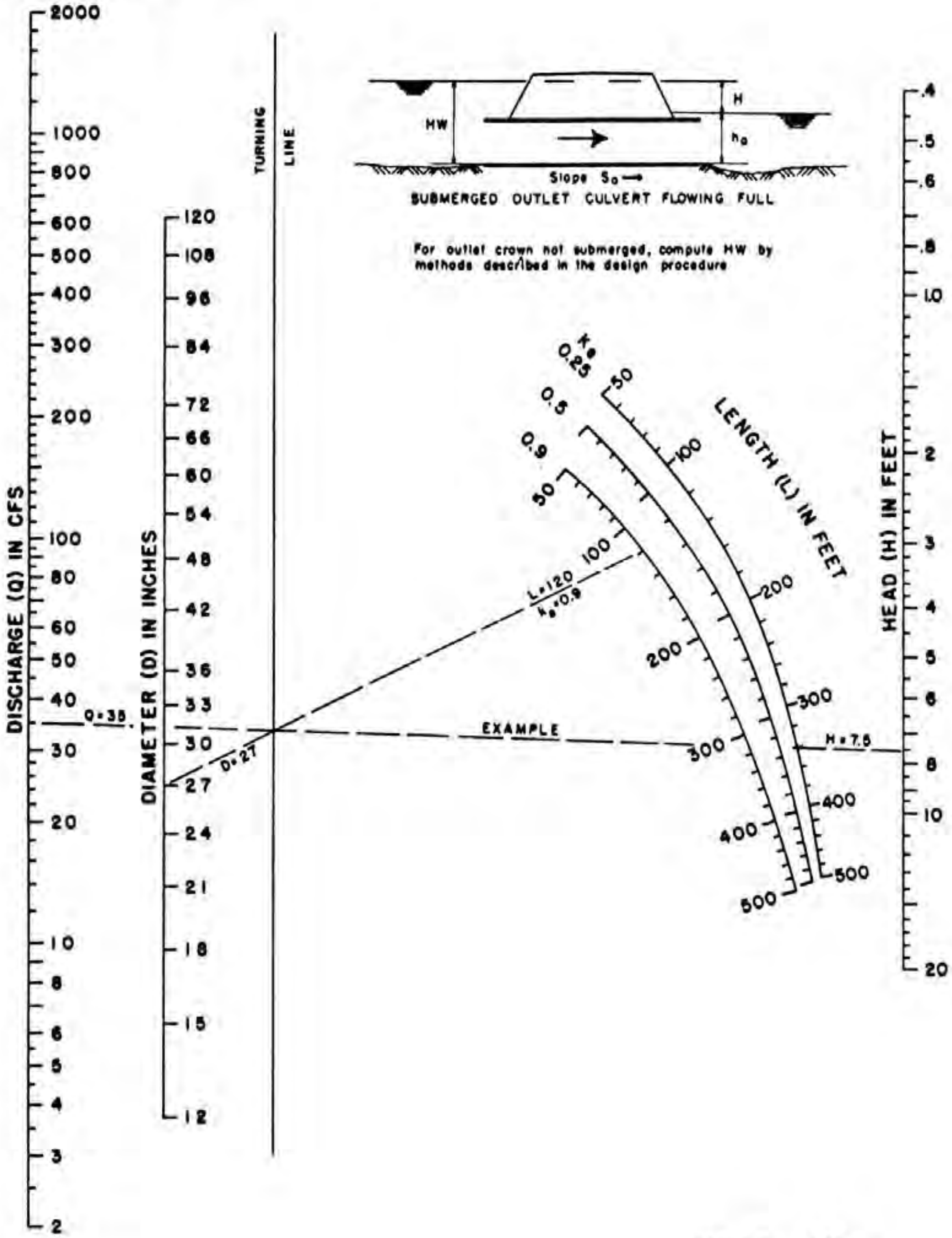
CHART 5



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 6



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

BUREAU OF PUBLIC ROADS JAN. 1963

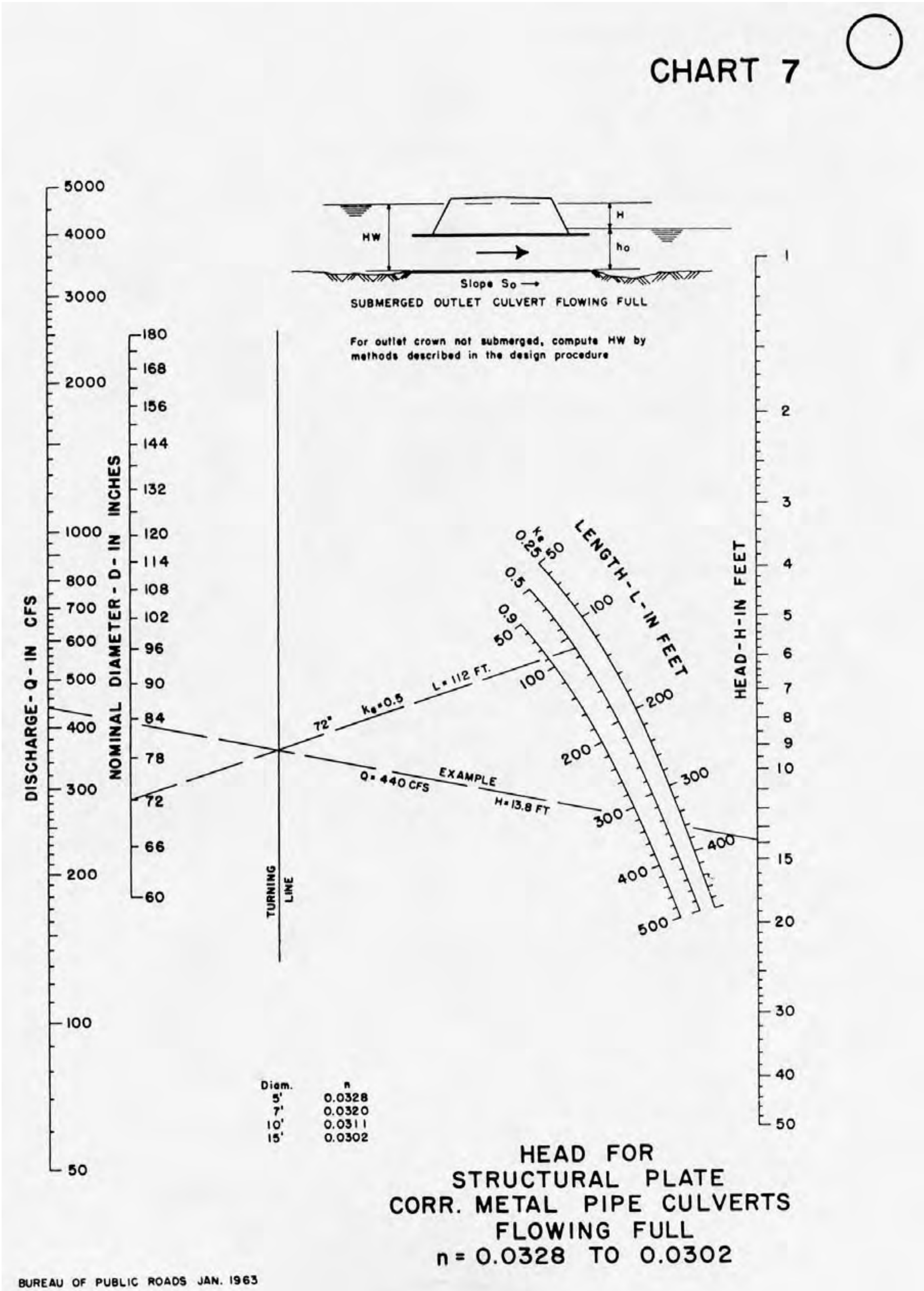
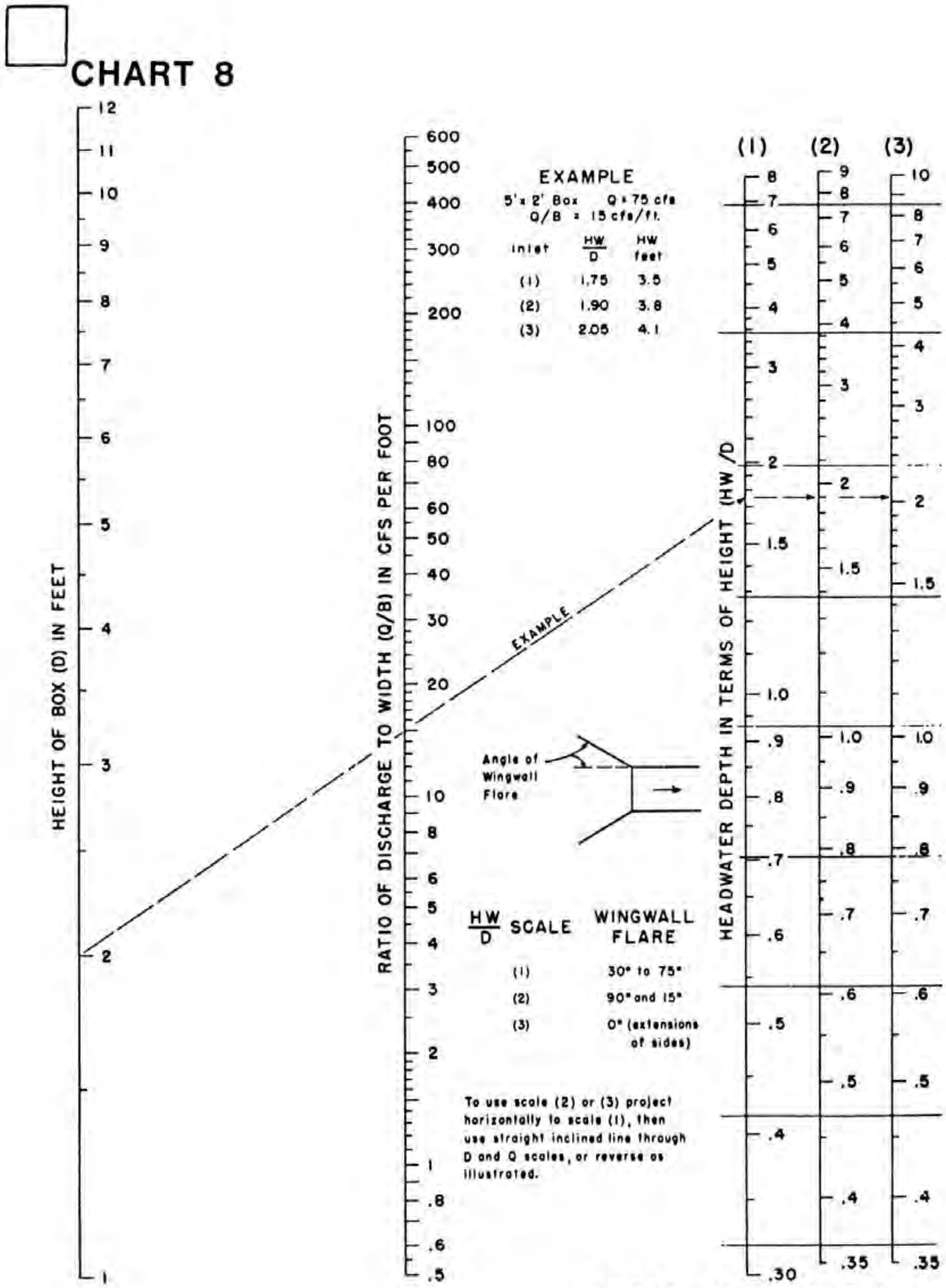


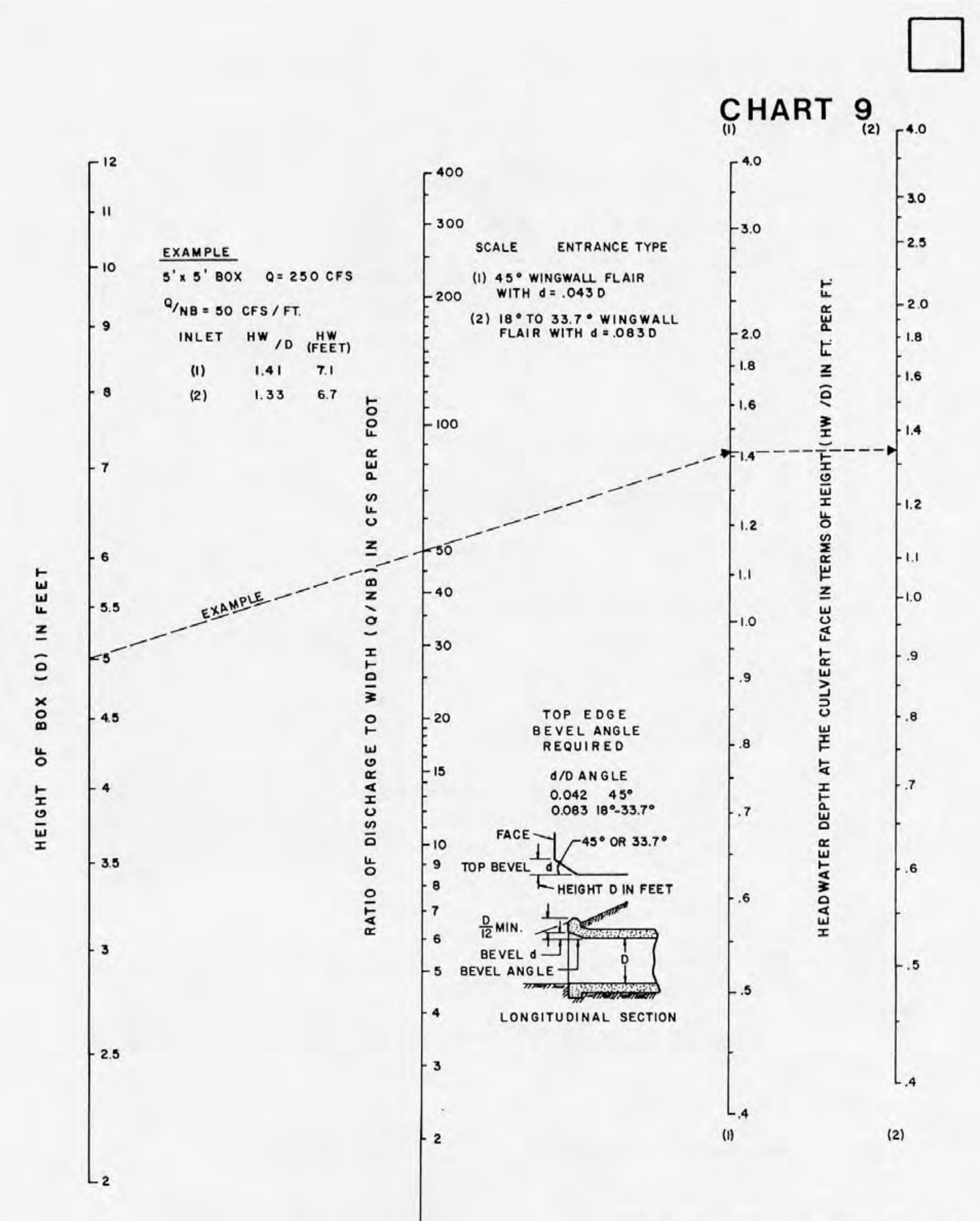
CHART 7
HW
H
ho
Slope So
SUBMERGED OUTLET CULVERT FLOWING FULL
For outlet crown not submerged, compute HW by methods described in the design procedure
DISCHARGE - Q - IN CFS
NOMINAL DIAMETER - D - IN INCHES
TURNING LINE
LENGTH - L - IN FEET
HEAD - H - IN FEET
ke
EXAMPLE
Q = 440 CFS
H = 13.8 FT
L = 112 FT.
ke = 0.5
72"
Diam. n
5' 0.0328
7' 0.0320
10' 0.0311
15' 0.0302
HEAD FOR STRUCTURAL PLATE CORR. METAL PIPE CULVERTS FLOWING FULL
n = 0.0328 TO 0.0302
BUREAU OF PUBLIC ROADS JAN. 1963

CHART 8



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS 18° TO 33.7° & 45°
WITH BEVELED EDGE AT TOP OF INLET

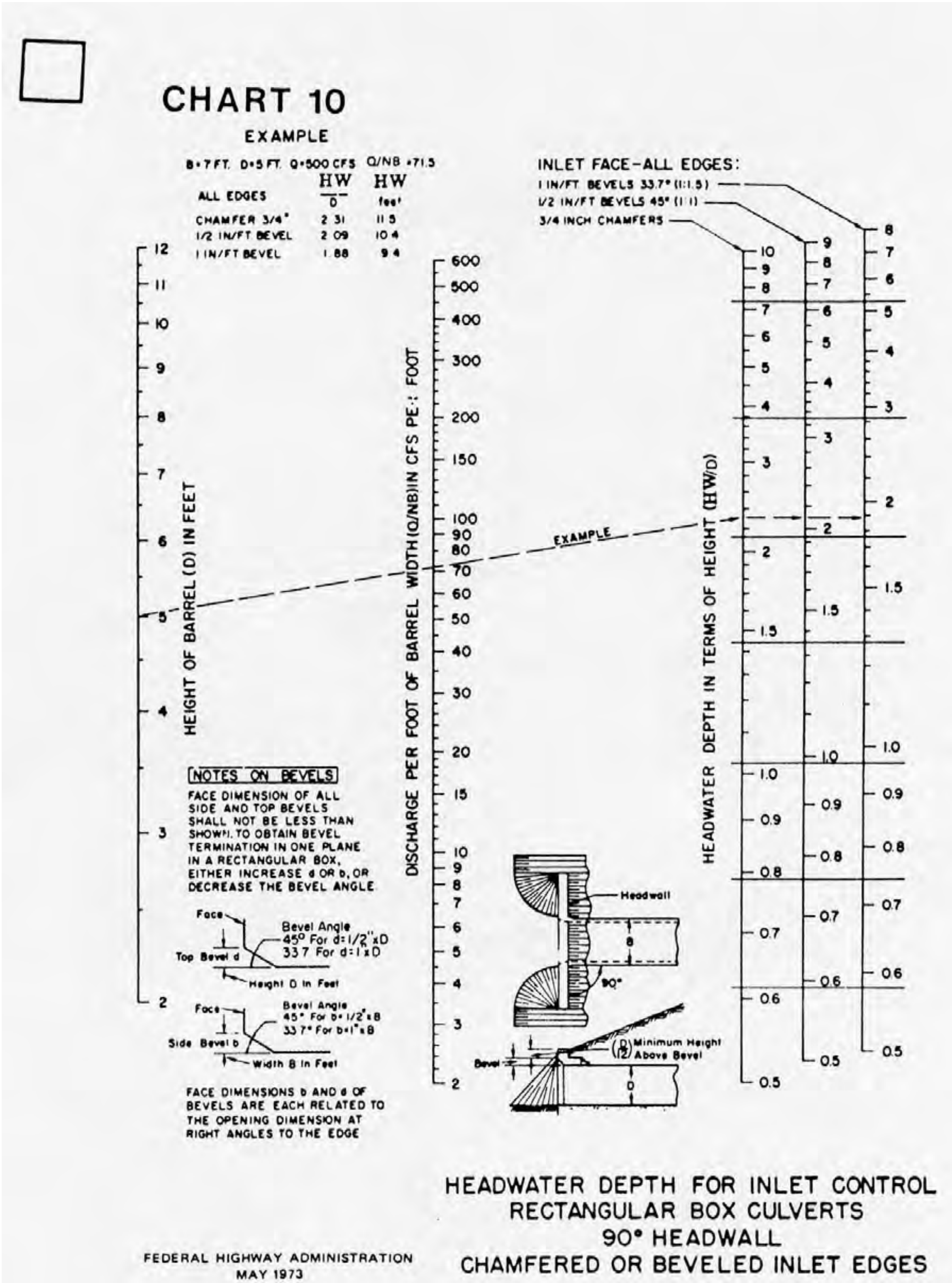
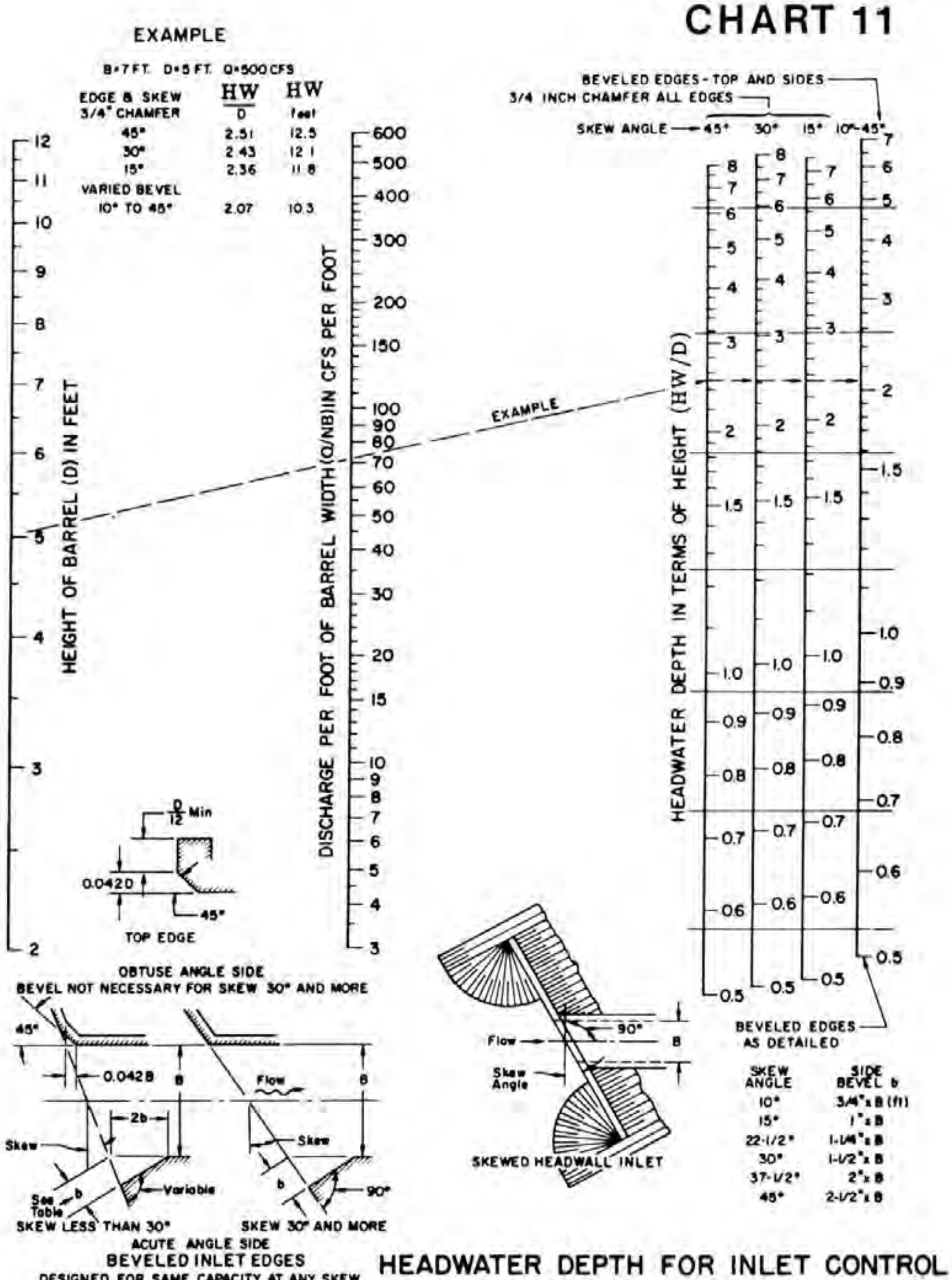


CHART 10
EXAMPLE
B=7 FT. D=5 FT. Q=500 CFS Q/NB =71.5
ALL EDGES HW/D HW feet
CHAMFER 3/4" 2.31 11.5
1/2 IN/FT BEVEL 2.09 10.4
1 IN/FT BEVEL 1.88 9.4
INLET FACE-ALL EDGES:
1 IN/FT BEVELS 33.7° (1:1.5)
1/2 IN/FT BEVELS 45° (1:1)
3/4 INCH CHAMFERS
HEIGHT OF BARREL (D) IN FEET
DISCHARGE PER FOOT OF BARREL WIDTH (Q/NB) IN CFS PER FOOT
HEADWATER DEPTH IN TERMS OF HEIGHT (HW/D)
EXAMPLE
NOTES ON BEVELS
FACE DIMENSION OF ALL SIDE AND TOP BEVELS SHALL NOT BE LESS THAN SHOWN. TO OBTAIN BEVEL TERMINATION IN ONE PLANE IN A RECTANGULAR BOX, EITHER INCREASE d OR b, OR DECREASE THE BEVEL ANGLE.
Face
Bevel Angle
45° For d=1/2"xD
33.7 For d=1"xD
Top Bevel d
Height D in Feet
Face
Bevel Angle
45° For b=1/2"xB
33.7° For b=1"xB
Side Bevel b
Width B in Feet
FACE DIMENSIONS b AND d OF BEVELS ARE EACH RELATED TO THE OPENING DIMENSION AT RIGHT ANGLES TO THE EDGE
Headwall
B
90°
(D/12) Minimum Height Above Bevel
Bevel
D
HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
90° HEADWALL
CHAMFERED OR BEVELED INLET EDGES
FEDERAL HIGHWAY ADMINISTRATION
MAY 1973

CHART 11



FEDERAL HIGHWAY ADMINISTRATION
MAY 1973

HEADWATER DEPTH FOR INLET CONTROL
SINGLE BARREL BOX CULVERTS
SKEWED HEADWALLS
CHAMFERED OR BEVELED INLET EDGES

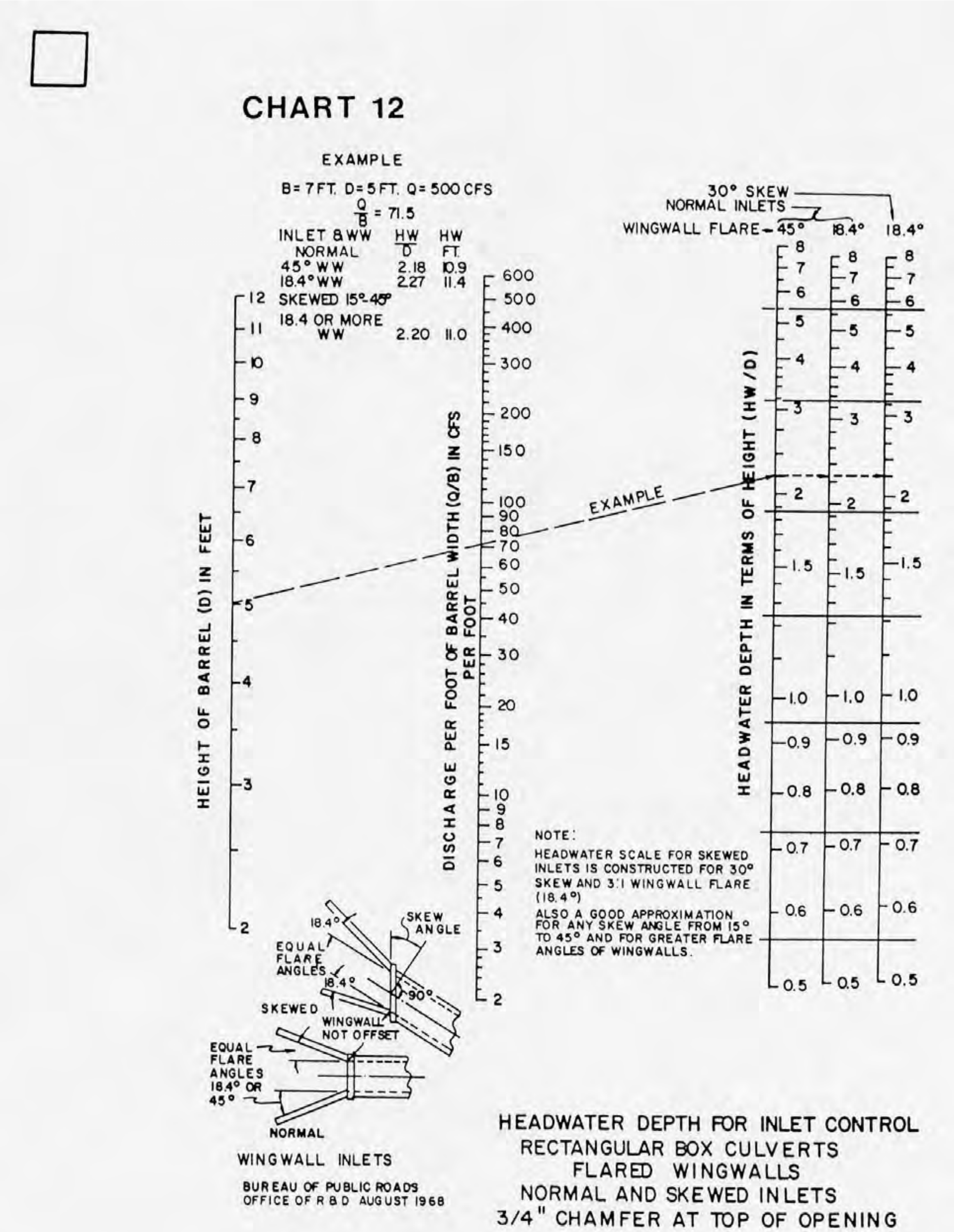
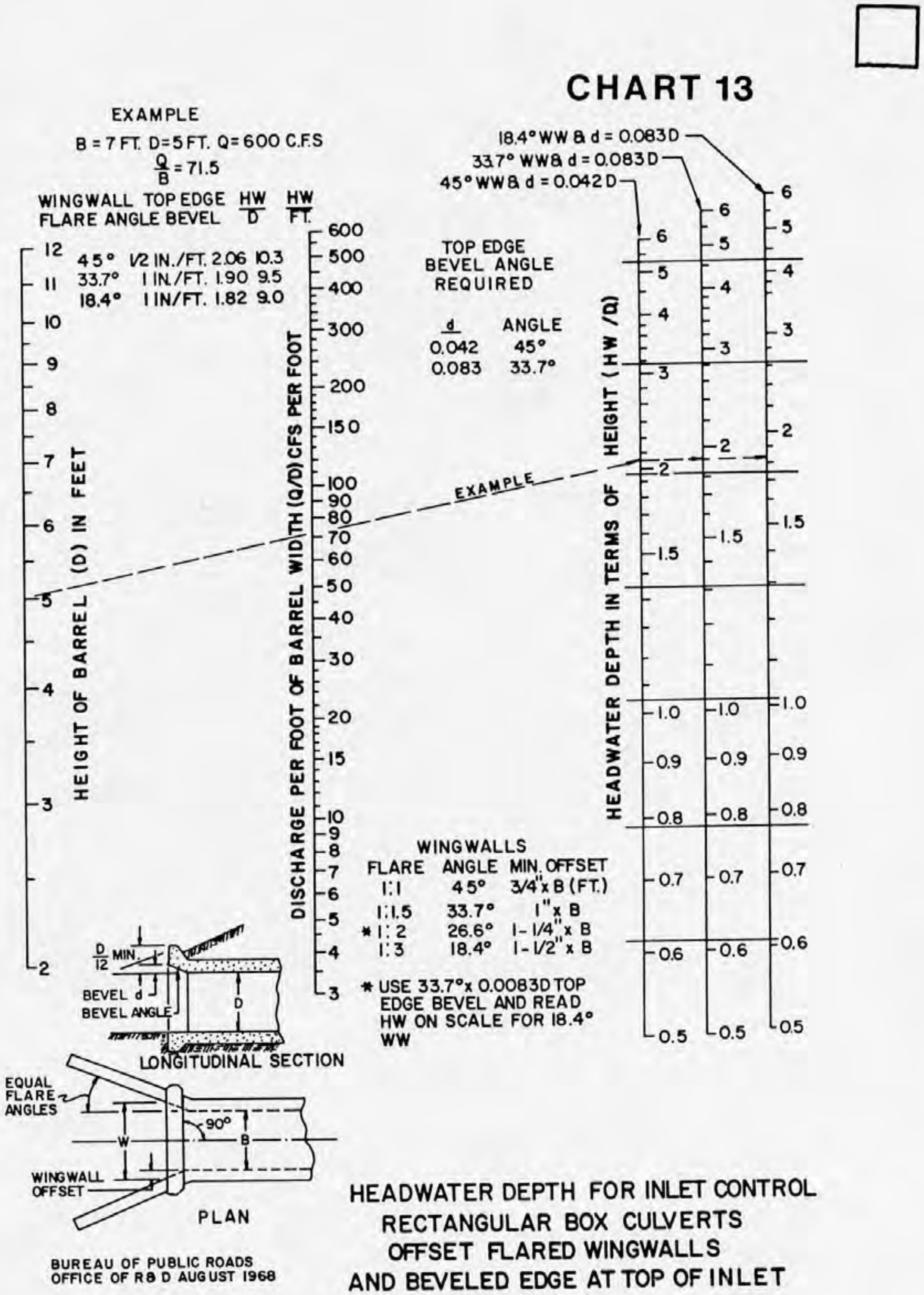
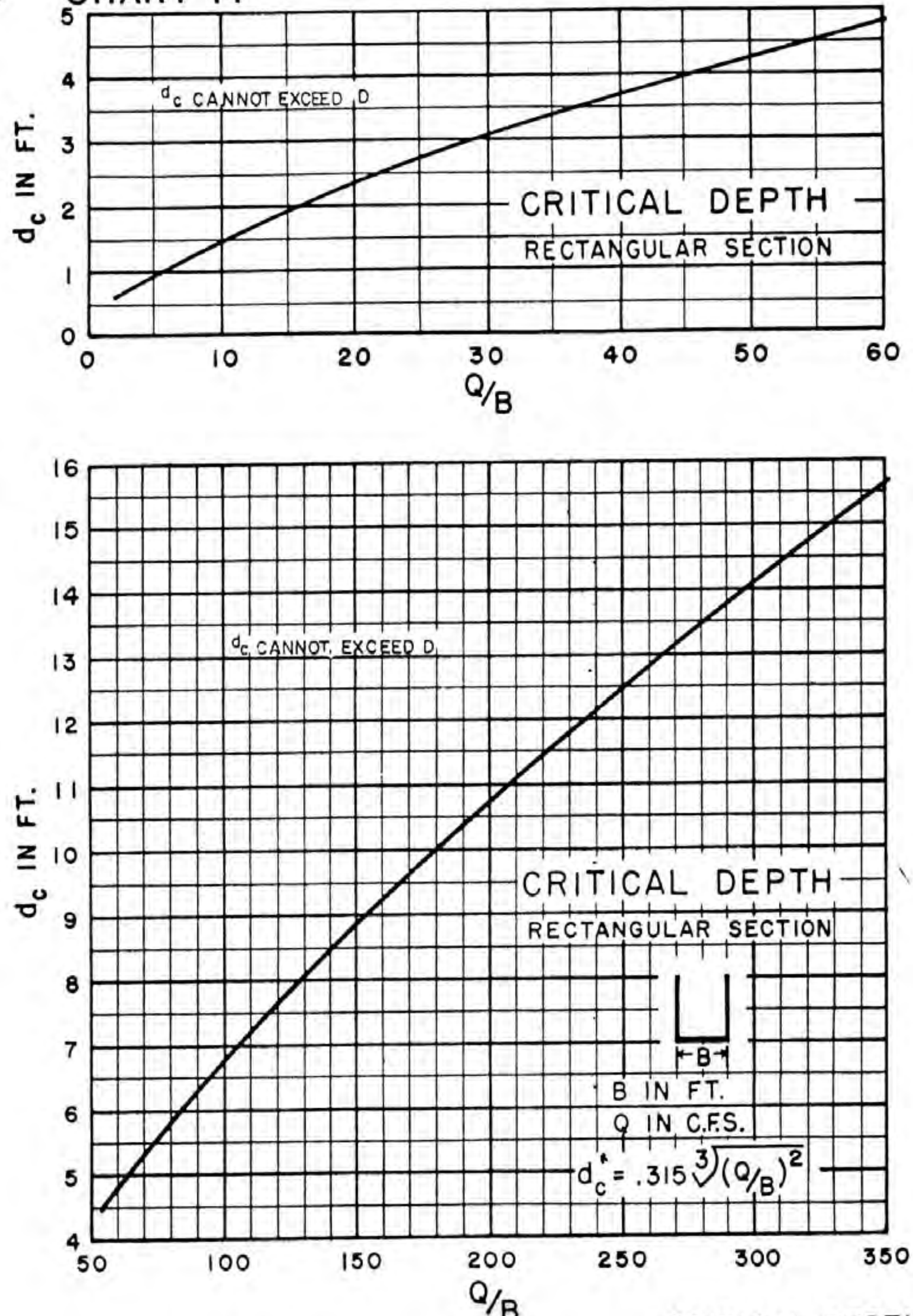


CHART 12
EXAMPLE
B= 7 FT. D= 5 FT. Q= 500 CFS
Q/B = 71.5
INLET & WW | HW/D | HW FT.
NORMAL
45° WW 2.18 10.9
18.4° WW 2.27 11.4
SKEWED 15°-45°
18.4 OR MORE WW 2.20 11.0
HEIGHT OF BARREL (D) IN FEET
DISCHARGE PER FOOT OF BARREL WIDTH (Q/B) IN CFS PER FOOT
HEADWATER DEPTH IN TERMS OF HEIGHT (HW/D)
EXAMPLE
30° SKEW
NORMAL INLETS
WINGWALL FLARE 45° 18.4° 18.4°
NOTE:
HEADWATER SCALE FOR SKEWED INLETS IS CONSTRUCTED FOR 30° SKEW AND 3:1 WINGWALL FLARE (18.4°)
ALSO A GOOD APPROXIMATION FOR ANY SKEW ANGLE FROM 15° TO 45° AND FOR GREATER FLARE ANGLES OF WINGWALLS.
SKEW ANGLE
EQUAL FLARE ANGLES
SKEWED
WINGWALL NOT OFFSET
EQUAL FLARE ANGLES 18.4° OR 45°
NORMAL
WINGWALL INLETS
BUREAU OF PUBLIC ROADS
OFFICE OF R B D AUGUST 1968
HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS
NORMAL AND SKEWED INLETS
3/4" CHAMFER AT TOP OF OPENING



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
OFFSET FLARED WINGWALLS
AND BEVELED EDGE AT TOP OF INLET

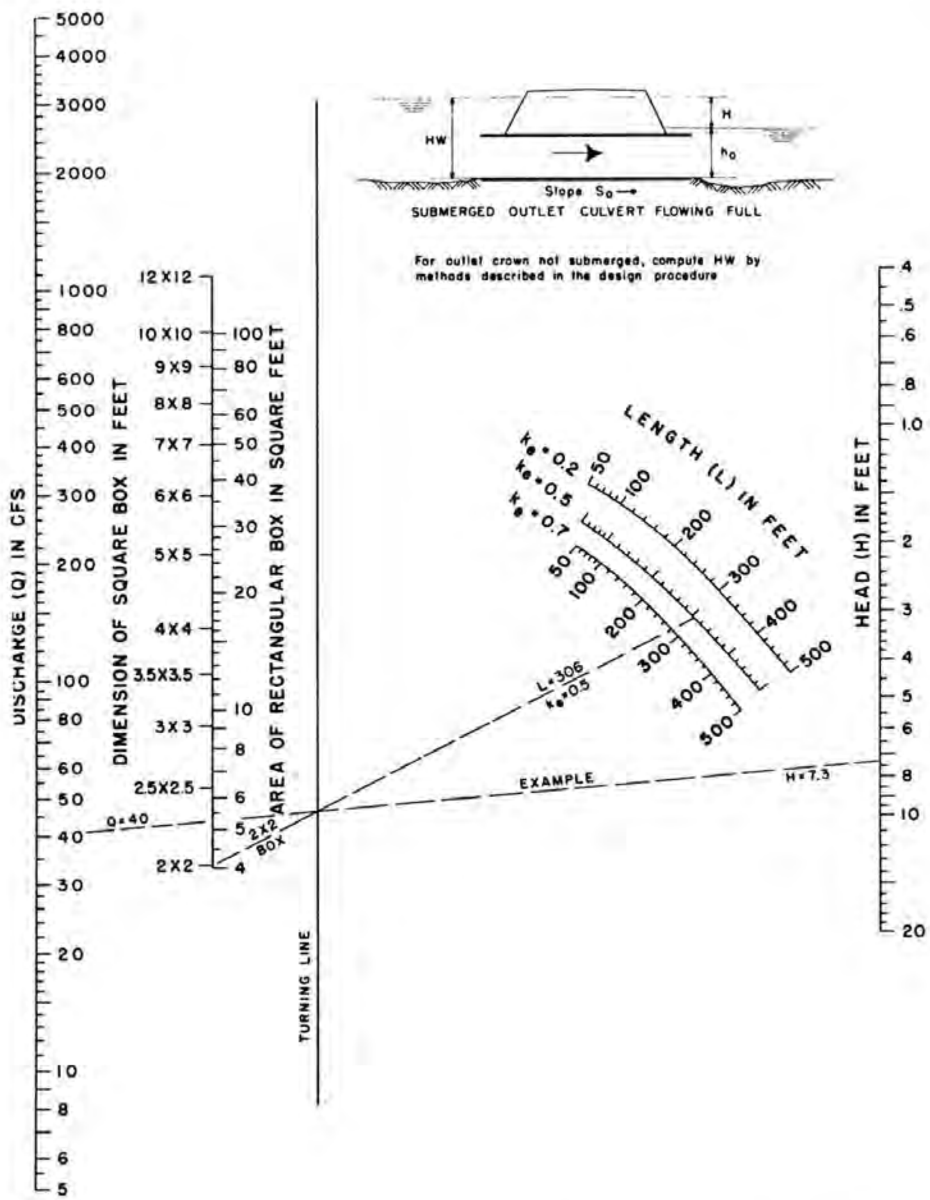
CHART 14



BUREAU OF PUBLIC ROADS JAN 1963

CRITICAL DEPTH
RECTANGULAR SECTION

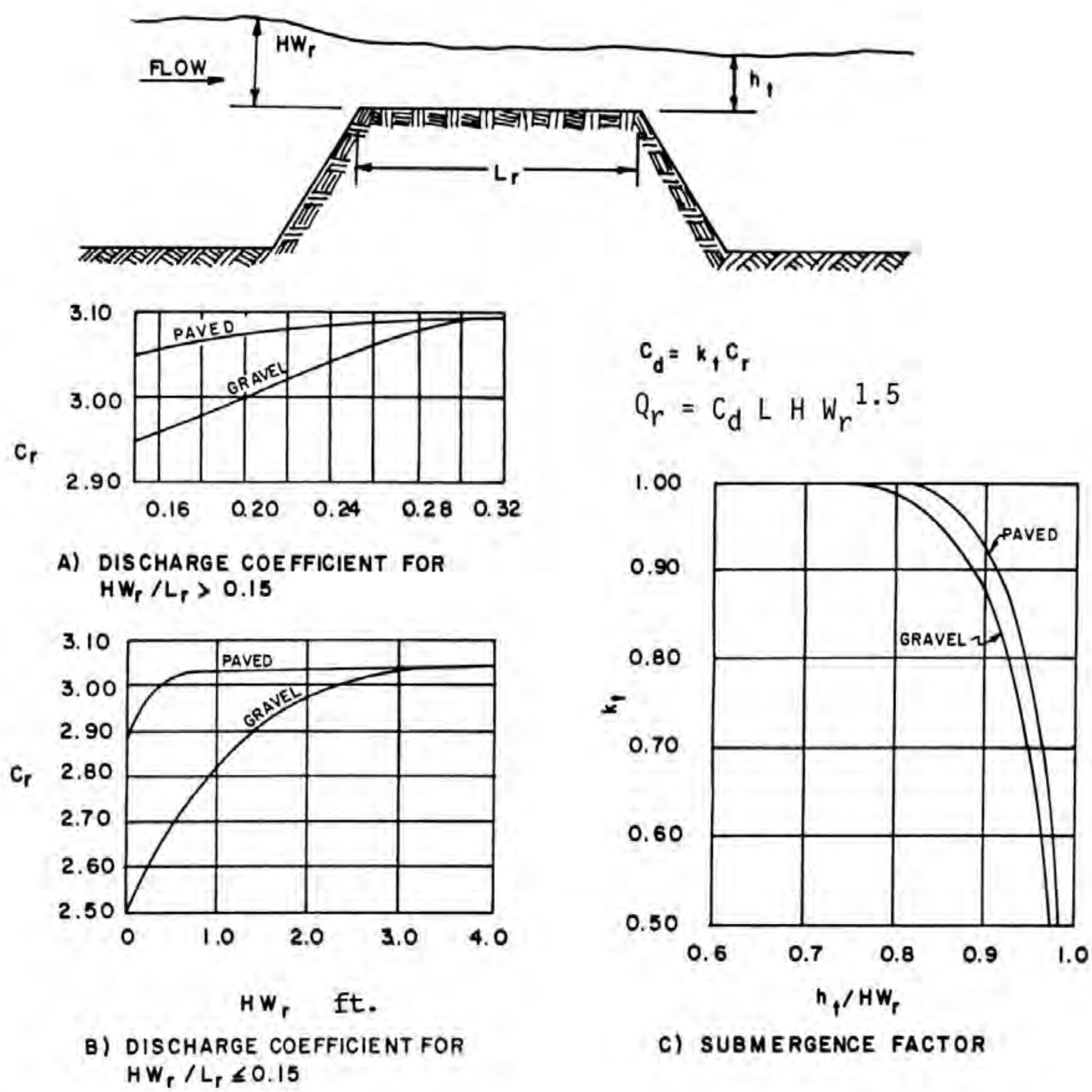
CHART 15



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
$n = 0.012$

AU OF PUBLIC ROADS JAN. 1963

CHART 16



DISCHARGE COEFFICIENTS
FOR ROADWAY OVERTOPPING

Appendix 8.B Tables and Forms

Table 1 RECOMMENDED MANNING'S n VALUES*

Type of Conduit	Wall Description	Manning's n
Concrete Pipe	Smooth walls	0.010-0.013
Concrete Boxes	Smooth walls	0.012-0.015
Corrugated Metal Pipes and Boxes, Annular or Helical Pipe (n varies barrel size) See HDS5	2 2/3 by 1/2 inch corrugations	0.022-0.027
	6 by 1 inch corrugations	0.022-0.025
	5 by 1 inch corrugations	0.025-0.026
	3 by 1 inch corrugations	0.027-0.028
	6 by 2 inch structural plate	0.033-0.035
	9 by 2 1/2 inch structural plate	0.033-0.037
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow	2 2/3 by 1/2 inch corrugations	0.012-0.024
Spiral Rib Metal	Smooth walls	0.012-0.013

*Note 1: The values indicated in this table are recommended Manning's "n" design values. Actual field values for older existing pipelines may vary depending on the effects of abrasion, corrosion, deflection and joint conditions. Concrete pipe with poor joints and deteriorated walls may have "n" values of 0.014 to 0.018. Corrugated metal pipe with joint and wall problems may also have higher "n" values, and in addition, may experience shape changes which could adversely effect the general hydraulic characteristics of the culvert.

Note 2: For further information concerning Manning n values for selected conduits consult Hydraulic Design of Highway Culverts, Federal Highway Administration, HDS No. 5, page 163.

TABLE 2 - ENTRANCE LOSS COEFFICIENTS
Outlet Control, Full or Partly Full

$$H_e = k_e \left[\frac{V^2}{2g} \right]$$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Socket end of pipe (groove-end)	0.2
Projecting from fill, socket end (groove-end)	0.2
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Mitered to conform to fill slope, paved or unpaved slope	0.7
Headwall or headwall and wingwalls square-edge	0.5
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Wingwalls at 10° to 25° or 30° to 75° to barrel	
Square-edged at crown	0.5
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Side-or slope-tapered inlet	0.2

*Note: "End Section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both <u>inlet</u> and <u>outlet</u> control. Some end sections, incorporating a <u>closed</u> taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

Project Name ____________________ Proj. No. ____________
Station/Location ____________________ Designer__________ Date__________
Subject ____________________ Checker__________ Date__________

Design Flows:

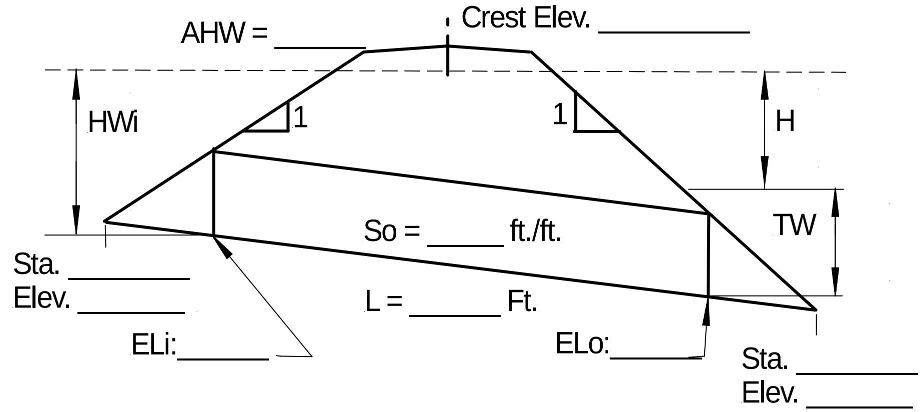
R.I (Years)	Flow (cfs)	T.W. (ft)

For Table: $EL_{hi} = EL_i + Hw_i$

h_o = greater of TW or $(d_c+D)/2$

H from chart 15

$El_{ho} = El_o + h_o + H$



Culvert Form

Culvert Description:																
Total Flow (cfs)	Flow/Ft: (cfs)	HEADWATER CALCULATIONS											Control HW:	Vo: Ft/sec	Comments
		Inlet Control: Chart ___				Outlet Control: Chart ___									
		Hw_i/D	Hw_i	Fall	EL_{hi}	TW	d_c	$(d_c+D)/2$:	h_o	k_e	H	EL_{ho}			

Appendix 8.C Tapered Inlets

8.C.1 General

A tapered inlet is a flared culvert inlet with an enlarged face section and a hydraulically efficient throat section. A tapered inlet may have a depression, or FALL, incorporated into the inlet structure or located upstream of the inlet. The depression is used to exert more head on the throat section for a given headwater elevation. Therefore, tapered inlets improve culvert performance by providing a more efficient control section (the throat). Tapered inlets with FALL also improve performance by increasing the head on the throat.

- Tapered inlets are recommended for use on culverts flowing in inlet control. This will maximize the benefit of opening the entrance to reduce the headwater.
- Tapered inlets are not recommended for use on culverts flowing in outlet control because the simple beveled edge is of equal benefit.
- Design criteria and methods have been developed for two basic tapered inlet designs: the side-tapered inlet and the slope-tapered inlet.
- Tapered inlet design charts are available for rectangular box culverts and circular pipe culverts.

8.C.2 Side-tapered

The side-tapered inlet has an enlarged face section with the transition to the culvert barrel accomplished by tapering the sidewalls (Figure 8.C.1). The face section is about the same height as the barrel height and the inlet floor is an extension of the barrel floor. The inlet roof may slope upward slightly, provided that the face height does not exceed the barrel height by more than 10 percent (1.1D). The intersection of the tapered sidewalls and the barrel is defined as the throat section.

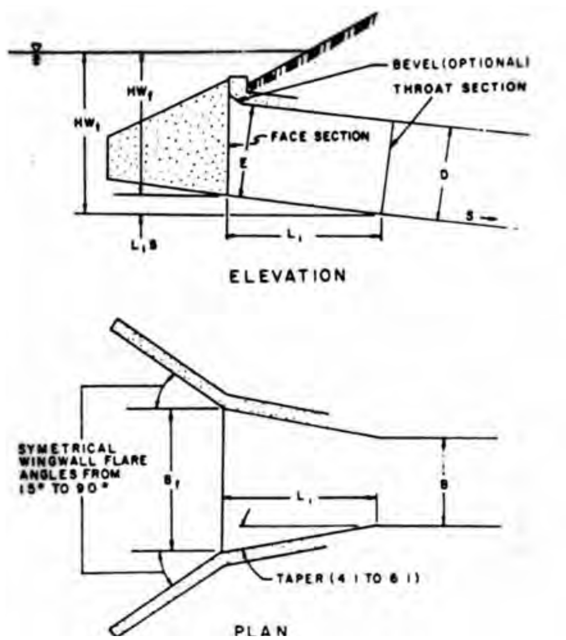


Figure 8.C.1 Side-Tapered Inlet

Appendix 8.C Tapered Inlets (continued)

8.C.2 Side-tapered (continued)

There are two possible control sections, the face and the throat. H_f, shown in Figure 8.A.2, is the headwater depth measured from the face section invert and H_t is the headwater depth measured from the throat section invert. The throat of a side-tapered inlet is a very efficient control section. The flow contraction is nearly eliminated at the throat. In addition, the throat is always slightly lower than the face so that more head is exerted on the throat for a given headwater elevation.

The beneficial effect of depressing the throat section below the streambed can be increased by installing a depression upstream of the side-tapered inlet. Figure 8.C.2 depicts a side-tapered inlet with the depression contained between wingwalls. For this type of depression, the floor of the barrel should extend upstream from the face a minimum distance of D/2 before sloping upward more steeply. The length of the resultant upstream crest where the slope of the depression meets the streambed should be checked to assure that the crest will not control the flow at the design flow and headwater. If the crest length is too short, the crest may act as a weir control section

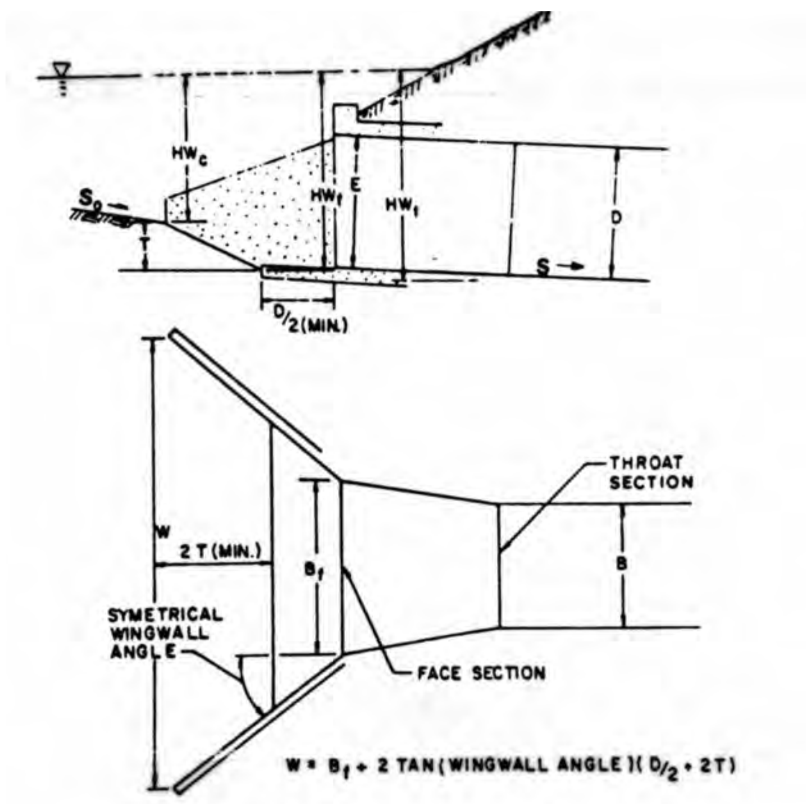


Figure 8.C.2 Side-tapered Inlet With Upstream Depression Contained Between Wingwalls

Appendix 8.C Tapered Inlets (continued)

8.C.3 Slope-tapered Inlet

The slope-tapered inlet, like the side-tapered inlet, has an enlarged face section with tapered sidewalls meeting the culvert barrel walls at the throat section. (Figure 8.C.3) In addition, a vertical FALL is incorporated into the inlet between the face and throat sections. This FALL concentrates more head on the throat section. At the location where the steeper slope of the inlet intersects the flatter slope of the barrel, a third section, designated the bend section, is formed.

A slope-tapered inlet has three possible control sections, the face, the bend, and the throat. Of these, only the dimensions of the face and the throat section are determined by the design procedures of this manual. The size of the bend section is established by locating it a minimum distance upstream from the throat so that it will not control the flow.

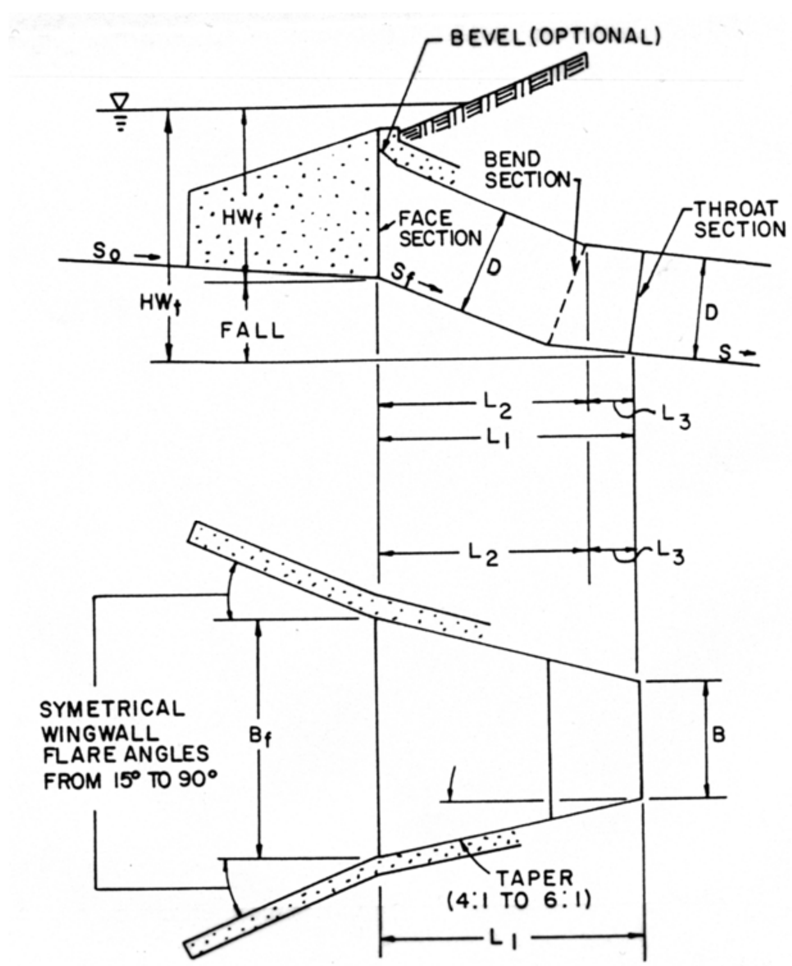


Figure 8.C.3 Slope-Tapered Inlet With Vertical Face

Appendix 8.C Tapered Inlets (continued)

8.C.3 Slope-tapered Inlet (continued)

The slope-tapered inlet combines an efficient throat section with additional head on the throat. The face section does not benefit from the FALL between the face and throat; therefore, the face sections of these inlets are larger than the face sections of equivalent depressed side-tapered inlets. The required face size can be reduced by the use of bevels or other favorable edge configurations. The vertical face slope-tapered inlet design is shown in Figure 8.C.3

The slope-tapered inlet is the most complex inlet improvement recommended in this manual. Construction difficulties are inherent, but the benefits in increased performance can be great. With proper design, a slope-tapered inlet passes more flow at a given headwater elevation than any other configuration. Slope-tapered inlets can be applied to both box culverts and circular pipe culverts. For the latter application, a square to round transition is normally used to connect the rectangular slope-tapered inlet to the circular pipe.

8.C.4 Hydraulic Design

Inlet Control

Tapered inlets have several possible control sections including the face, the bend (for slope-tapered inlets), and the throat. In addition, a depressed side-tapered inlet has a possible control section at the crest upstream of the depression. Each of these inlet control sections has an individual performance curve. The headwater depth for each control section is referenced to the invert of the section. One method of determining the overall inlet control performance curve is to calculate performance curves for each potential control section, and then select the segment of each curve which defines the minimum overall culvert performance. Figure 8.C.4

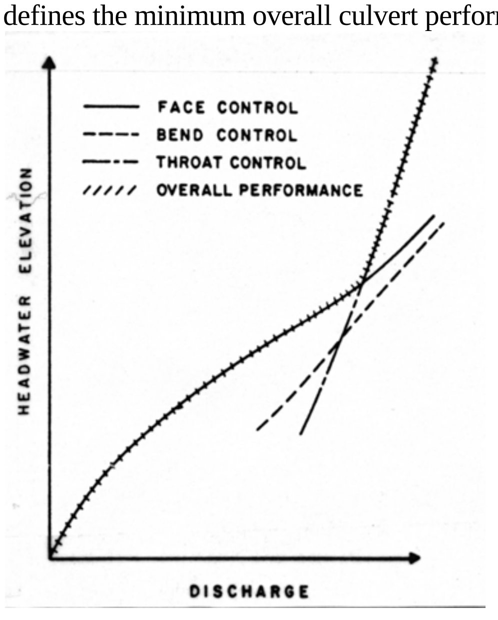


Figure 8.C.4 Inlet Control Performance Curve (Schematic)

Appendix 8.C Tapered Inlets (continued)

Inlet Control (continued)

Side-tapered Inlet

The side-tapered inlet throat should be designed to be the primary control section for the design range of flows and headwaters. Since the throat is only slightly lower than the face, it is likely that the face section will function as a weir or an orifice with downstream submergence within the design range. At lower flow rates and headwaters, the face will usually control the flow.

Slope-tapered Inlet

The slope-tapered inlet throat can be the primary control section with the face section submerged or unsubmerged. If the face is submerged, the face acts as an orifice with downstream submergence. If the face is unsubmerged, the face acts as a weir, with the flow plunging into the pool formed between the face and the throat. As previously noted, the bend section will not act as the control section if the dimensional criteria of this publication are followed. However, the bend will contribute to the inlet losses which are included in the inlet loss coefficient, K_E.

Outlet Control

When a culvert with a tapered inlet performs in outlet control, the hydraulics are the same as described in Section 8.5 for all culverts. The tapered inlet entrance loss coefficient (K_E) is 0.2 for both side-tapered and slope-tapered inlets. This loss coefficient includes contraction and expansion losses at the face, increased friction losses between the face and the throat, and the minor expansion and contraction losses at the throat.

8.C.5 Design Methods

Tapered inlet design begins with the selection of the culvert barrel size, shape, and material. These calculations are performed using the Culvert Design Form provided in Appendix B. The design nomographs contained in this Appendix are used to design the tapered inlet. The design procedure is similar to designing a culvert with other control sections (face and throat). The result will be one or more culvert designs, with and without tapered inlets, all of which meet the site design criteria. The designer must select the best design for the site under consideration.

In the design of tapered inlets, the goal is to maintain control at the efficient throat section in the design range of headwater and discharge. This is because the throat section has the same geometry as the barrel, and the barrel is the most costly part of the culvert. The inlet face is then sized large enough to pass the design flow without acting as a control section in the design discharge range. Some slight oversizing of the face is beneficial because the cost of constructing the tapered inlet is usually minor compared with the cost of the barrel.

Appendix 8.C Tapered Inlets (continued)

8.C.5 Design Methods (continued)

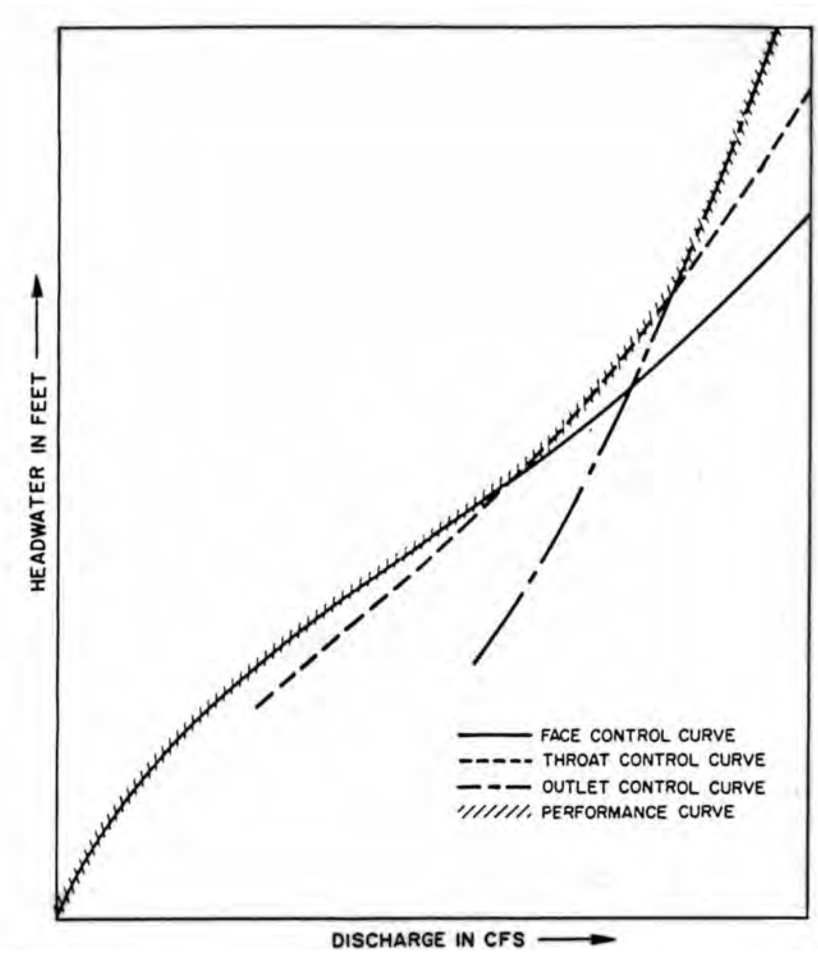
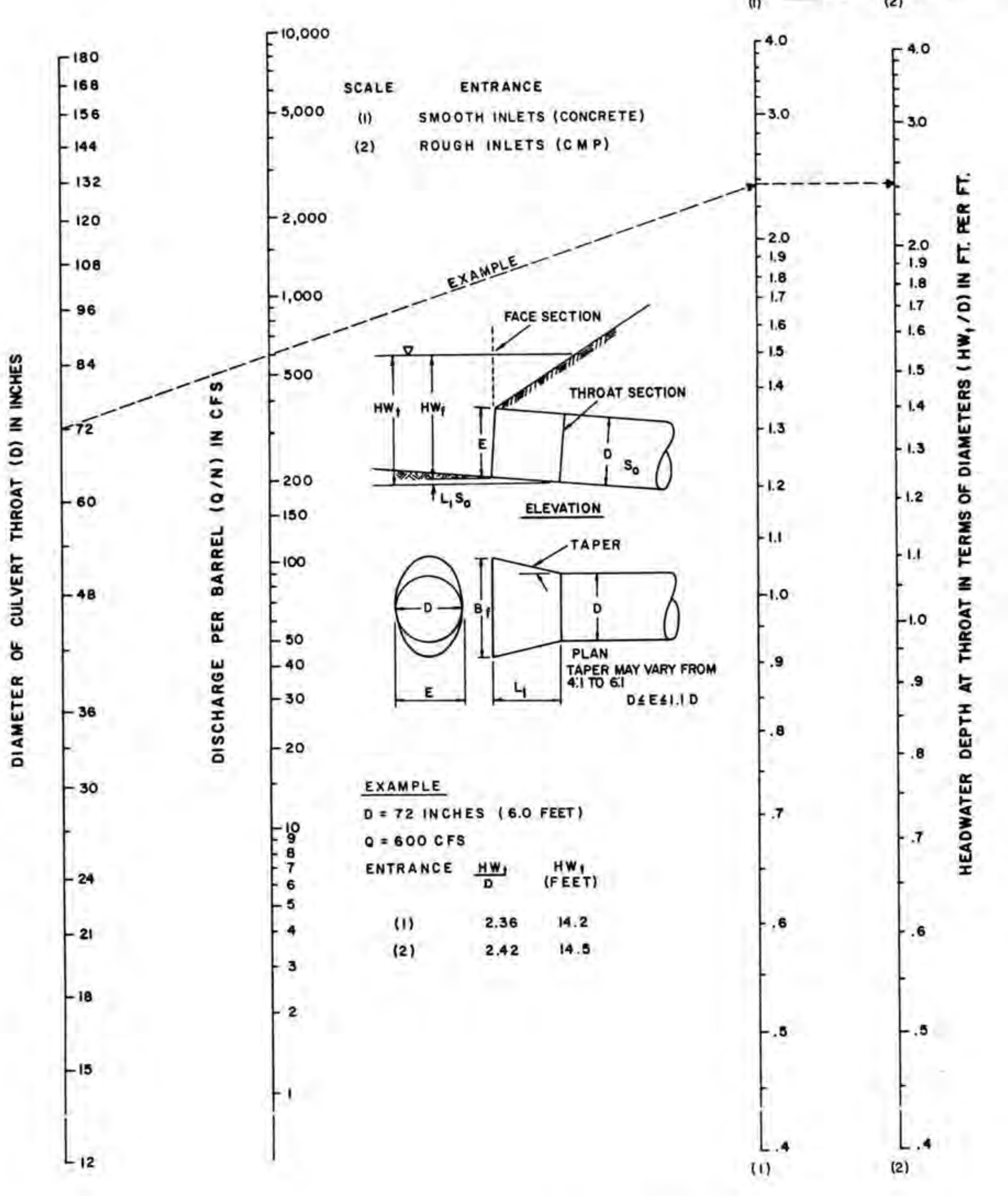


Figure 8.C.5- Culvert Performance Curve (Schematic)

Performance Curves

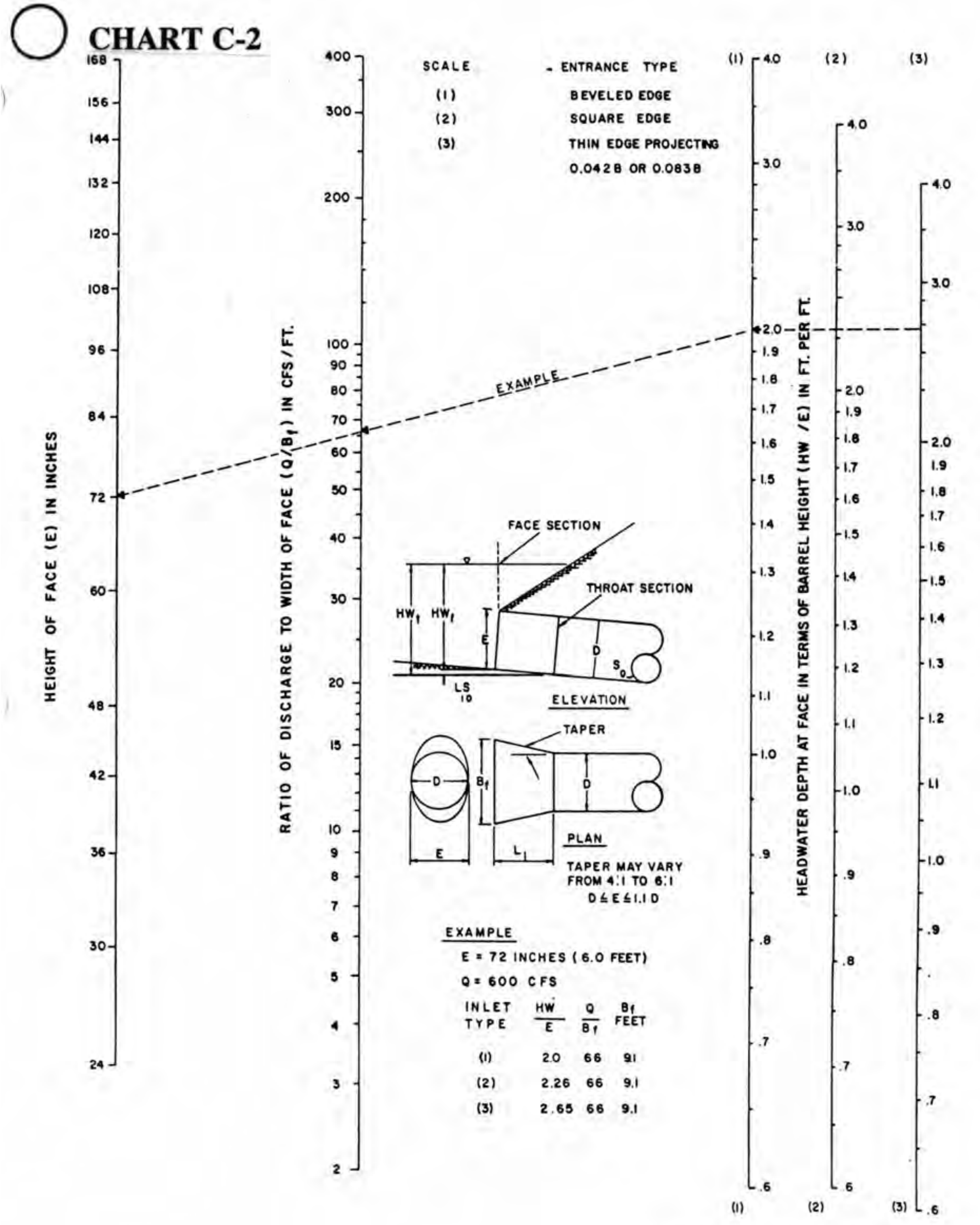
Performance curves are of utmost importance in understanding the operation of a culvert with a tapered inlet. Each potential control section (face, throat, and outlet) has a performance curve, based on the assumption that particular section controls the flow. Calculating and plotting the various performance curves results in a graph similar to Figure 8.C.5, containing the face control, throat control and outlet control curves. The overall culvert performance curve is represented by the hatched line. In the range of lower discharges face control governs; in the intermediate range, throat control governs; and in the higher discharge range, outlet control governs. The crest and bend performance curves are not calculated since they do not govern in the design range.

CHART C-1

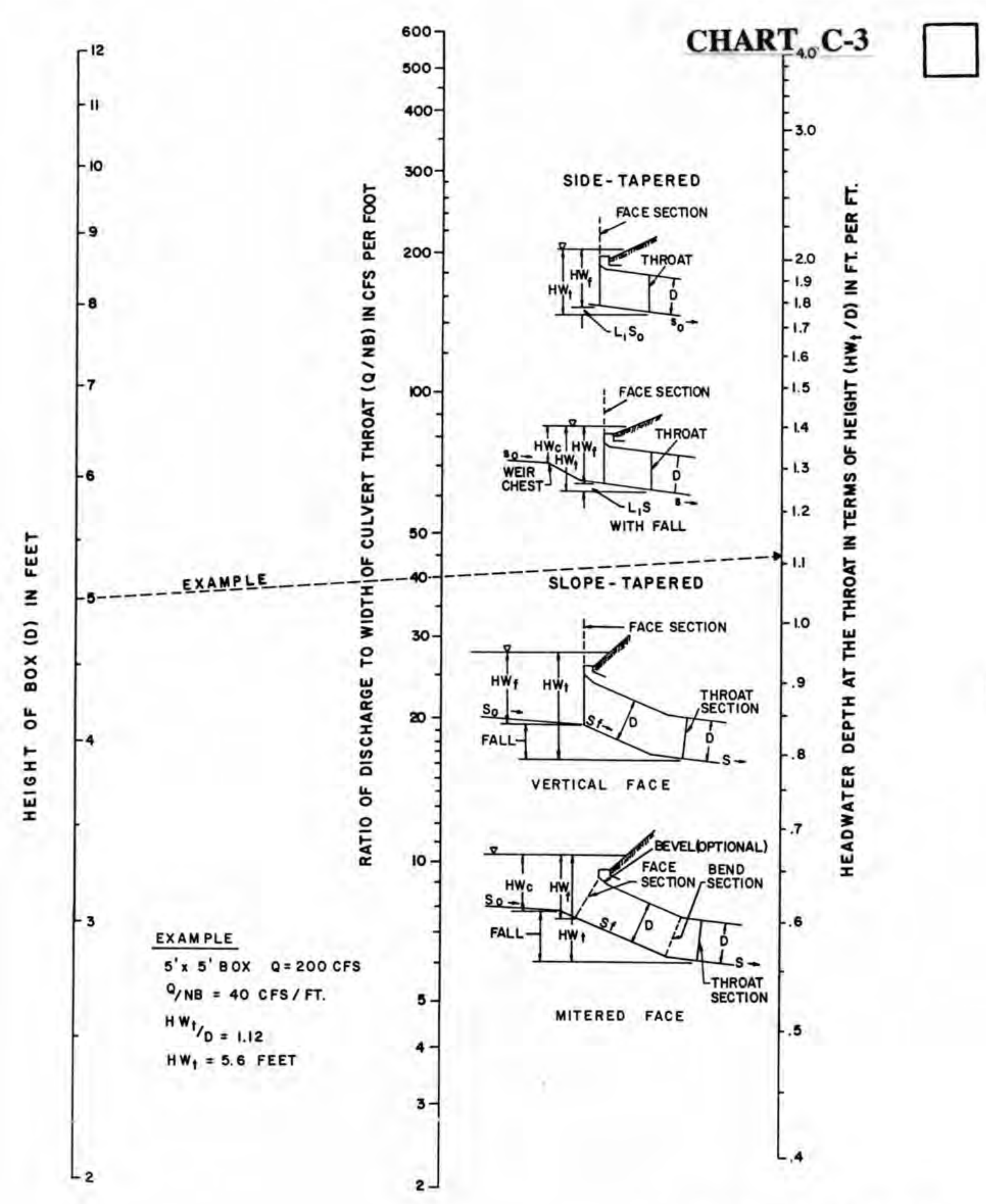


THROAT CONTROL
FOR SIDE-TAPERED INLETS TO PIPE CULVERT
(CIRCULAR SECTION ONLY)

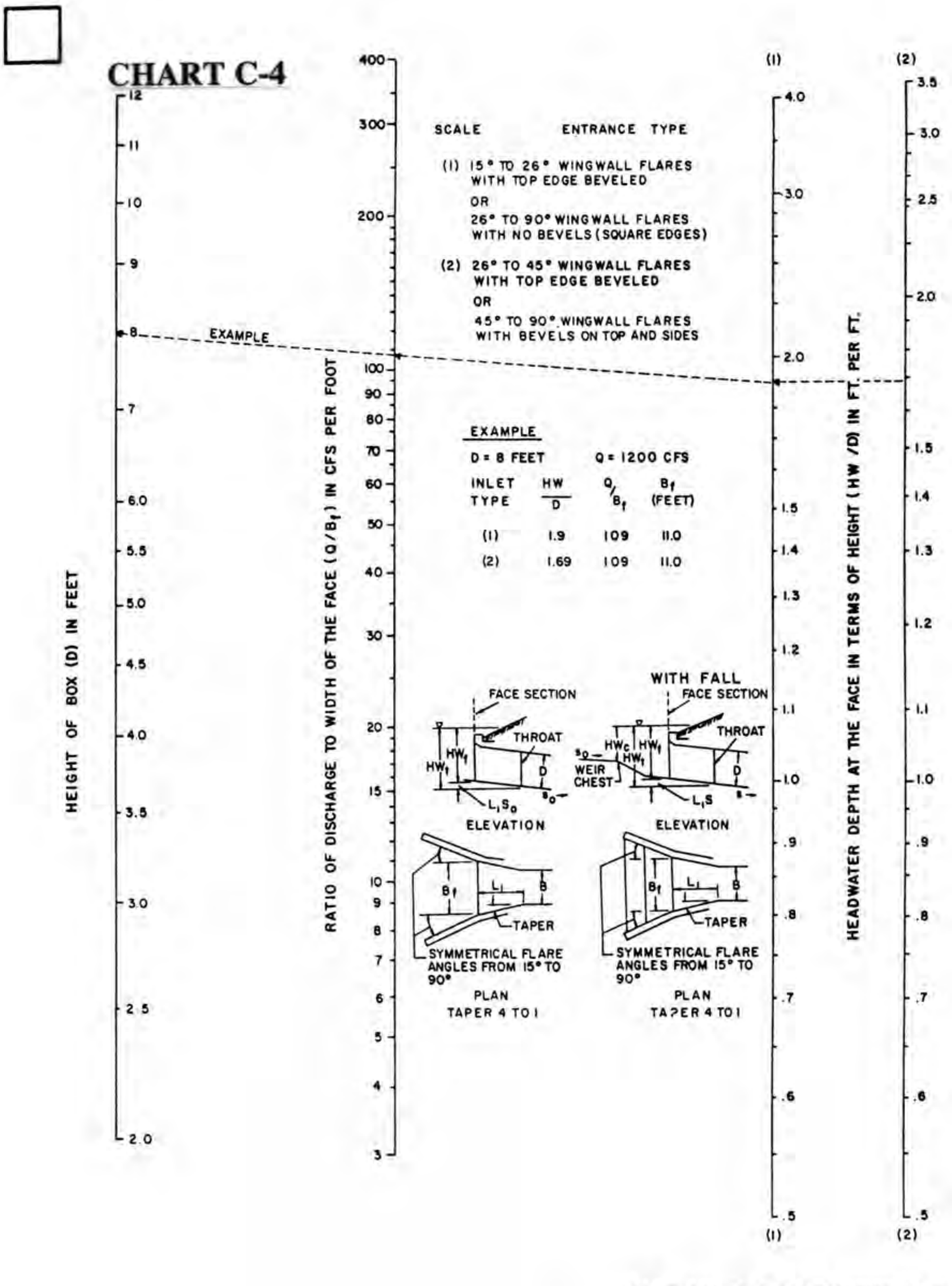
CHART C-2



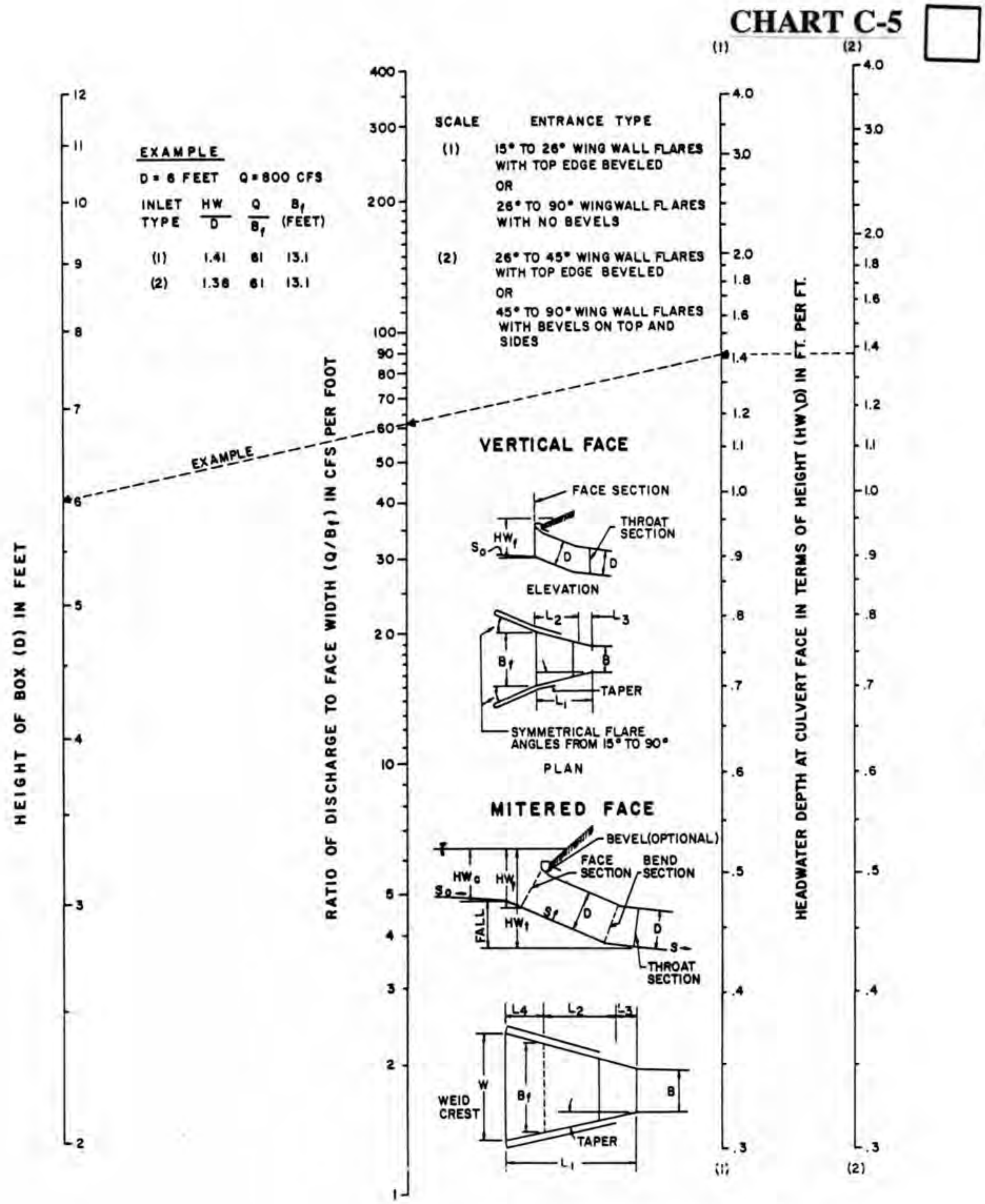
FACE CONTROL FOR SIDE-TAPERED INLETS TO PIPE CULVERTS (NON-RECTANGULAR SECTIONS ONLY)



THROAT CONTROL FOR BOX CULVERTS WITH TAPERED INLETS



FACE CONTROL FOR BOX CULVERTS WITH SIDE TAPERED INLETS



FACE CONTROL FOR BOX CULVERTS WITH SLOPE TAPERED INLETS

PROJECT : ______________________ ______________________	STATION : ______________ SHEET ______ OF ______	**TAPERED INLET DESIGN FORM** DESIGNER / DATE: ________ / ________ REVIEWER / DATE: ________ / ________

DESIGN DATA :

Q ______ = ______ cfs ; EL_{hi} ______________ ft

EL. THROAT INVERT ______________ ft

EL. STREAM BED AT FACE ______________ ft

FALL ________ ft TAPER ________ : 1 (4:1 TO 6:1)

STREAM SLOPE, So = ____________ ft/ft

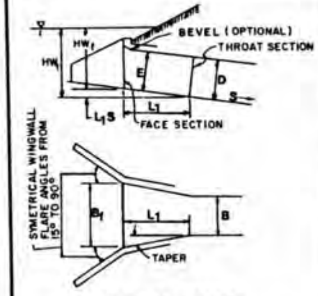
SLOPE OF BARREL, S = ____________ ft/ft

S_f ________ : 1 (2:1 TO 3:1)

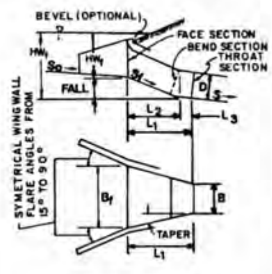
BARREL SHAPE AND MATERIAL : ______________________

N = ________, B = ____________, D = ____________

INLET EDGE DESCRIPTION ______________________



SIDE - TAPERED



SLOPE - TAPERED

COMMENTS

Q (cfs)	EL_{hi}	EL. THROAT INVERT	EL. FACE INVERT (1)	HW_f (2)	HW_f/E (3)	Q/B_f (4)	MIN. B_f (5)	SELECTED B_f	SLOPE-TAPERED ONLY: MIN. L_3 (6)	L_2 (7)	CHECK L_2 (8)	ADJ. L_3 (9)	ADJ. TAPER (10)	L_1 (11)	SIDE-TAPERED W/ FALL: EL. CREST INV.	HW_c (12)	MIN. W (13)

(1) SIDE - TAPERED : EL. FACE INVERT = EL. THROAT INVERT + 1 ft (APPROX.)
SLOPE - TAPERED : EL. FACE INVERT = EL. STREAM BED AT FACE

(2) $HW_f = EL_{hi}$ - EL. FACE INVERT

(3) $1.1\,D \geq E \geq D$

(4) FROM DESIGN CHARTS

(5) MIN. $B_f = Q/(Q/B_f)$

(6) MIN. $L_3 = 0.5\,NB$

(7) L_2 = (EL. FACE INVERT - EL. THROAT INVERT) S_f

(8) CHECK $L_2 = \left[\frac{B_f - NB}{2}\right] \cdot$ TAPER $- L_3$

(9) IF (8)>(7), ADJ. $L_3 = \left[\frac{B_f - NB}{2}\right] \cdot$ TAPER $- L_2$

(10) IF (7)>(8), ADJ. TAPER $= (L_2 + L_3) \Big/ \left[\frac{B_f - NB}{2}\right]$

(11) SIDE - TAPERED : $L = \left[\frac{B_f - NB}{2}\right] \cdot$ TAPER
SLOPE - TAPERED : $L_1 = L_2 + L_3$

(12) $HW_c = EL_{hi}$ - EL. CREST INVERT

(13) MIN. $W = 0.35\,Q / HW_c^{1.5}$

SELECTED DESIGN

B_f ____________

L_1 ____________

L_2 ____________

L_3 ____________

BEVELS ANGLE ____°

b = ______ in ; d = ______ in

TAPER ____________ : 1

S_f = ____________ : 1

PROJECT : ______________________________

____

STATION : ______________________________

SHEET ________ OF ________

MITERED INLET DESIGN FORM

DESIGNER / DATE : ____________ / ________

REVIEWER / DATE : ____________ / ________

DESIGN DATA : N ________ ; B ________ ; D ________

Q ________ = ________ cfs ; EL_{hi} ________ ft

EL. THROAT INVERT ________ ft EL. STREAM BED AT CREST ________ ft

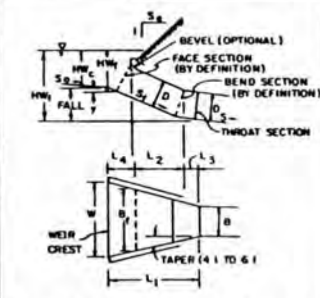
FALL ________ ft ; TAPER ________ : 1 (4:1 TO 6:1)

STREAM SLOPE , S_o ________ ft/ft ; BARREL SLOPE , S = ________ ft/ft

SLOPE OF THE EMBANKMENT S_e = ________ : 1 ; S_f ________ : 1 (2:1 TO 3:1)

BARREL SHAPE AND MATERIAL : ______________________________

INLET EDGE DESCRIPTION : ______________________________



SLOPE - TAPERED INLET/MITERED FACE

COMMENTS

Q (cfs)	EL_{hi}	EL. THROAT INVERT	y (1)	EL. FACE INVERT (2)	HW_f (3)	$\frac{HW_f}{E}$ (4)	$\frac{Q}{B_f}$ (5)	MIN. B_f (6)	SELECTED B_f	MIN. L_3 (7)	L_4 (8)	L_2 (9)	CHECK L_2 (10)	ADJ. L_3 (11)	ADJ. TAPER (12)	L_1 (13)	EL. CREST INV.	HW_c (14)	MIN. W (15)	W (16)

(1) $y = \left[\frac{(S_e \cdot S_f) - 1}{(S_e + S_f)(S_f^2 + 1)^{0.5}}\right] \cdot D$

(2) EL. FACE INVERT = EL. STREAM BED AT CREST - y

(3) $HW_f = EL_{hi}$ - EL. FACE INVERT

(4) $1.1D \geq E \geq D$

(5) FROM DESIGN CHARTS

(6) MIN. $B_f = Q/(Q/B_f)$

(7) MIN. $L_3 = 0.5$ NB

(8) $L_4 = S_f y + D/S_f$

(9) L_2 = (EL. CREST INVERT - EL. THROAT INVERT) $S_f - L_4$

*** IF L_2 IS NEGATIVE DO NOT USE THIS INLET

(10) CHECK $L_2 = \left[\frac{B_f - NB}{2}\right]$ TAPER $- L_3$

(11) IF (10)>(9), ADJ. $L_3 = \left[\frac{B_f - NB}{2}\right]$ TAPER $- L_2$

(12) IF (9)>(10), ADJ. TAPER $= (L_2 + L_3) / \left[\frac{B - NB}{2}\right]$

(13) $L_1 = L_2 + L_3 + L_4$

(14) $HW_c = EL_{hi}$ - EL. CREST INVERT

(15) MIN. $W = 0.35Q/(HW_c)^{1.5}$

(16) $W = NB + 2\left[\frac{L_1}{TAPER}\right]$

IF W<MIN. W , ADJUST TAPER

SELECTED DESIGN

B_f ________

L_1 ________

L_2 ________

L_3 ________

L_4 ________

BEVELS ANGLE ____°

b= ____ in; d= ____ in

TAPER ________ : 1

S_f ________ : 1

CHAPTER 9

ENERGY DISSIPATORS

Chapter 9 - Energy Dissipators
Table Of Contents

Chapter 9 - Energy Dissipators
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9.1 Introduction

9.1.1 Overview

The concentration of flow at culverts often results in increased velocities with a corresponding increase in erosion potential. If unchecked, the increased erosion potential can cause damage or failure of structures and the highway. To protect the culvert and adjacent areas, it is sometimes desirable to use an energy dissipator.

9.1.2 Definition

Energy dissipators are devices designed to reduce culvert outlet velocities to acceptable limits.

9.1.3 Purpose

This chapter provides:

- Criteria for selecting type of energy dissipator
- Design procedure for various types of energy dissipators including
  - weir-block energy dissipator
  - stilling basin
  - drop structures
  - impact basins, USBR Type VI

- Design procedures for selected types of dissipators that are based on FHWA Hydraulic Engineering Circular Number 14 (HEC 14) "Hydraulic Design of Energy Dissipators for Culverts and Channels," September 1983, revised in 1995.
- Results of analysis using the HY8 software for stilling basins, drop structures, and USBR Type VI impact basins.

9.1 Introduction (continued)

9.1.4 Symbols

Table 9-1 Symbols, Definitions And Units

Symbol	Definition	Units
A	Cross sectional area	ft^2
A_o	Area of flow at culvert outlet	ft^2
d_E	Equivalent depth at brink	ft.
d_o	Normal flow depth at brink	ft.
D	Height of culvert	ft.
D_{50}	Mean diameter of riprap	in.
DI	Discharge Intensity Modified	-
Fr	Froude Number	-
H	Height of weir block	ft.
h_S	Depth of dissipator pool	ft.
L	Length of culvert	ft.
L_B	Overall length of basin	ft.
L_S	Length of dissipator pool	ft.
Q	Rate of discharge	ft^3/sec
S_o	Slope of streambed	ft./ft.
TW	Tailwater depth	ft.
V_d	Velocity downstream	ft./sec
V_L	Velocity -- (L) feet from brink	ft./sec
V_o	Normal velocity at brink	ft./sec
W_o	Diameter or width of culvert	ft.
W_S	Width of scour hole	ft.
Yo	Depth of flow at brink	ft.
Y_2	Sequent Depth (Alternate Depth)	ft.

9.2 Design Goals

9.2.1 Overview

The goal of using energy dissipators is to manage the effects of high velocity by providing a structure that will reduce the outlet velocity of the design flow to an acceptable value. The desired level is within 120% of the average main channel velocity. The reduction of velocity may be accomplished be a change in direction (drop structure), an increase in the flow resistance (rip-rap mattress), or by forcing a change in the flow from supercritical to subcritical (weir block). Changing flow conditions from supercritical to subcritical will require the formation of a hydraulic jump.

9.2.2 Selection Criteria

The dissipator type selected for a site must be appropriate to the location. Natural scour holes may need to be estimated to determine the value of providing additional protective measures. The selection of the means to accomplish a change in flow conditions is affected by the flow conditions in the downstream channel. Discussed in this chapter are natural scour holes, hydraulic jumps using weirs or stilling basins, and USBR Type VI Impact Basins.

The hydraulic jump is a means to accomplish a reduction in velocity. If the downstream flow results in low tail water, some artificial means is used to create the additional tailwater necessary to force a hydraulic jump. For low Froude numbers, i.e. less than 3, this may be accomplished by using a “dam” or weir or dropping the outlet apron. For culverts 48” or larger, with flow less than 150 cfs, a weir block and apron at the end of the wing wall may achieve the desired results. For situations where the approaching subcritical flow must be dropped some distance, a straight drop stilling basin may be the appropriate solution. For supercritical flows, a sloping drop stilling basin may be necessary. Other situations may require the use of baffles.

Table 9-2 Dissipator Selection Criteria

Approach Flow	Downstream Flow	Dissipator	Debris Restrictions[1] Floating	Debris Restrictions[1] Boulders	Tailwater Required.
Subcritical	Subcritical	Straight Drop	M	L	Yes
Supercritical, Fr=1 to 4	Supercritical	Weir Block	M	L	No
Fr=1 to 4	Subcritical	Abrupt Rise	M	L	Yes
Fr=1.7 to 17	Subcritical	SAF stilling basin	M	L	Yes
Fr=2.5 to 4.5	Subcritical	USBR Type VI	M	L	No

NOTE 1: Debris Restrictions: M-Medium, L-Large

9.2 Design Goals (continued)

9.2.3 Design Considerations

Debris

The occurrence of debris shall be considered in the design of energy dissipators. Debris control shall be designed using Hydraulic Engineering Circular No. 9, "Debris-Control Structures" and shall be considered:

- where potential for large debris exists, and
- where clean out access is limited.

Design Flood Frequency

The flood frequency used in the design of the energy dissipator device is usually the same flood frequency used for the culvert or channel design. The use of a greater frequency may be appropriate if justified by documented special site conditions, such as downstream concerns.

Maximum Dissipator Exit Velocity

The dissipator exit velocity should be within 120% of the average main channel velocity.

Tailwater Relationship

The hydraulic conditions downstream shall be evaluated to determine a tailwater depth and the maximum velocity for the range of discharges to be considered.

Large water bodies shall be evaluated using the water elevation that has the same frequency as the design flood for the culvert and/or channel if events are known to occur concurrently, statistically dependent. If statistically independent, evaluate the joint probability of flood magnitudes and use a likely combination.

Safety Considerations

Traffic shall be protected from external energy dissipators by locating them outside the appropriate "clear zone" distance per AASHTO Roadside Design Guide or shielding them with a traffic barrier.

9.2 Design Goals (continued)

9.2.4 Design Options

Weep Holes

If weep holes are used to relieve uplift pressure, they shall be designed in a manner similar to underdrain systems.

Culvert Outlet Type

In choosing a dissipator, the selected culvert end treatment has the following implications:
- Culvert ends which are projecting or mitered to the fill slope offer no outlet protection.
- Aprons do not reduce outlet velocity. They shall not protrude above the normal streambed elevation.

9.2.5 Related Designs

Culvert

The culvert shall be designed considering the site requirements. If it is determined that energy dissipation is needed, the design shall be reviewed to ascertain any impacts that can be mitigated by changes in the culvert design (Chapter 8 Culverts). The culvert design shall include computation of outlet velocity before the final outlet protection is designed.

Downstream Channel

The downstream channel protection shall be designed concurrently with dissipator design (Chapter 7 Channels).

9.3 Design Philosophy

9.3.1 Overview

The energy dissipator design approach used in this chapter is discussed in the following sections:

9.3.2 Alternative Analysis

Choose alternatives, which satisfy:

- Topography, and
- design policies and criteria.

Analyze alternatives for:

- environmental impact, such as need for animal passage
- hydraulic efficiency, and
- risk and cost.

Select an alternative that best integrates engineering, economic and environmental considerations:

The chosen dissipator should meet the selected structural and hydraulic criteria and should be based on:
- construction and maintenance costs,
- risk of failure,
- property damage,
- traffic safety,
- environmental and aesthetic considerations, and
- land use requirements.

9.3.3 Design Methods

The designer has to determine:

- whether to mitigate or monitor erosion problem,
- design conditions of local scour and/or channel degradation, and
- the type of energy dissipator to be used

.

9.3 Design Philosophy (continued)

9.3.3 Design Methods (continued)

9.3.3.1 Types Of Scour

Local Scour

Local scour is the result of high velocity flow at the culvert outlet and extends only a limited distance downstream.

Channel Degradation

Channel degradation may proceed in a fairly uniform manner over a long length or may be evident in one or more abrupt drops (headcuts) progressing upstream with every runoff event.
- It should be investigated as an essential part of the site investigation.
- It should be mitigated and addressed in the initial construction (see Channels, Chapter 7).
- It is usually mitigated with drop structures.

9.3.3.2 Dissipator Types

The dissipator types discussed include:
- Weir block hydraulic jump (Section 9.6).
- USBR Type VI impact basin (Section 9.7).
- Saint Anthony Falls stilling basin, SAF (Section 9.8).
- Straight Drop Stilling Basin (Section 9.9)

Other dissipator types are discussed in the FHWA HEC 14 "Hydraulic Design of Energy Dissipators for Culverts and Channels," September 1985.
- Riprap.
- CSU rigid boundary.
- Contra Costa.
- Hook.
- USBR Type II.
- USBR Type III
- USBR Type IV.

9.3 Design Philosophy (continued)

9.3.3 Design Methods (continued)

9.3.3.3 Computational Methods

Charts

Charts required for the design of USBR Type VI impact basin, SAF stilling basin, and Straight Drop stilling basins are included in this Chapter. Charts required for the design of other types of energy dissipators are found in HEC 14.

Computer Software

HY-8 (FHWA Culvert Analysis Software) Version 4.1 or greater, contains an energy dissipator module which can be used to analyze most types of energy dissipators in HEC 14.

9.4 Design Equations

9.4.1 General

An exact theoretical analysis of flow at energy dissipators is extremely complex because the following approaches are required:

- analyzing non-uniform and rapidly varying flow,
- applying energy and momentum balance,
- determining where a hydraulic jump will occur, and
- applying the results of hydraulic model studies.

9.4.2 Design Approach

The design procedures presented in this Chapter are based on the following:

- Discussions regarding hydraulic jumps (Chow)
- Model studies were used to calibrate the equations and charts for scour hole estimating and energy dissipator design.
- HEC 14 (revised version, 1995) is the base reference and contains explanation of the equations and procedures used in this Chapter for stilling basins, drop structures, and riprap basins.

9.4 Design Equations (continued)

9.4.3 Flow Conditions

The approach flow condition establishes the hydraulic parameters for design of energy dissipators.

Depth (ft.), $\mathbf{d_o}$.

- Is the depth at the beginning of the energy dissipator
- The normal depth assumption should be reviewed and a water surface profile calculated if L < 50 d_o.
- The brink depth and not critical depth (see HEC 14 for curves) should be used for mild slopes and low tailwater.

Equivalent Depth (ft), $\mathbf{d_E}$

$$d_E = (A_o/2)^{0.5}$$

Equivalent depth is an artificial depth that is calculated for culverts that are not rectangular so that a reasonable Fr can be determined.

Area (ft^2), $\mathbf{A_o}$.

The cross-sectional area of flow at the approach section should be calculated using (d_o).

Velocity (ft/sec), $\mathbf{V_o}$

The velocity at the approach to the dissipator should be calculated as follows:

$$\mathbf{V_o = Q/A_o} \qquad \textbf{(9.1)}$$

Where: Q= discharge, cfs

Froude Number, **Fr**

The Froude number is a flow parameter that has traditionally been used to design energy dissipators and is calculated using:

$$\mathbf{Fr = V_o/[(g\ d_o)^{0.5}]} \qquad \textbf{(9.2)}$$

Where: g = acceleration of gravity, 32.2 ft/sec^2

9.4 Design Equations (continued)

9.4.3 Flow Conditions (continued)

Sequent Depth, (Alternate depth), ft

The sequent depth (alternate depth) is the depth to which supercritical flow changes in a hydraulic jump. It has the same energy at the subcritical flow energy condition.

$$Y_1+V_1^2/2g=Y_2+V_2^2/2g$$

$$\mathbf{Y_2 = (Y_1/2)*[(1+8Fr^2)^{0.5}-1]} \qquad \textbf{(9.3)}$$

Where: $\mathbf{Y_1}$ = initial depth of water, ft.
$\mathbf{Y_2}$= sequent depth of jump, ft.
$\mathbf{Fr_1}$= Froude number of flow entering basin based on d_1

Discharge Intensity, $\mathbf{DI_c}$.

Discharge Intensity is a flow parameter similar to Fr that is used for circular culverts of diameter (D) that are flowing full.

$$\mathbf{DI_c = Q/(g^{0.5}D^{2.5})} \qquad \textbf{(9.4)}$$

Discharge Intensity Modified, **DI**.

Referring to Chapter V, HEC 14, revised version 1995, the Modified Discharge Intensity, DI, for all culvert shapes are:

$$\mathbf{DI = Q/(g^{0.5}\, R_c^{\ 2.5})} \qquad \textbf{(9.5)}$$

Where: Q = discharge, cfs
A_c= culvert area, ft^2
P_c= culvert perimeter, ft
R_c= (A_c/P_c)

See Appendix for table of diameter, area, perimeter, and hydraulic radius.

9.5 Scour Hole Estimation

9.5.1 Overview

Scour holes may form at the outlets of culverts or at channel drops. Chapter V of HEC 14 (revised version, 1995) contains a procedure for estimating scour hole geometry for culvert outlets based on soil, flow data and culvert geometry. The US Bureau of Reclamation has procedures for estimating the scour below channel drops.

9.5.2 Outlet Scour Prediction

It is intended that this scour prediction procedure be used along with the maintenance history and site reconnaissance information for determining energy dissipator needs. Only scour hole in cohesionless material will be discussed in this Chapter. For scour hole in cohesive soil, the designer should refer to Chapter V, HEC 14. The results of the tests made by the US Army Waterways Experiment Station, Vicksburg, Mississippi indicate that the scour hole geometry varies with the tailwater conditions. The maximum scour geometry occurs at tailwater depths less than half the culvert height. The maximum depth of scour, d_S, occurs at a location approximately $0.4L_S$ downstream of the culvert, where L_S is the length of the scour.

9.5.2.1 Culvert Outlet Scour - Equations

The following empirical equations from the reference **Scour at Culvert Outlets in Mixed Bed Materials (**Ruff, J.F., S.R. Abt, C. Mendosa, A. Shaikh, and R. Kloberdanz.). These equations define the relationship between the culvert discharge intensity, time and the length, width, depth and volume of the scour hole for the maximum or extreme scour case.

$$(ds/Rc),(Ws/Rc),(Ls/Rc)=C_sC_h[\alpha/\sigma^{0.33}]\,[DI]^{\beta}[t/316]^{\theta} \tag{9.6}$$

$$d_S,\ W_S,\ \text{or}\ L_S = F_1F_2F_3R_c \tag{9.7}$$

$$F_1 = C_s C_h\left(\frac{\alpha}{\sigma^{1/3}}\right)$$

$$F_2 = \left(\frac{Q}{\sqrt{g}\,R_c^{2.5}}\right)^{\beta} = (DI)^{\beta}$$

$$F_3 = \left(\frac{t}{316}\right)^{\theta}$$

9.5 Scour Hole Estimation (continued)

9.5.2 Outlet Scour Prediction (continued)

Where: d_S = maximum depth of scour hole, ft
L_S = length of scour hole, ft
W_S = width of scour hole, ft
t = 30 min or the time of concentration, if longer
R_c= hydraulic radius of the flow at the exit of the culvert
σ = material standard deviation, generally, σ = 2.10 for gravel and 1.87 for sand
a, b, Q, C_S and C_h are coefficients, as shown in Table 9-2
F_1 , F_2 and F_3 are factors to aid the computation

Table 9-3 Scour Hole Coefficients

A. Coefficient for Culvert Outlet Scour - Cohesionless Materials

	a	**b**	**Q**
Depth, d_S	7.96	0.26	0.09
Width, W_S	26.42	0.62	0.06
Length, L_S	64.54	0.56	0.17
Volume, V_S			

B. Coefficient C_S for Outlets Above the Bed

H_S	Depth	Width	Length	Volume
0	1.00	1.00	1.00	1.00
3.28	1.22	1.51	0.73	1.28
6.56	1.26	1.54	0.73	1.47
13.12	1.34	1.66	0.73	1.55
H_S is the height above bed in pipe diameters, ft				

9.5 Scour Hole Estimation (continued)

9.5.2.1 Culvert Outlet Scour – Equations (continued)

C. Coefficient C_h for Culvert Slope

Slope %	Depth	Width	Length	Volume
0	1.00	1.00	1.00	1.00
2	1.03	1.28	1.17	1.30
5	1.08	1.28	1.17	1.30
>7	1.12	1.28	1.17	1.30

9.5.2.2 Culvert Outlet Scour – Design Procedure

The following design procedures are intended to provide a convenient and organized method for designing energy dissipators. The designer should be familiar with all the equations in section 9.4 before using these procedures. In addition, application of the following design method without an understanding of hydraulics can result in an inadequate, unsafe, or costly structure.

Step 1 Assemble Site Data And Project File

a. See design file for site survey.

Step 2 Determine Hydrology, Select Design Q

See design file.
b. Select flood frequency.
c. Determine Q.

Step 3 Design Downstream Channel

a. Determine channel slope, cross section, normal depth and velocity.
b. Check bed and bank materials stability.

Step 4 Design Culvert

See design file and obtain: design discharge, outlet flow conditions (velocity and depth), culvert type (size, shape and roughness), culvert slope and performance curve, if necessary.

Step 5 Summarize Data On Design Form

a. Enter data from steps 1-4 into Figure 9-1: Scour Hole Estimation Form.

Step 6 Estimate Scour Hole Size

a. Enter input for scour equation on Figure 9-1.
b. Calculate d_S, W_S, L_S, using equations 9.6 or 9.7

9.5 Scour Hole Estimation (continued)

9.5.2.2 Culvert Outlet Scour – Design Procedure (continued)

Step 7 Determine Need For Dissipator

An energy dissipator is needed if:
a. the estimated scour hole dimensions exceed the allowable right-of-way, undermines the culvert cutoff wall, or presents a safety or aesthetic problem;
b. downstream property is threatened; or
c. V_o is greater than 150 %V_d.

Step 8 Select Design Alternative

a. Calculate Froude number, Fr.
b. Choose energy dissipator types.
- •If Fr < 3, design a weir block hydraulic jump basin, a riprap basin or a USBR Type VI. These types are recommended only if Q < 500 ft^3/s for each barrel and little debris is expected. If these are not acceptable or economical, try other dissipators in HEC 14.
- •If Fr > 3, design a SAF stilling basin.

Step 9 Design Dissipators

Use the following design procedures and charts:
•Section 9.6 for the weir block hydraulic jump basin.
•Section 9.7 for the USBR Type VI impact basin.
•Section 9.8 for the SAF basin.

Step 10 Design Riprap Transition

a. Most dissipators require some apron adjacent to the basin exit.
b. The length of apron protection can be judged based on the difference between V_o and V_d. See HEC 14 for length required for size and length of protection needed.

Step 11 Review Results

a. If preferred energy dissipator affects culvert hydraulics, return to step 4 and calculate culvert performance.
b. If debris-control structures are required upstream, consult HEC 9.

Step 12 Documentation

a. See Documentation Chapter.

9.5 Scour Hole Estimation (continued)

9.5.2 Outlet Scour Prediction (continued)

Scour Hole Estimation Form
Project Name____ Project No.__________
Subject ____Page ___ of _____
By ___________________________ Date ______ Checked By ____________________ Date _____

Data Summary		
Parameter	Approach Channel	Downstream Channel
Station		
Control		
Type		
Height, D		
Width, B		
Length, L		
Material		
Manning's n		
Side Slope		
Slope		
Discharge		
Depth, d		
Velocity, V		
Froude No.		
Flow Area		

Equation Input Data	
Factor	Value
Discharge, Q cfs	
Flow Area, A, Ft^2	
Wetted Perimeter, Ft	
Hydraulic Radius, R=A/P	
Discharge Intensity	
Time of Concentration, t, min	

Scour Equations			
$(d_s/R_c),(W_s/R_c),(L_s/R_c)=C_sC_h[\mathbf{a}/\mathbf{s}^{0.33}]\ [DI]^{\beta}[t/316]^{\theta} = F_1F_2F_3R_c$			
Scour Computation			
Factor	Depth	Width	Length
Alpha, **a**			
Beta, **b**			
Theta, **Q**			
$F_1 = C_sC_h[\mathbf{a}/\mathbf{s}^{0.33}]$			
$F_2 = [DI]^{\mathbf{b}}$			
$F_3 = [t/316]^{\mathbf{Q}}$			
Scour			

Figure 9-1 Scour Estimation Form

9.5 Scour Hole Estimation (continued)

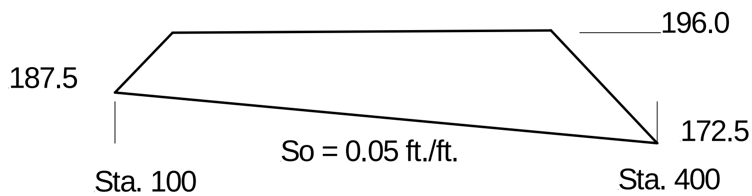
9.5.2 Outlet Scour Prediction (continued)

9.5.2.3 Design Example

The following example uses a 7x6 culvert. This is not a standard ADOT culvert. Its use here is not to be understood as endorsement of using this size of culvert.

Step 1 Assemble Site Data And Project File

a. Site survey - The culvert project file contains site and location maps; roadway profile and embankment cross sections. Site visit notes indicate no sediment or debris problems and no nearby structures.



Culvert 7x6

b. Studies by other agencies - none.
c. Environmental, risk assessment shows no problems.
d. Design criteria:

- 50-year frequency for design

Step 2 Determine Hydrology

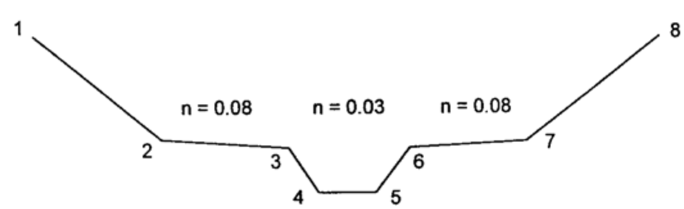
From project file:

- Q_{50} = 400 cfs
- Q_{100} = 500 cfs

Select Design Q: Use Q_{50} = 400 cfs

Step 3 Design Downstream Channel

a. Cross section of channel with slope = 0.05 ft./ft.



9.5 Scour Hole Estimation (continued)

9.5.2.3 Design Example (continued)

Step 3 Design Downstream Channel (continued)

Point	Station, ft	Elevation, ft
1	12	180
2	22	175
3	32	174.5
4	34	172.5
5	39	172.5
6	41	174.5
7	51	175
8	61	180

b. Rating Curve for Channel
Calculating normal depth yields:

Q (cfs)	TW (ft)	V (ft/sec)
100	1.4	11
200	2.1	14
300	2.5	16
400	2.8	18
500	3.1	19

c. For Q_{50} = 400 cfs, with V_{50} = 18 ft/sec the 3-in gravel material which makes up the channel boundary is not stable and riprap is needed (see Channel, Chapter 7) for a transition.

Step 4 Design Culvert

A 7x6 RCB with a beveled entrance on a slope of 0.05 ft/ft was the selected design. The FHWA HY8 program showed that this culvert is operating at inlet control and has the following hydraulic behavior:

	Q (cfs)	HW_i (ft)	V_o (ft/sec)
Q_{50} =	400	7.6	32
Q_{ot} =	430	8.5	
Q_{100} =	500	8.6	34

Step 5 Summarize Data On Design Form

See Figure 9-2.

Step 6 Determine Size of Scour Hole

The size of the scour hole is determined using equations 9.5 and 9.6. For channel with gravel bed, the standard deviation of the material, σ is 2.10. Table 9-2 shows that the value of C_S =1.00 and C_h = 1.08. See Figure 9-2 for a summary of the computation.

9.5 Scour Hole Estimation (continued)

9.5.2.3 Design Example (continued)

Step 7 Determine Need For Dissipator

The scour hole dimensions are excessive, and since V_o = 32 ft/sec is much greater than V_d = 18 ft/sec, an energy dissipator is needed.

Step 8 Review Results

The downstream channel conditions are matched by the dissipator.

Step 9 Documentation

a. See Documentation Chapter 5.
b. Include computations in the culvert report or file.

9.5 Scour Hole Estimation (continued)

9.5.2 Outlet Scour Prediction (continued)

Scour Hole Estimation Form
Project Name__Design Example____________________________________ Project No._ADT064____ Subject _______ Scour Hole Example ______________________________________Page _1_ of _____ By ______________________________ Date ______ Checked By ____________________ Date ______

Data Summary		
Parameter	Approach Channel	Downstream Channel
Station	125+50	4+00
Control	Inlet	Supercritical
Type	CBC	Natural
Height, D	6	7.5’
Width, B	7	29’
Length, L	300	---
Material	Concrete	Gravel
Manning’s n	0.012	0.03 & 0.08
Side Slope	---	1:1
Slope	0.05	0.05
Discharge	400	40
Depth, d	1.8’	2.8’
Velocity, V	32 ft/sec	18 ft/sec
Froude No.	4.2	1.9
Flow Area	12.5	22.2

Equation Input Data	
Factor	Value
Discharge, Q cfs	400
Flow, A, Ft^2	42
Wetted Perimeter, Ft	26
Hydraulic Radius, R=A/P	1.62
Discharge Intensity	1.32
Time of Concentration, t, min	30

Scour Equations			
$(d_s/R_c),(W_s/R_c),(L_s/R_c)=C_sC_h[\mathbf{a}/\mathbf{s}^{0.33}]\ [DI]^{\beta}[t/316]^{\theta} = F_1F_2F_3R_c$			
Scour Computation			
Factor	Depth	Width	Length
Alpha, **a**	7.6	26.42	64.54
Beta, **b**	0.26	0.62	0.56
Theta, **Q**	0.09	0.06	0.17
$F_1 = C_sC_h[\mathbf{a}/\mathbf{S}^{0.33}]$	8.6	31.4	75.4
$F_2 = [DI]^{\mathbf{b}}$	0.8	0.9	0.7
$F_3 = [t/316]^{\mathbf{Q}}$			
Scour	7	28	53

Figure 9-2 Scour Hole Estimation Form
Example Problem

9.5 Scour Hole Estimation (continued)

9.5.2.3 Design Example (continued)

Computer Output

The scour hole geometry can also be computed by using the "Energy Dissipators" module of the FHWA microcomputer program HY-8, Culvert Analysis, Version 4.1 or later. A hardcopy of the output of module is as shown below. The dimensions of the scour hole computed by the HY-8 program are pretty close to the values calculated in the previous section.

FHWA CULVERT ANALYSIS, HY-8, VERSION 6.0

CURRENT DATE	CURRENT TIME	FILE NAME	FILE DATE
06-04-1997	10:56:51	CHPTR11A	06-04-1997

CULVERT AND CHANNEL DATA

CULVERT NO. 1	DOWNSTREAM CHANNEL
CULVERT TYPE: 7.0 x 6.0 BOX	CHANNEL TYPE: IRREGULAR
CULVERT LENGTH = 300.0 FT	BOTTOM WIDTH = 7.0 FT
NO. OF BARRELS = 1.0	TAILWATER DEPTH = 2.8 FT
FLOW PER BARREL = 400 CFS	TOTAL DESIGN FLOW = 400 CFS
INVERT ELEVATION = 172.5 FT	BOTTOM ELEVATION = 172.5 FT
OUTLET VELOCITY = 31.3 FPS	NORMAL VELOCITY = 17.5 FPS
OUTLET DEPTH = 3.2 FT	

SCOUR HOLE GEOMETRY AND SOIL DATA

LENGTH = 91.4 FT	WIDTH = 49.3 FT
DEPTH = 9.2 FT	VOLUME = 4609.7 CU FT

MAXIMUM SCOUR OCCURS 36.6 FT DOWNSTREAM OF CULVERT

SOIL TYPE: NONCOHESIVE
SAND SIZES:
D16 = 8 mm
D50 = 14 mm
D84 = 18 mm

9.5 Scour Hole Estimation (continued)

9.5.3 Channel Drop Scour

The "Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona", 1989 presents a discussion on scour below drop structures. The following paragraphs are a summary of that information. Scour below channel drops, such as grade control structures, is a special case of local scour. Where a drop consist of a free, unsubmerged overfall, the depth of scour shall be computed as follows:

$$Z_{lsf} = 1.32\ q^{0.54}\ H_t^{\ 0.225} - TW \qquad \textbf{(9.8)}$$

Where

Z_{lsf} = Depth of local scour due to free-overfall drop measured below the streambed surface downstream of the drop, in ft.

q = Discharge per unit width of the channel bottom, in cfs.

H_t = Total drop in head, measured from the upstream energy grade line to the downstream energy grade line, in feet,

TW = Tailwater depth, in feet.

When the drop is submerged the depth of scour shall be computed as

$$Z_{lss} = 0.581\ q^{0.667}\ (h/Y)^{0.411}[1-(h/Y)]^{-0.118} \qquad \textbf{(9.9)}$$

Where: h/Y <= 0.99 and

Z_{lss} = Depth of scour due to submerged drop measured from the downstream streambed surface, in feet.

q = Discharge per unit width of the channel bottom, in cfs.

h = Drop height in ft.

Y = Downstream depth of flow, ft

If h/Y is greater than 0.85, the predicted scour should also be computed using the free drop equation. The smaller of the two values thus computed should be used.

The longitudinal extent of a scour hole for either free or submerged overfall is calculate as:

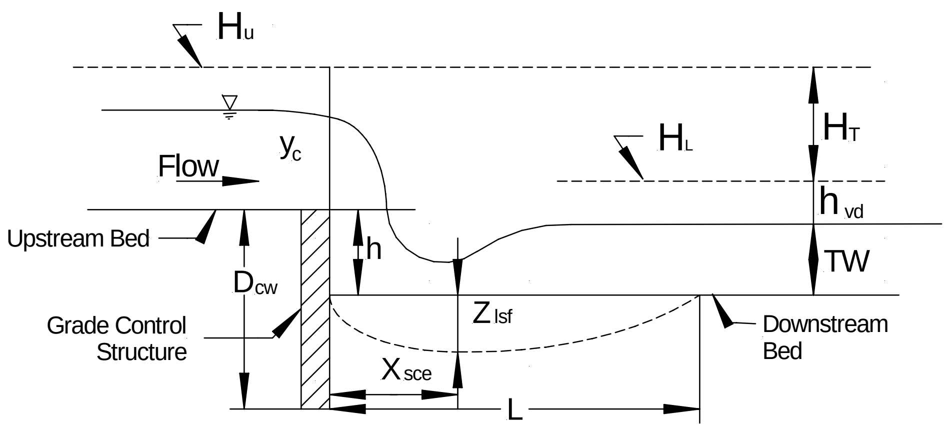
$$x_{sce} = 6.0*Z_{lsf} \text{ or } 6.0*Z_{lss} \qquad \textbf{(9.10)}$$

$$L_s = 12.0*Z_{lsf} \text{ or } 12.0*Z_{lss} \qquad \textbf{(9.11)}$$

Bank protection toe-downs downstream of grade control structures shall extend to the computed depth of scour for a distance equal to x_{sce} beyond the grade-control structure. The toe-down shall then taper to the normal toe-down depth at the distance L_s. Note that L_s includes x_{sce}.

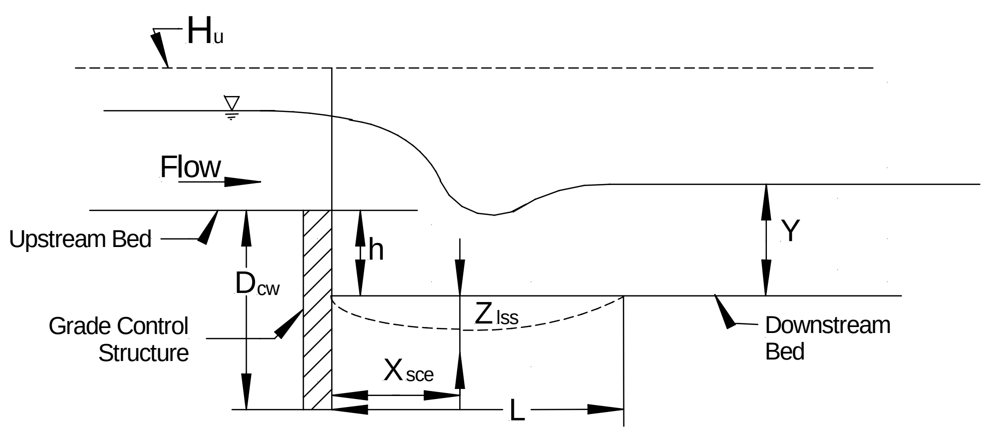
9.5 Scour Hole Estimation (continued)

9.5.3 Channel Drop Scour (continued)



(a)

Scour below Free Outfall



(b)

Scour below Submerged Outfall

Figure 9-3 Scour below outfall: (a) Free (b) Submerged

9.6 Weir Block Basin

9.6.1 Overview

The weir block basin design is based on creating a hydraulic jump by placing an obstruction to the flow. The goal is a velocity reduction that meets the requirements for minimal erosion. Following are the principal features of the basin:

- Using a weir block of at least height Z: that forces a hydraulic jump.
- Constructing the floor at either the culvert outlet or depth of Z below the invert, where Z is the depth necessary to accomplish the required Y_2.
- Sizing the length of the basin to the weir block equal to $5*Y_2$.
- Providing a splash apron of length equal to the greater of L_b (equation 9.10) or 6'.
- Layout details are shown on Figure 9-4a or 9-4b.

Low tailwater, $Y_3<Y_c$

Downstream Flow is supercritical

- A weir block may be sufficient: the critical velocity over the weir will be lower than the downstream velocity.

Medium Depth Tailwater, $Y_c<Y_3<Y_2$

Have a choice of using a small sill to create hydraulic jump or a weir block to force critical depth.

For any sill height, a minimum Froude number is necessary, otherwise it acts as a weir.

$Z<=(F^{5/3})/6$, maximum sill height for raised step behavior.

High Tailwater, $Y_3>Y_2$

Flow will rise to tailwater depth, Y_3, an abrupt rise may be used to control the location of the jump.

9.6.2 Design Procedure : Weir Block

Step 1 Determine Input Flow parameters: (Q, approach depth, and approach velocity)

d_o or d_E, V_o, Fr at the culvert outlet (d_E = the equivalent depth at the brink = $(A/2)^{0.5}$).
Calculate approach Froude number.

Step 2 Determine downstream conditions: (depth, Y_3 and velocity)

Calculate downstream Froude number

9.6 Weir Block Basin (continued)

9.6.2 Design Procedure (continued)

SUPERCRITICAL FLOW

If downstream flow is supercritical, then the only need is to create a weir condition where flow over weir is at critical depth: this will reduce the velocity to less than the downstream velocity.

Step 3 Determine

a. Y_2, the alternate depth

$$\mathbf{Y_2 = (Y_1/2)*[(1+8Fr^2)^{0.5}-1]} \qquad \textbf{(9.3)}$$

Step 4 Size Basin

a. Determine length of the jump, L_j.

$$L_j = 5*Y_2 \text{ minimum.} \qquad \textbf{(9-12)}$$

Check L versus culvert apron length. Is it appropriate to locate weir block on or at end of apron. If length of jump is greater than apron length, add wire-tied apron.

b. B, the width of the weir block perpendicular to flow. Use an expansion ration of 1/(3*Fr) from culvert end. Allow 1' to 2' of open area for outflow of low flows. Place weir block perpendicular to flow.

c. For width B, determine the unit discharge over the weir block and critical depth, $Y_{c.}$

d. Determine height required, Z,

$$Z=Y_2 -Y_c. \qquad \textbf{(9-13)}$$

If Z is greater than $(F^{5/3})/6$, then the flow is analyzed as flow over a weir.
If Z is less than $(F^{5/3})/6$, then the flow is analyzed for an abrupt rise.

e. Determine V_B

- Basin exit depth, d_B = critical depth at basin exit.
- Basin exit velocity, $V_B = Q/(W_B)(d_B)$.
- Compare V_B with the average normal flow velocity in the natural channel, V_d.

Step 5 Determine length of downstream apron, L_B.

$$L_B = 4.3*H*(q^2/(gZ^3))^{0.27} \text{ or} \qquad \textbf{(9-14)}$$

L_B = 6 feet minimum.

9.6 Weir Block Basin (continued)

9.6.2 Design Procedure (continued)

SUBCRITICAL FLOW

If downstream flow is subcritical, then the need is to create a condition where a hydraulic jump occurs: the flow over the end block weir will be subcritical.

Step 3 Determine approach condition: Y_1, V_1, Fr_1.

This is based on the width at the upstream end of the energy dissipator. Can locate beginning of dissipator at the headwall of the culvert or at the end of the apron and wingwalls. If using the location at the headwall, then B is the culvert width, or equivalent width for non-rectangular culverts. If using the end of apron, may need to recalculate the hydraulic parameters based on a flare related to the Froude number.

Step 4 Size Basin

a. Y_2, the alternate depth

$$\mathbf{Y_2 = (Y_1/2)*[(1+8Fr^2)^{0.5}-1]} \qquad \textbf{(9.3)}$$

b. Determine length of the jump, L_j.

$$L_j = 5*Y_2 \text{ minimum.} \qquad \textbf{(9-12)}$$

Check L versus culvert apron length. Is it appropriate to locate an abrupt rise at end of apron. If the length of jump is greater than apron length, add wire-tied apron.

d. Determine height required, Z,

$$Z=Y_2 -TW \qquad \textbf{(9-15)}$$

If Z is greater than $(F^{5/3})/6$, then the flow is analyzed as flow over a weir.
If Z is less than $(F^{5/3})/6$, then the flow is analyzed for an abrupt rise.

e. Determine V_B

- Basin exit depth, TW = Tailwater depth at basin exit.
- Basin exit velocity, $V_B = Q/(W_B)(TW)$.
- Compare V_B with the average normal flow velocity in the natural channel, V_d.

Step 5 Determine length of downstream apron, L_B.

L_B = 6 feet minimum.

9.6 Weir Block Basin (continued)

9.6.3 Design Example

SUPERCRITICAL FLOW

Given:

2-10x4 box culvert, 0° degree skew with a discharge, Q, of 204 cfs and at a slope of 0.0303
Channel is trapezoidal with 10 feet bottom width and 1.5:1, side slopes, Manning's n=0.035, slope=0.0303. Consider weir block at end of concrete apron.

Step 1 Determine Input Flow parameters:(Q, approach depth, and approach velocity)

d_o or d_E, V_o, Fr at the culvert outlet (d_E = the equivalent depth at the brink = $(A/2)^{0.5}$).
Y_1=0.65 ft, V_1=15.6 ft./sec.

Calculate approach Froude number.
Fr=3.39

Step 2 Determine downstream conditions:(depth, Y_3 and velocity)

Y_3=1.77 ft, V_3=9.10 ft./sec.
Calculate downstream Froude number
Fr= 1.33, downstream flow is supercritical, use a weir block.

Step 3 Determine Alternate Depth

$Y_{2,}$ the alternate depth
Y_2=2.83 ft., less than height of box.

Step 4 Size Basin

a. Determine length of the jump, L_j.

$L_j = 5*Y_2$ minimum.

L_j =5*(2.83)=14.2 ft. Length of culvert apron=11.5 ft. Since Y_2 is less than height of box, locate weir block at end of apron. Therefore expansion length=11.5 ft.

b. B, the width of the weir block perpendicular to flow. Use an expansion ration of 1/(3*Fr) from culvert end.

$$B' = B + 2*L_e/(3*F) = 20.83 + \frac{2(11.5)}{3(3.39)}$$

B'=22.3, use B'=24 Ft.

9.6 Weir Block Basin (continued)

9.6.3 Design Example (continued)

SUPERCRITICAL FLOW (continued)

Step 4 Size Basin (continued)

c. For width 'B', determine the unit discharge over the weir block, critical depth, Y_c, and velocity V_c.

$q=Q/B'=204/24= 8.5$ cfs
$Y_c=1.31$ ft.

d. Determine V_B

- Basin exit depth, d_B = critical depth at basin exit.=1.31 Ft.
- Basin exit velocity, $V_B = Q/(W_B)(d_B).=(204)/(24*1.31)=6.5$ ft/sec
- Compare V_B with the average normal flow velocity in the natural channel, V_d.

V_d =9.10 ft/sec.
$V_B < V_d$ ok.

e. Determine height of block required, Z,

$Z=Y_2 - Y_c$.
$Z=2.83-1.31=1.52$ ft., use 2 ft.

If Z is greater than $(F^{5/3})/6$, then the flow is analyzed as flow over a weir.
If Z is less than $(F^{5/3})/6$, then the flow is analyzed for an abrupt rise.

$(F^{5/3})/6=1.28$, $Z>(F^{5/3})/6$, therefore weir flow condition

Step 5 Determine length of downstream splash apron, L_a.

$L_a = 4.3*Z*(q^2/(gZ^3))^{0.27} = 4.3*(2)*(8.5)^2/\{(32.2*(2.0)^3)\}^{0.27}$=6.1 ft, or

= 6 feet, minimum . USE 6' wire-tied apron.

9.6 Weir Block Basin (continued)

9.6.3 Design Example (continued)

SUPERCRITICAL FLOW (continued)

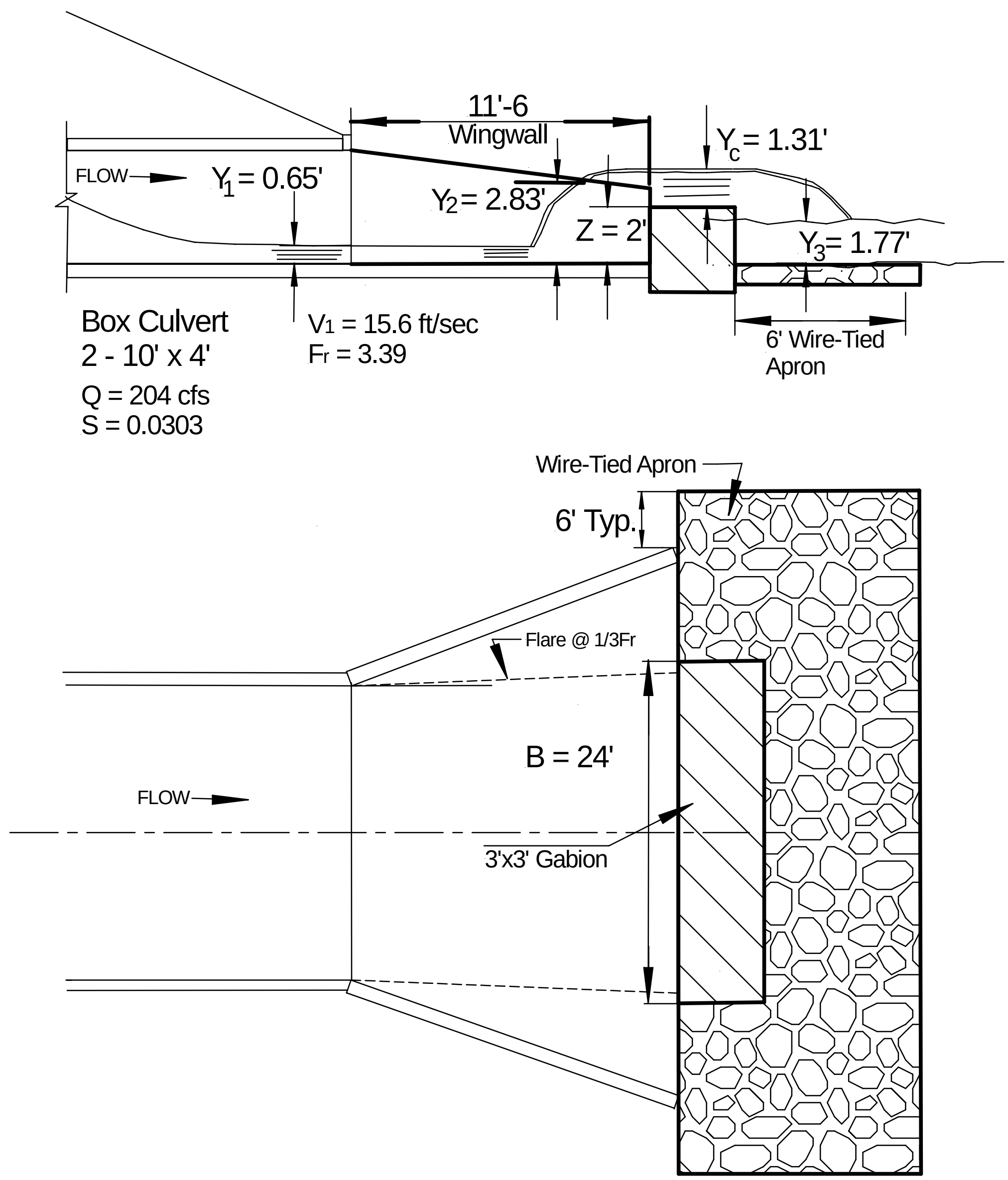


Figure 9-4a, Weir Block layout

9.6 Weir Block Basin (continued)

9.6.3 Design Example

SUBCRITICAL FLOW

Given:

A 2-8x8 box culvert, 0° degree skew with a discharge, Q, of 204 cfs and at a slope of 0.0303
Channel is trapezoidal with 20 feet bottom width and 1.5:1, side slopes, Manning's n=0.035, slope=0.008.
Tailwater depth is 1.77 feet.

Step 1 Determine Input Flow parameters:(Q, approach depth, and approach velocity)
Consider providing energy dissipator downstream of concrete apron. Therefore, parameters are at the end of the apron.
At culvert headwall, Y = 0.76 Ft., V = 16.8 Ft./sec., and Fr = 3.41

W_b = 16.75 + 2(cos(20°)(13.0)/3*3.41 = 16.75+2.42 = 19.17 Ft.
q= Q/ W_b = 204/19.17 = 10.64 cfs.
Y_1=0.67 Ft., V_1=15.8 Ft./sec., Fr=3.40

Step 2 Determine downstream conditions:(depth, Y_3 and velocity)

Y_3=1.77 Ft., V_3= 5.0 Ft./sec.
Calculate downstream Froude number
Fr= 0.70, downstream flow is subcritical, use an abrupt rise.

Step 3 Determine Alternate Depth

$Y_{2,}$ the alternate depth
Y_2=2.91 Ft.

Step 4 Size Basin

a. Determine length of the jump, L_j.
  L_j = 5*Y_2 minimum.
  L_j =5*(2.91)=14.55 Ft.
  Use 15.0 Ft.

b. B, the width of the basin perpendicular to flow.
  Width at end of wingwall = W_b = 16.75 +2(sin(20°)(13.0)) = 25.65 Ft.
  Use 26.0 Ft.

9.6 Weir Block Basin (continued)

9.6.3 Design Example (continued)

SUBCRITICAL FLOW (continued)

Step 4 Size Basin(continued)

c. Determine height of block required, Z,

$Z=Y_2$ -TW.

Z=2.91-1.78=1.13 Ft.

For Abrupt Rise, Maximum Z = $(F^{5/3})/6 = (3.39^{5/3})/6$ = 1.28 Ft.

Use Z = 1.25 Ft.

d. For width B, determine the unit discharge over the weir block, depth, Y, and velocity V.

q=Q/B=204/26.0= 7.85 cfs

$Y= Y_2 - Z$

Y= 2.91 - 1.25 = 1.66 Ft.

e. Determine V_B

- Basin exit velocity, $V_B = Q/(W_B)(Y)$.=(204)/(26.0*1.66)=4.72 Ft./sec.
- Compare V_B with the average normal flow velocity in the natural channel, V_d.

V_d =5.0 Ft./sec.

V_B/V_d = 4.72/5.0 = 0.95, OK.

Step 5 Determine length of apron, L_a.

= 6 feet, minimum . USE 6.0 Ft.

9.6 Weir Block Basin (continued)

Design Example (continued)

SUBCRITICAL FLOW (continued)

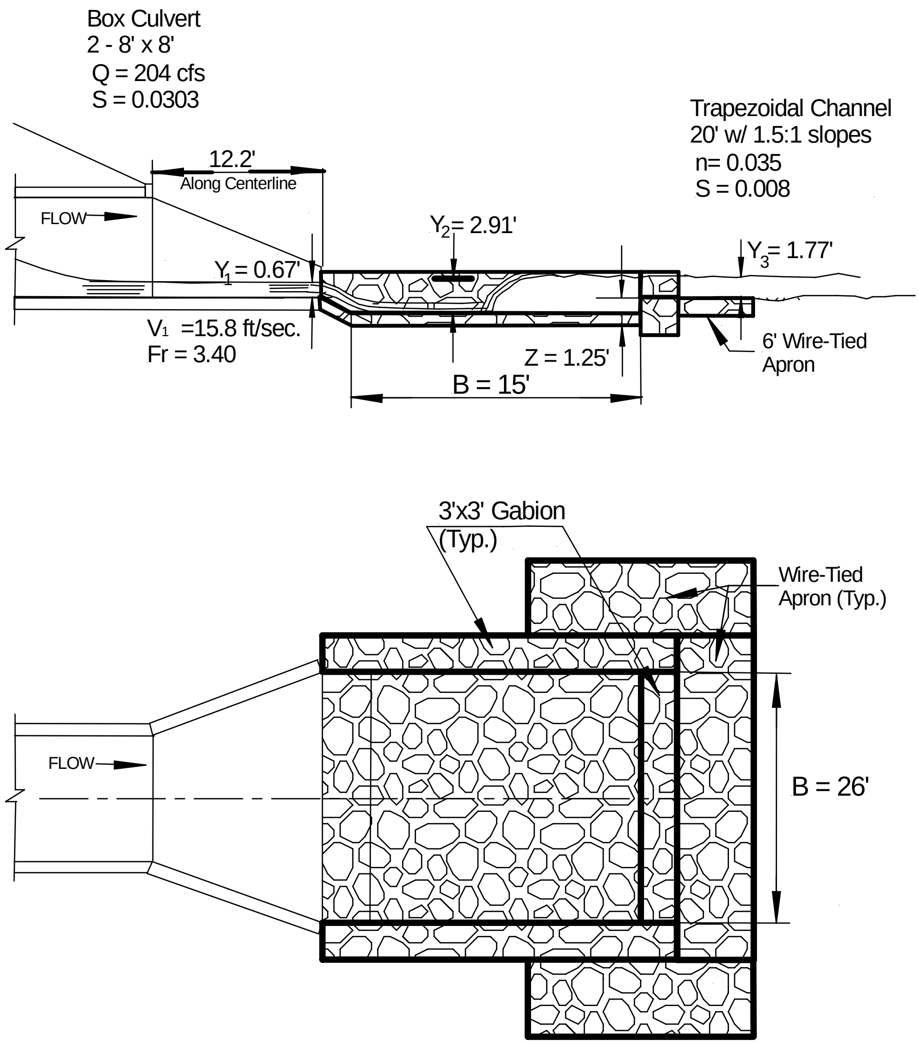


Figure 9-4b, Abrupt Rise Layout

9.7 Impact Basin USBR Type VI

9.7.1 Overview

The USBR Type VI basin, Figure 9-5, was developed by the U.S. Bureau of Reclamation (USBR):

- is referred to as the USBR Type VI basin or hanging baffle,
- is contained in a relatively small box-like structure,
- requires no tailwater for successful performance,
- may be used in open channels as well, and
- is not recommended where debris or ice buildup may cause substantial clogging.

Hanging Baffle

Energy dissipation is initiated by flow striking the vertical hanging baffle and being deflected upstream by the horizontal portion of the baffle and by the floor, creating horizontal eddies.

Notches in Baffle

Notches are provided to aid in cleaning the basin. The notches provide concentrated jets of water for cleaning. The basin is designed to carry the full discharge over the top of the baffle if the space beneath the baffle becomes completely clogged.

Equivalent Depth

This depth must be calculated for a pipe or irregular shaped conduit. The cross section flow area in the pipe is converted into an equivalent rectangular cross section in which the width is twice the depth of flow.

Limitations

Discharges up to 400 cfs per barrel and velocities as high as 50 ft/sec can be used without subjecting the structure to cavitation damage.

Tailwater

A moderate depth of tailwater will improve performance. For best performance, set the basin so that maximum tailwater does not exceed $h_3 + (h_2/2)$.

Slope

If culvert slope is greater than 15°, a horizontal section of at least four culvert widths should be provided upstream.

9.7 Impact Basin USBR Type VI (continued)

9.7.1 Overview (continued)

End Treatment

An end sill with a low-flow drainage slot, 45° wingwalls and a cutoff wall should be provided at the end of the basin.

Apron

An apron of riprap should be placed downstream of the basin for a length of at least four conduit widths.

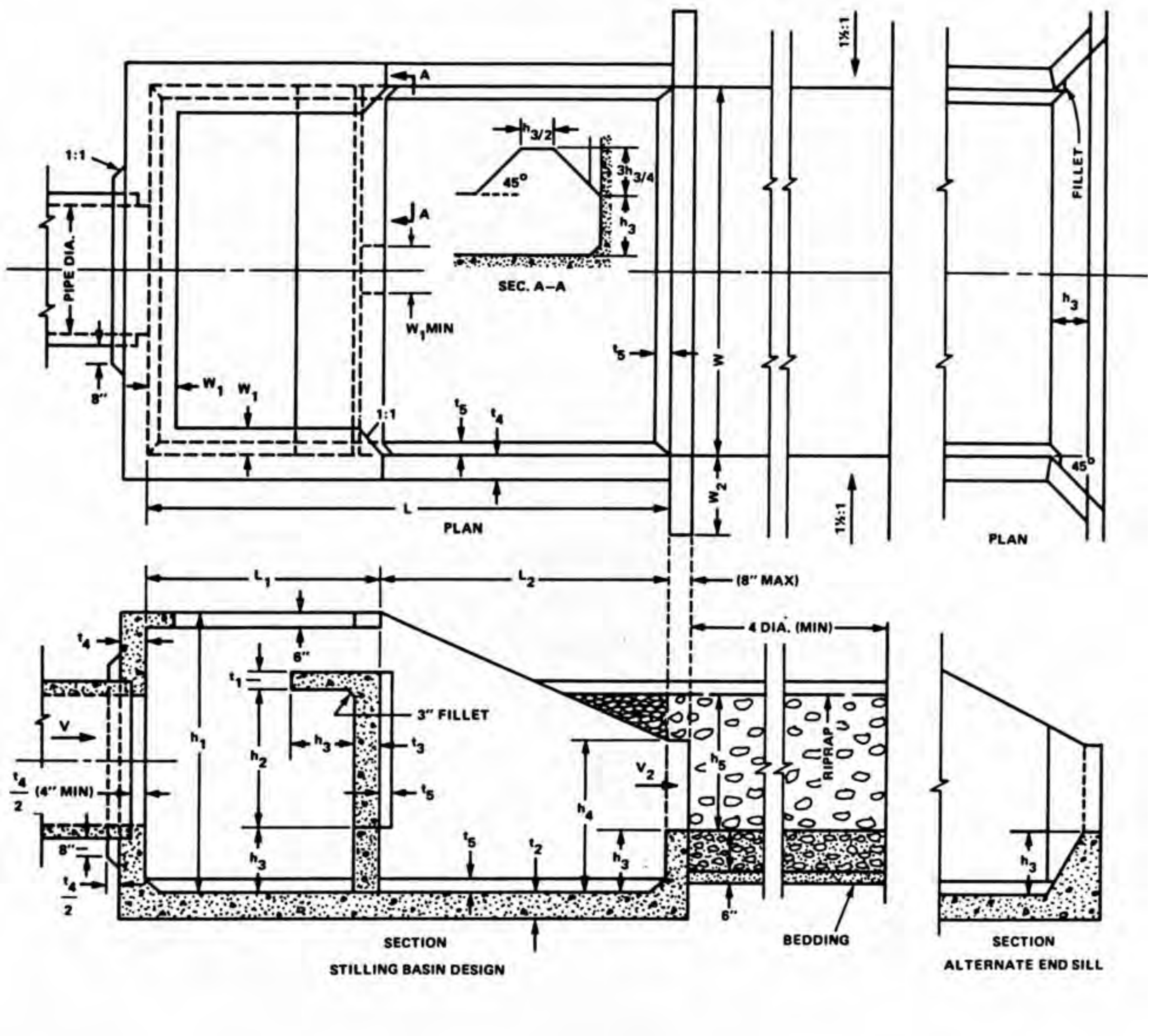


Figure 9-5 USBR Type VI (Impact) Dissipator

9.7 Impact Basin USBR Type VI (continued)

9.7.1 Overview (continued)

Table 9-4 Dimensions Of USBR Type VI Basin
Dimensions in Feet-inches
(See Figure 9-5)

W	h_1	h_2	h_3	h_4	L	L1	L_2
4	3-1	1-6	0-8	1-8	5-5	2-4	3-1
5	3-10	1-11	0-10	2-1	6-8	2-11	3-10
6	4-7	2-3	1-0	2-6	8-0	3-5	4-7
7	5-5	2-7	1-2	2-11	9-5	4-0	5-5
8	6-2	3-0	1-4	3-4	10-8	4-7	6-2
9	6-11	3-5	1-6	3-9	12-0	5-2	6-11
10	7-8	3-9	1-8	4-2	13-5	5-9	7-8
11	8-5	4-2	1-10	4-7	14-7	6-4	8-5
12	9-2	4-6	2-0	5-0	16-0	6-10	9-2
13	10-2	4-11	2-2	5-5	17-4	7-5	10-0
14	10-9	5-3	2-4	5-10	18-8	8-0	10-9
15	11-6	5-7	2-6	6-3	20-0	8-6	11-6
16	12-3	6-0	2-8	6-8	21-4	9-1	12-3
17	13-0	6-4	2-10	7-1	21-6	9-8	13-0
18	13-9	6-8	3-0	7-6	23-11	10-3	13-9
19	14-7	7-1	3-2	7-11	25-4	10-10	14-7
20	15-4	7-6	3-4	8-4	26-7	11-5	15-4

W	W_1	W_2	t_1	t_2	t_3	t_4	t_5
4	0-4	1-1	0-6	0-6	0-6	0-6	0-3
5	0-5	1-5	0-6	0-6	0-6	0-6	0-3
6	0-6	1-8	0-6	0-6	0-6	0-6	0-3
7	0-6	1-11	0-6	0-6	0-6	0-6	0-3
8	0-7	2-2	0-6	0-7	0-7	0-6	0-3
9	0-8	2-6	0-7	0-7	0-8	0-7	0-3
10	0-9	2-9	0-8	0-8	0-9	0-8	0-3
11	0-10	3-0	0-8	0-9	0-9	0-8	0-4
12	0-11	3-0	0-8	0-10	0-10	0-9	0-4
13	1-0	3-0	0-8	0-11	0-10	0-10	0-4
14	1-1	3-0	0-8	1-0	0-11	0-11	0-5
15	1-2	3-0	0-8	1-0	1-0	1-0	0-5
16	1-3	3-0	0-9	1-0	1-0	1-0	0-6
17	1-4	3-0	0-9	1-1	1-0	1-0	0-6
18	1-4	3-0	0-9	1-1	1-1	1-1	0-7
19	1-5	3-0	0-10	1-2	1-1	1-1	0-7
20	1-6	3-0	0-10	1-2	1-2	1-2	0-8

9.7 Impact Basin USBR Type VI (continued)

9.7.1 Overview (continued)

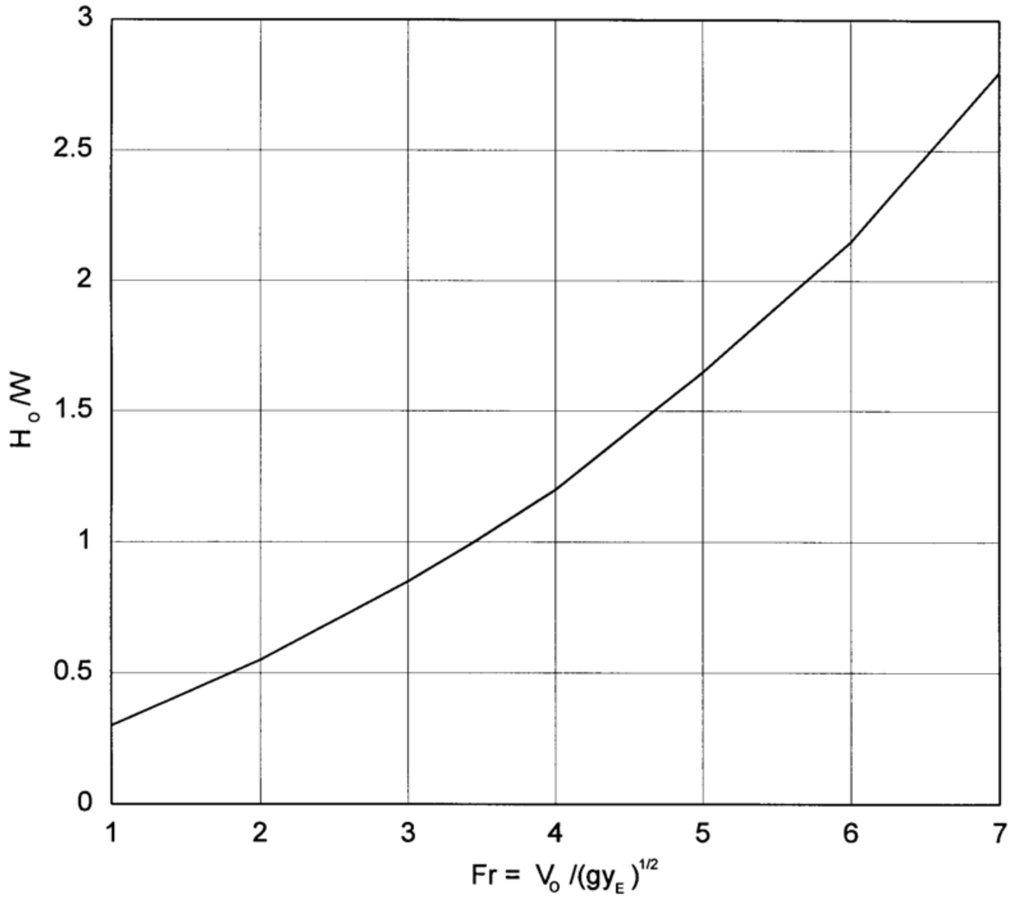


Figure 9-6 Design Curve For USBR Type VI Dissipator

9.7 Impact Basin USBR Type VI (continued)

9.7.1 Overview (continued)

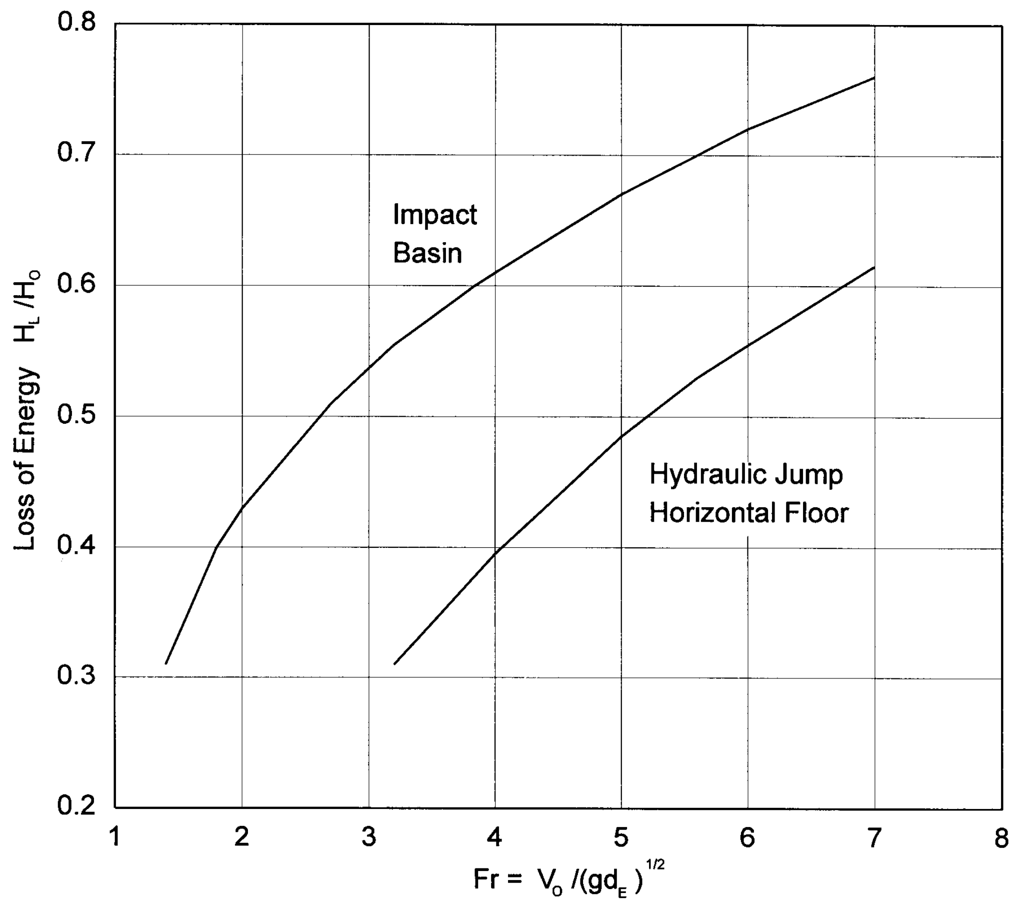


Figure 9-7 Energy Loss For USBR Type VI Dissipator

9.7 Impact Basin USBR Type VI (continued)

9.7.2 Design Procedures

Step 1 Calculate equivalent depth, d_E

a. Rectangular section, $d_E = d_o = y_o$.
b. Other sections, $d_E = (A/2)^{0.5}$.

Step 2 Determine Input Flow

a. Froude number, $Fr = V_o/(gd_E)^{0.5}$.
b. Specific energy, $H_o = d_E + V_o^2/2g$.

Step 3 Determine Basin Width, W

a. Use Figure 9-6.
b. Enter with Fr and read H_o/W.
c. $W = H_o/(H_o/W)$.

Step 4 Size Basin

a. Use Table 9-3 and W.
b. Obtain the remaining dimensions.

Step 5 Energy Loss

a. Use Figure 9-7.
b. Enter with Fr and read H_L/H_o.
c. $H_L = (H_L/H_o)H_o$.

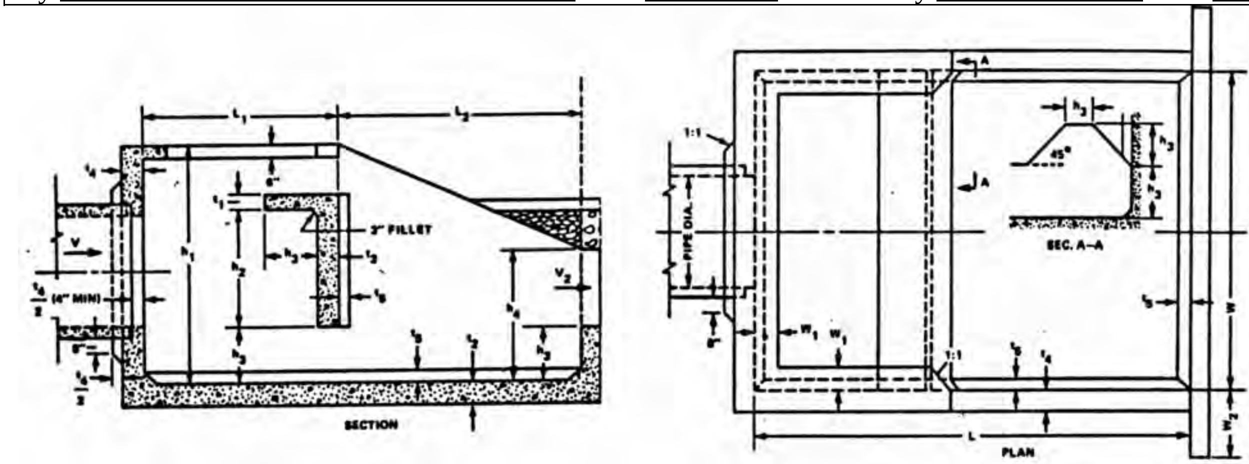
Step 6 Exit Velocity (V_B)

a. Exit energy (H_E) = $H_o - H_L$.
b. $H_E = d_B + V_B^2/2g$
$V_B = (Q/W)/d_B$

9.7 Impact Basin USBR Type VI (continued)

9.7.2 Design Procedures (continued)

Project Name: ____ Project No.: __________
Subject ____ Page ___ of _______
By ________________________________ Date ________ Checked By _____________ Date _______



DESIGN DATA: BASIN WIDTH	TRIALS 1	2	Final
$d_e = y_e$			
V_o			
$H_o = d_e + V_o^2/2g$			
F_r			
H_o /W			
$W = H_o /(H_o/W)$			

CHECK OUTLET VELOCITY, V_B			
H_L/H_o			
$H_L=(H_L/H_o)H_o$			
$H_L=(H_L/H_o)H_o$			
d_B			
$V_B = (Q/W)/d_B$			
$(H_e)_T = d_B+V_B^2/2g$			
IF $(H_e)T$<> H_e, choose another d_B			

BASIN DIMENSIONS, Feet-inches							
W	h_1	h_2	h_3	h_4	L	L_1	L_2
W	W_1	W_2	t_1	t_2	t_3	t_4	t_5

Figure 9-8 USBR Basin Type VI - Design Form

9.7 Impact Basin USBR Type VI (continued)

9.7.3 Design Example

Inputs

D = 48 inch pipe, S_o = 0.15 ft/ft, n = 0.015
Q = 300 cfs, d_o = 2.3 ft., V_o = 40 ft/s

Step 1 Calculate Equivalent Depth, d_E.

Other sections, $d_E = (A/2)^{0.5}$
$A = Q/V_o = 300/40 = 7.5\ ft^2$
$d_E = (7.5/2)^{0.5} = 1.94$ ft.

Step 2 Determine Input Flow

a. Froude number, $Fr_o = V_o/(gd_E)^{0.5}$
$Fr = 40/[32.2(1.94)]^{0.5} = 5.05$
b. Specific energy, $H_o = d_E + V_o^2/2g$
$H_o = 1.94 + (40)^2/(2)(32.2) = 26.8$ ft.

Step 3 Determine Basin Width, W.

a. Use Figure 9-6
b. Enter with Fr = 5.05 and read H_o/W = 1.68
c. $W = H_o/(H_o/W)$ = 23.8/1.68 = 16 ft.

Step 4 Size Basin

a. Use Table 9-3 and W.
b. Obtain the remaining dimensions.

Step 5 Energy Loss

a. Use Figure 9-7, Impact Basin
b. Enter with Fr = 5.05 and read H_L/H_o = 0.67
c. $H_L = (H_L/H_o)H_o$ = 0.67(26.8) = 18 ft.

Step 6 Solve Energy Equation for Exit Velocity (V_B)

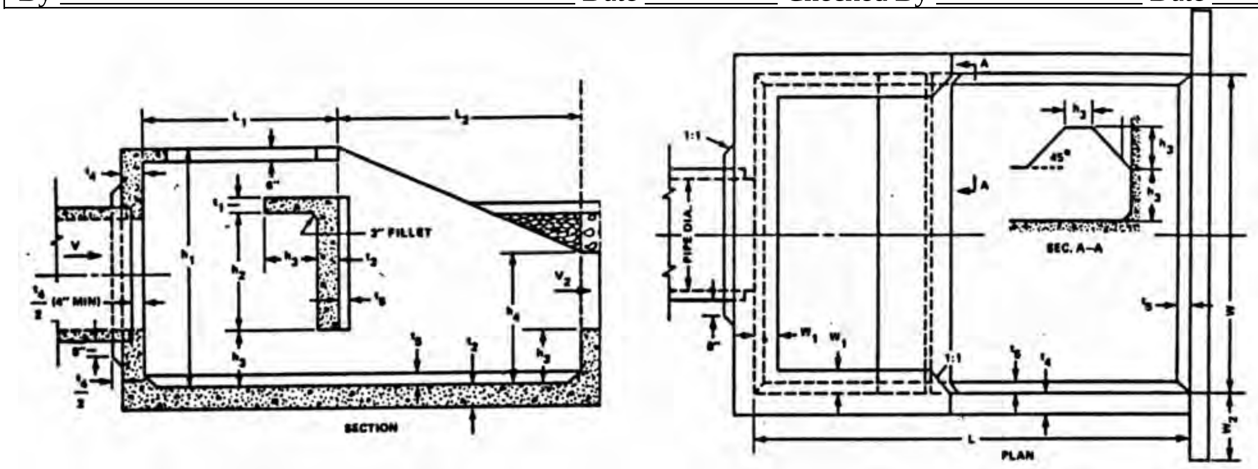
a. Exit energy (H_E) = H_o - H_L = 26.8 - 18 = 8.8 ft.
b. $H_E = d_B + V_B^2/2g$ = 8.8 ft
$V_B = (Q/W)/d_B = (300/16)/d_B = 18.75/d_B$

d_B	V_B	$V_B^2/2g$	$d_B+V_B^2/2g = 8.8$
2.3 = d_c	8.1	1.02	3.3
1.0	18.8	5.49	6.5
0.8	23.4	8.50	9.3
0.9	20.8	6.72	7.6
0.85	22.1	7.58	8.4
0.83	22.6	7.93	8.76 ~ 8.8 use.

9.7 Impact Basin USBR Type VI (continued)

9.7.3 Design Example (continued)

Project Name: _Design Example_______________________________________ Project No.: _ADT064____
Subject ___USBR Type VI Impact Basin___________________________________ Page _1_ of _1_____
By ____________________________________ Date _________ Checked By ______________ Date ________



DESIGN DATA: BASIN WIDTH	TRIALS 1	2	Final
$d_e = y_e$	2.3’		
V_o	40		
$H_o = d_e + V_o^2/2g$	26.8’		
F_r	5.1		
H_o/W	1.68		
$W = H_o/(H_o/W)$	16		

CHECK OUTLET VELOCITY, V_B			
H_L/H_o		0.67	
$H_L=(H_L/H_o)H_o$		18’	
$H_e=H_o-H_L$		8.8	
d_B	2.3’	0.9’	0.8’
$V_B = (Q/W)/d_B$	8.1	20.8	23.4
$(H_e)_T = d_B+V_B^2/2g$	3.3’	7.6’	9.3’
IF $(H_e)_T$ <> H_e, choose another d_B			

BASIN DIMENSIONS, Feet-inches							
W	h_1	h_2	h_3	h_4	L	L_1	L_2
16	12-3	6-0	2-8	6-8	21-4	9-1	12-3
W	W_1	W_2	t_1	t_2	t_3	t_4	t_5
16	1-3	3-0	0-9	1-0	1-0	1-0	0-6

Figure 9-9 USBR Basin Type VI - Design Example

9.7 Impact Basin USBR Type VI (continued)

9.7.4 Computer Results

The dissipator geometry can be computed using the "Energy Dissipator" module that is available in microcomputer program HY-8, Culvert Analysis. The output of the culvert and channel input data, and computed geometry using this module are shown below.

FHWA CULVERT ANALYSIS, HY-8, VERSION 6.0

CURRENT DATE	CURRENT TIME	FILE NAME	FILE DATE
06-02-1997	16:13:53	ENERGY4	06-02-1997

CULVERT AND CHANNEL DATA

CULVERT NO. 1	DOWNSTREAM CHANNEL
CULVERT TYPE: 4 FT CIRCULAR	CHANNEL TYPE: IRREGULAR
CULVERT LENGTH = 300.0 FT	BOTTOM WIDTH = 7.0 FT
NO. OF BARRELS = 1.0	TAILWATER DEPTH = 2.5 FT
FLOW PER BARREL = 300 CFS	TOTAL DESIGN FLOW = 300.0 CFS
INVERT ELEVATION = 172.5 FT	BOTTOM ELEVATION = 172.5 FT
OUTLET VELOCITY = 40.0 FPS	NORMAL VELOCITY = 15.9 FPS
OUTLET DEPTH = 3.94 FT	

USBR TYPE 6 DISSIPATOR — FINAL DESIGN

BASIN OUTLET VELOCITY = 23.4 FPS

W = 16.0 FT	W1 = 1.3 FT	W2 = 3.0 FT
L = 21.3 FT	L1 = 9.1 FT	L2 = 12.3 FT
H1 = 12.3 FT	H2 = 6.0 FT	H3 = 2.7 FT
H4 = 6.7 FT	T1 = 0.8 FT	T2 = 1.0 FT
T3 = 1.0 FT	T4 = 1.0 FT	T5 = 0.5 FT

9.8 SAF Stilling Basin

9.8.1 Overview

The St. Anthony Falls (SAF) stilling basin uses a forced hydraulic jump to dissipate energy, and:

- is based on model studies conducted by US Soil Conservation Service (SCS) at the St. Anthony Falls (SAF) Hydraulic Laboratory of the University of Minnesota;
- uses chute blocks, baffle blocks and an end sill to force the hydraulic jump and reduce jump length by about 80%;
- is recommended where Fr = 1.7 to 17.

9.8.2 Equations

Basin Width, W_B

- for box culvert W_B = B = Culvert width, ft.
- for pipe, use W_B = Culvert diameter, D, ft, or

$$\mathbf{W_B = 0.54Q/D^{1.5}} \qquad \textbf{(9.16)}$$

whichever is larger.

Where: Q = discharge, cfs

Flare (1:z)

Flare is optional, if used it should be flatter than 2:1.

Basin Length, L_B

$$\mathbf{Y_2 = 0.5*Y_1[(1+8*Fr_1^2)^{0.5}-1]} \qquad \textbf{(9.3)}$$

Where: Y_1 = initial depth of water, ft.
Y_2 = sequent depth of jump, ft.
Fr_1= Froude number entering basin, Fr

$$\mathbf{L_B = 4.5\ Y_2/Fr_1^{0.76}} \qquad \textbf{(9.17)}$$

9.8 SAF Stilling Basin (continued)

9.8.2 Equations (continued)

Basin Floor

The basin floor should be depressed below the streambed enough to obtain the following depth (d_2) below the tailwater:

- For Fr_1 = 1.7 to 5.5

$$d_2 = Y_2[1.1 - (Fr_1^2/120)] \quad \textbf{(9.18)}$$

- For Fr_1 = 5.5 to 11

$$d_2 = 0.85\ Y_2 \quad \textbf{(9.19)}$$

- For Fr_1 = 11 to 17

$$d_2 = Y_2[1.1 - (Fr_1^2/800)] \quad \textbf{(9.20)}$$

Chute Blocks

Height, $h_1 = d_1$
Width, W_1 = Spacing, $W_1 = 0.75d_1$
Number of blocks = $N_c = W_B/2W_1$, rounded to a whole number
Adjusted $W_1 = W_2 = W_B/2N_c$
N_c includes the 1/2 block at each wall

Baffle Blocks

Height, $h_3 = d_1$
Width, W_3 = Spacing, $W_4 = 0.75d_1$
Basin width at baffle blocks, $W_{B2} = W_B + 2L_B/3z$
Number of blocks = $N_B = W_{B2}/2W_3$, rounded to a whole number
Adjusted $W_3 = W_4 = W_{B2}/2N_B$
Check total block width to insure that 40 to 55% of W_{B2} is occupied by block.
Staggered with chute blocks
Space at wall $\geq 0.38d_1$
Distance from chute blocks (L_{1-3}) = $L_B/3$

End Sill Height, $h_4 = 0.07d_j$

Sidewall Height = $d_2 + 0.33d_j$

Wingwall Flare = 45°

9.8 SAF Stilling Basin (continued)

9.8.3 Design Procedure

The design of a St. Anthony Falls (SAF) basin consists of several steps as follows:

Step 1 Select Basin Type

a. Rectangular or flared.
b. Choose flare (if needed), 1:z.
c. Determine basin width, W_B.

Step 2 Select Depression

a. Choose the depth d_2 to depress below the streambed, B_d.
b. Assume $B_d = 0$ for first trial.

Step 3 Determine Input Flow

a. d_1 and V_1, using energy equation.
b. Froude Number, Fr_1

Step 4 Calculate Basin Dimensions

a. Y_2 (equation 9.8).
b. L_B (equation 9.9).
c. d_2 (equation 9.10, 9.11, or 9.12).
d. $L_S = (d_2 - TW)/S_S$
e. $L_T = (B_d)/S_T$ (see Figure 9-10).
f. $L = L_T + L_B + L_S$ (see Figure 9-10).

Step 5 Review Results

a. If $d_2 \neq (B_d - LS_o + TW)$ return to Step 2.
b. If approximately equal, continue.

Step 6 Size Elements

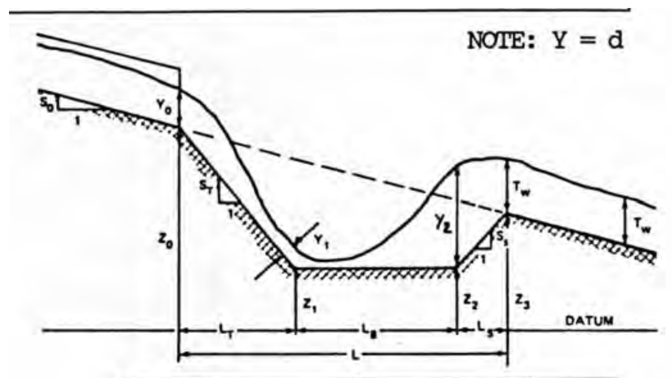
a. Chute blocks (h_1, W_1, W_2, N_c).
b. Baffle blocks (h_3, W_3, W_4, N_B, $L_{1\text{-}3}$).
c. End sill (h_4).
d. Side wall height ($h_5 = d_2 + 0.33d_j$).

9.8 SAF Stilling Basin (continued)

9.8.3 Design Procedure (continued)

SAF Basin		
Project Name: ______________________________	Project No.: __________	
Subject ______________________________	Page ___ of _______	
By ______________________ Date _________	Checked By ______________	Date _______

DATA SUMMARY	TRIALS		
	1	2	3
TYPE			
FLARE (Z:1)			
WIDTH (W_B)			
DEPRESSION (B_d)			
$S_s = S_t$			
DEPTH (d_o)			
VELOCITY (V_o)			
$B_d+d_o+V_o^2/2g$			
DEPTH (d_1)			
$d_1+V_1^2/2g$			
Fr_1			
D_j			
L_B			
D_2			
L_S			
$L_T = (B_d)/S_T$			
$L = L_T+L_B+L_S$			
$B_d - LS_o+TW = d_2$			



Dimensions	TRIALS		
H_1, Chute Block	1	2	Final
W_1			
N_c			
W_2			
H_3, Baffle Block			
W_3			
W_4			
N_B			
H_4, End Sill			
H_5, Side Wall			

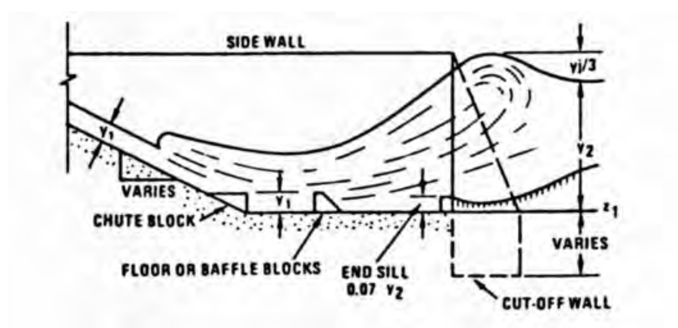


Figure 9-10 - St. Anthony Falls Basin Checklist

9.8 SAF Stilling Basin (continued)

9.8.4 Design Example

- See Section 9.5.2 for input values.
- See Figure 9-12 for completed computation form.

Step 1 Select Basin Type

a. Use rectangular
b. No flare
c. Basin width, W_B =7 ft., unit discharge = 400/7=57.1 cfs

Step 2 Select Depression

Trial 1 B_d = 8 ft., $S_S = S_t = 1$

Step 3 Determine Input Flow

Trial 1
a. Energy equation (culvert to basin):

Culvert outlet = $B_d + d_o + V_o^2/2g = 8 + 1.8 + (32)^2/(2*32.2)=25.7$ ft.
Basin floor = $0 + d_1 + V_1^2/2g$
Solve: $25.7 = d_1 + V_1^2/2g$
Velocity=V_1=q/d_1

d_1	V_1	$V_1^2/2g$	$d_1+V_1^2/2$
1.5	38	22.4	24 < 25.7
1.4	41	26.1	27.5 >25.7, Use.

b. $Fr_1 = 41/(1.4 \times 32.2)^{0.5} = 6.1$

Step 4 Calculate Basin Dimensions

Trial 1
a. d_j = 11.4 ft. (equation 9.8)
b. L_B = 13.0 ft. (equation 9.9)
c. d_2 = 9.7 ft. (equation 9.10)
d. $L_S = (d_2 - TW)/S_S = (9.7 – 2.8)/1 = 6.9$ ft.
e. $L_T = (B_d)/S_T = 8/1 = 8$ ft.
f. $L = L_T + L_B + L_S = 8 + 13 + 7 = 28$ ft.

Step 5 Review Results

Trial 1
a. If d_2 does not equal ($B_d - LS_o + TW$), then adjust drop

$9.7 \neq (8 - 28(0.05) + 2.8) = 9.4$ ft.

b. Add (9.7 – 9.4) = 0.3 ft. more drop and return to Step 2.

9.8 SAF Stilling Basin (continued)

9.8.4 Design Example (continued)

Step 2 Select Depression

Trial 2

B_d = 8.3 ft., $S_S = S_T = 1$

Step 3 Determine Input Flow

Trial 2

a. Energy equation (culvert to basin):

Culvert outlet = $B_d + d_o + V_o^2/2g = 8.3 + 1.8 + (32)^2/2g = 26$ ft.

Basin floor = $0 + d_1 + V_1^2/2g$

Solve: $26 = d_1 + V_1^2/2g$

d_1	V_1	$V_1^2/2g$	$d_1 + V_1^2/2g$
1.4	41	26.1	27.5 ≈ 26, Use.

b. $Fr_1 = 41/(1.4 \times 32.2)^{0.5} = 6.1$

Step 4 Calculate Basin Dimensions

Trial 2

a. d_j = 11.4 ft (equation 9.8)

b. L_B = 13.0 ft. (equation 9.9)

c. d_2 = 9.7 ft. (equation 9.10)

d. $L_S = (d_2 - TW)/S_S$ = 6.9 ft.

e. $L_T = (B_d)/S_T = 8.3/1 = 8.3$ ft.

f. $L = L_T + L_B + L_S = 8.3 + 13 + 7 = 28.3$ ft.

Step 5 Review Results

Trial 2

a. $d_2 = 9.7 \approx (8.3 - 28.3(0.05) + 2.8) = 9.7$ ft. Is approximately equal, continue.

Step 6 Size Elements

Trial 2

a. Chute blocks (h_1, W_1, W_2, N_c)

$h_1 = d_1 = 1.4$ ft.

$W_1 = 0.75d_1 = 1.0$ ft

$N_c = W_B/2(W_1) = 7/2(1) = 3.5$, use 4

Adjusted $W_1 = 7/2(4) = 0.9$ ft $= W_2$

Use 3 full blocks, 4 spaces and a half of block at each wall.

b. Baffle blocks (h_3, W_3, W_4, N_B, L_{1-3})

$h_3 = d_1 = 1.4$ ft.

$W_3 = 0.75d_1 = 1$ ft.

Use 4 blocks, and adjusted as above $W_3 = W_4 = 0.9$ ft.

$L_{1-3} = L_B/3 = 13/3 = 4.3$ ft.

9.8 SAF Stilling Basin (continued)

9.8.4 Design Example (continued)

c. End sill (h_4) = $0.07d_j$ = 0.07(11.4) = 0.8 ft

d. Side wall height (h_5) = $d_2 + 0.33d_j$ = 9.7+0.33*(11.4)= 13.5 ft.

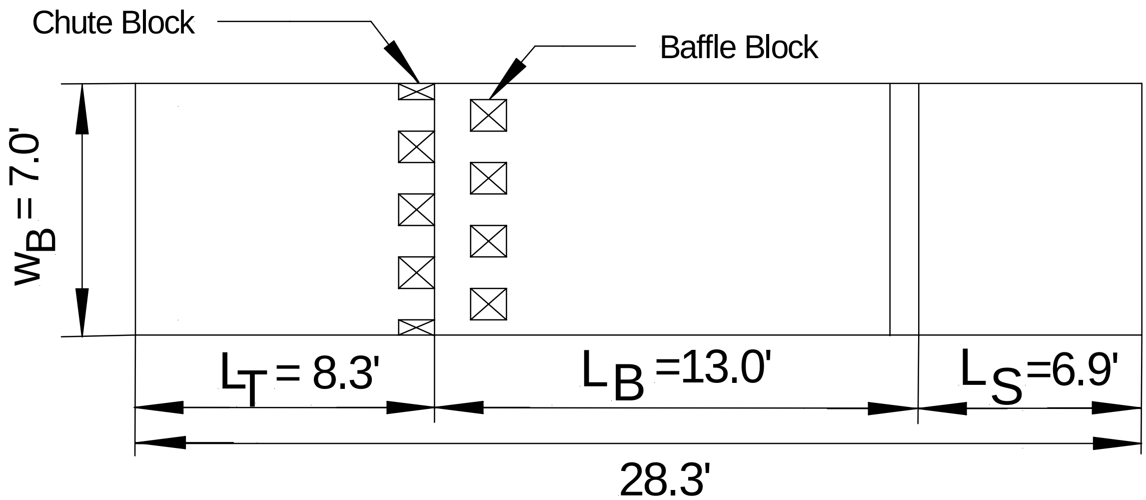


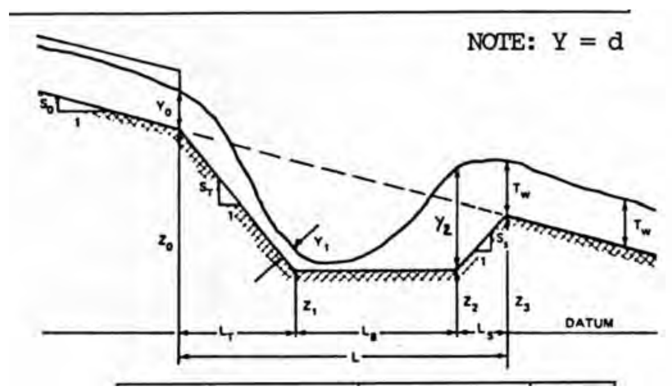
Figure 9-11 SAF Stilling Basin - Design Example

9.8 SAF Stilling Basin (continued)

9.8.4 Design Example (continued)

Project Name: ___Design Example____________________________________ Project No.: _ADT064____
Subject ________SAF Stilling Basin____ Page _1__ of _______
By ____________________________________ Date _________ Checked By _______________ Date ________

DATA SUMMARY	TRIALS		
	1	2	3
TYPE	Rect		
FLARE (Z:1)	0:1		
WIDTH (W_B)	7'		
DEPRESSION (B_d)	8'	8.3	
$S_s = S_t$	1:1	1:1	
DEPTH (d_o)	1.8'	1.8'	
VELOCITY (V_o)	32	32	
$B_d+d_o+V_o^2/2g$	25.7	26.0	
DEPTH (d_1)	1.4	1.4'	
VELOCITY (V_o)	41	41	
$d_1+V_1^2/2g$	27.5	27.5	
Fr_1	6.1	6.1	
d_j	11.4'	11.4	
L_B	13'	13'	
d_2	9.7'	9.7'	
L_S	6.9'	7.0'	
$L_T = (B_d)/S_T$	8'	8.3'	
$L = L_T+L_B+L_S$	28'	28.3'	
$B_d - LS_o+TW = d_2$	9.4<>	=9.7'	



Dimensions	TRIALS		
	1	2	Final
H_1, Chute Block	1.4'		1.4'
W_1	1'		0.9'
N_c	4		4
W_2	0.9		0.9'
H_3, Baffle Block	1.4		1.4'
W_3	1'		0.9'
W_4	1'		0.9'
N_B	4		4
H_4, End Sill	0.8'		0.8'
H_5, Side Wall	13.5'		13.5'

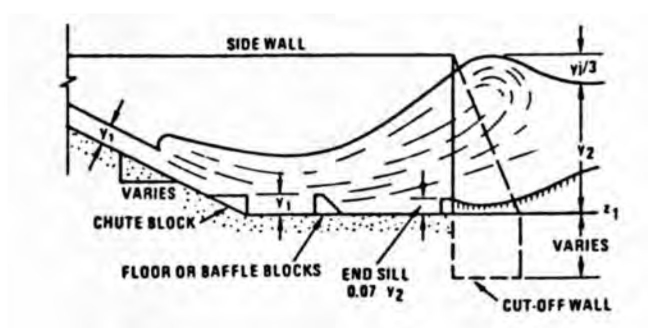


Figure 9-12 St. Anthony Falls Basin Example Problem

9.8 SAF Stilling Basin (continued)

9.8.5 Computer Results

The dissipator geometry can be computed using the "Energy Dissipator" module, which is available in microcomputer program HY-8, Culvert Analysis. The output of the culvert and channel input data, and computed geometry using this module are shown below.

FHWA CULVERT ANALYSIS, HY-8, VERSION 6.0

CURRENT DATE	CURRENT TIME	FILE NAME	FILE DATE
09-19-1996	15:26:05	CHPTR9A	09-19-1996

CULVERT AND CHANNEL DATA

CULVERT NO. 1	DOWNSTREAM CHANNEL
CULVERT TYPE: 7.0 x 6.0 BOX	CHANNEL TYPE: IRREGULAR
CULVERT LENGTH = 300.0 FT	BOTTOM WIDTH = 7.0 FT
NO. OF BARRELS = 1.0	TAILWATER DEPTH = 2.8 FT
FLOW PER BARREL = 400.0 CFS	TOTAL DESIGN FLOW = 400.0 CFS
INVERT ELEVATION = 172.5 FT	BOTTOM ELEVATION = 172.5 FT
OUTLET VELOCITY = 31.3 FPS	NORMAL VELOCITY = 17.5 FPS
OUTLET DEPTH = 2.02 FT	

ST. ANTHONY FALLS BASIN -- FINAL DESIGN

LB = 11.9 FT   LS = 5.8 FT   LT = 7.1 FT
L = 24.8 FT   Y1 = 1.3 FT   Y2 = 8.7 FT
Z1 = 165.4 FT   Z2 = 165.4 FT   Z3 = 171.3 FT
WB = 8.2 FT   WB3 = 8.2 FT

------------------------------ CHUTE BLOCKS ------------------------------

H1 = 1.3 FT   W1 = 1.0 FT   W2 = 1.0 FT   NC = 4.0

------------------------------ BAFFLE BLOCKS ----------------------------

W3 = 1.0 FT   W4 = 1.0 FT   NB = 4.0
H3 = 1.3 FT   LCB = 4.0 FT

------------------------------ END SILL ----------------------------

H4 = 0.7 FT
BASIN OUTLET VELOCITY = 17.5 FPS

9.9 Straight Drop Stilling Basin

9.9.1 Overview

The Straight Drop Stilling basin uses a vertical drop and is rectangular shaped to develop a forced hydraulic jump to dissipate energy:

- was developed for **subcritical** approach flow
- is effective for drop height ratios, H/Y_c <=15 provided the approach width, B is >=1.5 Y_c
- requires a tailwater of at least 2.15 Y_c
- uses a row of blocks and an end sill to force the hydraulic jump

The Straight drop-stilling basin has been used for supercritical flow by using the normal depth or the drawdown depth for the depth at the brink. For supercritical flow, the basin should be greater than determined from figure 9-13.

9.9.2 Equations

Basin Width, W_B

- for box culvert W_B = B = Culvert width, ft.
- for pipe, use W_B = Culvert diameter, D, ft, or

$$\mathbf{W_B = 0.54Q/D^{1.5}} \qquad \mathbf{(9.16)}$$

whichever is larger.

Where: Q = discharge, cfs

Batter

Batter is optional, if used it should be no flatter than 1:1.

Basin Length, L_b

$$L_b = L_A + L_B + L_c \qquad \mathbf{(9.21)}$$

L_A= drop length, figure 9-19

L_B=0.8 Y_c

L_c=2.15 Y_c

$$Y_2 = 0.5*Y_1[(1+8*Fr_1^{2})^{0.5}-1] \qquad \mathbf{(9.3)}$$

Where: Y_1 = initial depth of water, ft.
Y_2 = sequent depth of jump, ft.
Fr_1= Froude number entering basin, Fr

9.9 Straight Drop Stilling Basin (continued)

9.9.2 Equations (continued)

Basin Floor

The basin floor should be depressed below the streambed enough to obtain the following depth (Y_3) below the tailwater:

Y_3=2.15 Y_c **(9-22)**

Baffle Blocks

Height, h_B = 0.8* Y_c **(9-23)**
Width, b_B= (0.4+/-0.15)*Y_c so that blockage is about 50%. **(9-24)**

Basin width at baffle blocks, W_B
Number of blocks = N_B = W_B/ b_B, rounded to a whole number
Adjusted b_B= W_{B2}/2N_B
Check total block width to insure that 40 to 55% of W_B is occupied by block.
Space at wall ≥ 0.38d_1

End Sill Height,

h_4 = 0.4*Y_c **(9-25)**

Sidewall Height

= 0.85*Y_c above tailwater level **(9-26)**
= 2.4*Y_c to 3.0* Y_c

Wingwall Flare = 45°

9.9.3 Design Procedure

The design of a straight drop stilling basin consists of several steps as follows:

Step 1 Determine Input Flow

a. d_1 and V_1.
b. Froude Number, Fr_1
c. Calculate Specific Head, H=yo+Vo^2/2g
d. Calculate Critical Depth, y_c=2H/3

9.9 Straight Drop Stilling Basin (continued)

9.9.3 Design Procedure (continued)

Step 2 Select Basin Dimensions

a. Determine basin width, W_B
b. Calculate the minimum depth of pool, Y_3=2.15 Yc
c. Calculate the distance to tailwater, h_2. In figure 9-13, the crest is the datum, i.e., tailwater below the crest is a negative number. (h_2=distance from crest to floor – depth of water on floor.)

Step 3 Calculate Minimum Length of Basin

$L_b= L_1+ L_2 + L_3$

L_A= drop length, Figure 9-13

L_2=0.8 Y_c

L_3=1.75 Y_c

Step 4 Calculate Size of Blocks

a. Baffle blocks (h_B, b_B, N_B).

Height, h_B = 0.8* Y_c
Width, b_B= (0.4+/-0.15)*Y_c so that blockage is about 50%.

Basin width at baffle blocks, W_B
Number of blocks = N_B = W_B/ 2b_B, rounded to a whole number
Adjusted b_B= W_{B2}/2N_B
Check total block width to insure that 50% to 60% of W_B is occupied by block.
Space at wall ≥ 0.38d_1

b. End sill (h_4).

h_4 = 0.4*Y_c

Step 5 Sidewall Height

.
= 0.85*Y_c above tailwater level
= 2.4*Y_c to 3.0* Y_c

9.9 Straight Drop Stilling Basin (continued)

9.9.3 Design Procedure (continued)

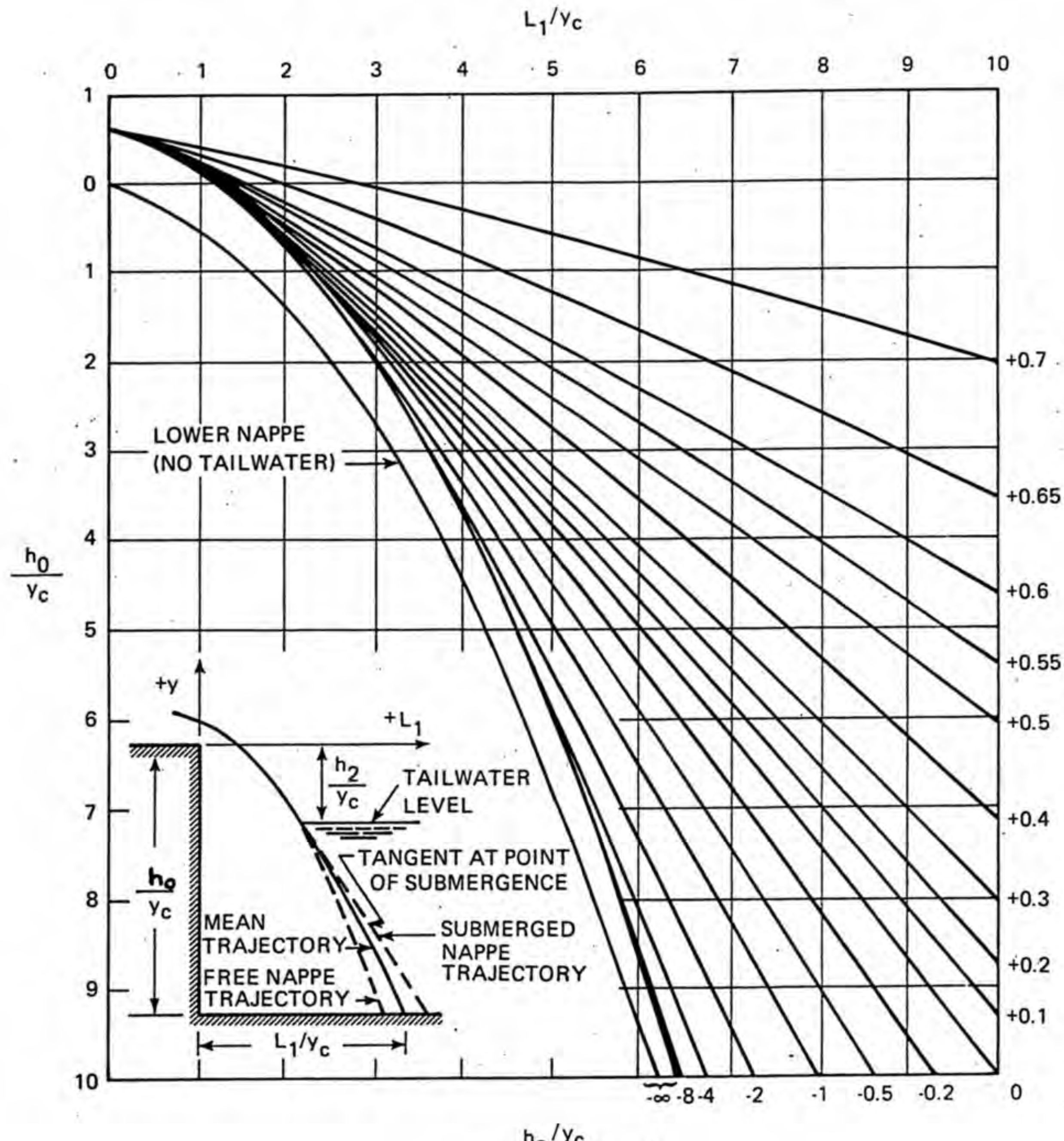


Figure 9-13 Determination of L_1 for Straight Drop Stilling Basin.

9.9 Straight Drop Stilling Basin (continued)

9.9.3 Design Procedure (continued)

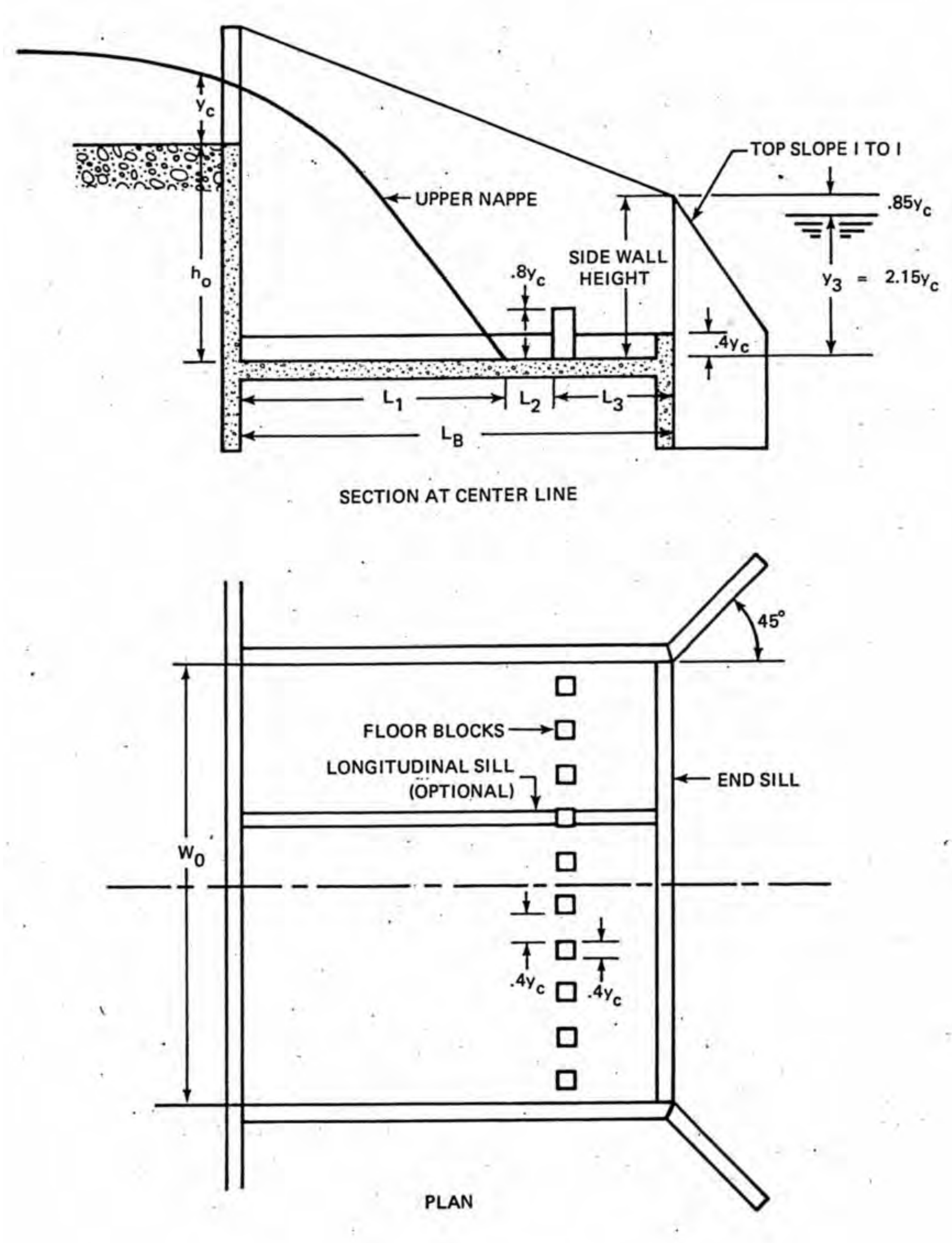


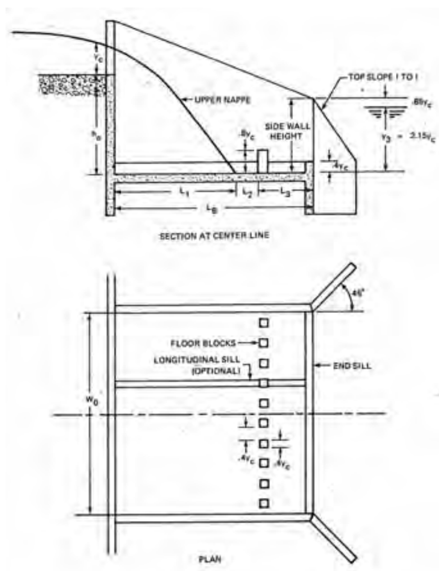
Figure 9-14 Layout of Drop Stilling Basin

9.9 Straight Drop Stilling Basin (continued)

9.9.3 Design Procedure (continued)

Straight Drop Stilling Basin	
PROJECT:________________________________	**PROJECT NO.:**_________________________
SUBJECT____	**PAGE** ____ **OF** ____
BY ____	**DATE** _____________
CHECKED BY ____	**DATE** _____________

Straight Drop Stilling Basin DESIGN VALUES	**TRIAL**	
	1	**2**
Discharge, cfs		
Approach slope		
Manning's n		
Approach Depth,(d_1)		
Approach Velocity, V_1		
Specific Head, $d_1+V_1^2/2g$		
Critical Depth, Y_c		
Approach Froude No, Fr_1		
Sequent Depth, Y_2		
Drop; Crest to channel Invert		
WIDTH (W_B)		
Minimum depth of Pool, Y_3		
Distance to tailwater, h_2		
$h_o= h_2- Y_3$		
Z, depth below channel invert, Drop+ h_o		
h_o/ Y_c		
h_2/ Y_c		
L_1/ Y_c, Fig 9-19		
L_1		
$L_2 = 0.8 Y_c$		
$L_3 >= 1.75 Y_c$		
$L_B= L_1 + L_2 + L_3$		



DIMENSIONS OF ELEMENTS	TRIAL		
	1	2	FINAL
Baffle Bocks			
$h_B = 0.8 Y_c$,			
$b_B = 0.4 Y_c$			
Spacing			
$NB = W_B/2 b_B$			
H_E, END SILL, 0.4 Y_c			
H_{SW}, SIDEWALL, 3 Y_c			

Figure 9-15 Straight Drop Stilling Basin Form

9.9 Straight Drop Stilling Basin (continued)

9.9.4 Design Example

- See Section 9.6 for input values.
- See Figure 9-15 for computation form.

Step 1 Determine Input Flow Parameters

Approach and downstream channel, trapezoidal with b=10’ Side slopes 3:1
So=0.002, n=0.03

Q=250 cfs, Drop from crest to invert of downstream channel=6.0 feet

a. Y_o and V_o.
Y_o=3.36 ft V_o=3.7 fps

Therefore drop to tailwater is 6.0-3.36 = 2.64 ft.

b. Froude Number, Fr_1

$Fr_1= V_o/(g*Y_o)^{0.5}=0.44$

c. Calculate Specific Head, H

$H= Y_o + V_o{}^2/2g=3.36+(3.7)^2/(2*32.2)= 3.57$ ft.

d. Calculate Critical Depth, Y_c
$Y_c =2H/3=2*3.57/3=2.38$ ft

Step 2 Select Basin Dimensions

a. Determine basin width, W_B
Use approach channel top width, 10+2*3*3.36=30.16

b. Calculate the minimum depth of pool,
$Y_3=2.15\ Y_c =2.15*2.38=5.12$ ft

c. Calculate the distance to tailwater, h_2.
h_2=-(distance from crest to channel invert – depth of water on downstream channel.)

$h_2=-(6.00-3.36)=-2.64$

9.9 Straight Drop Stilling Basin (continued)

9.9.4 Design Example (continued)

Step 2 Select Basin Dimensions (continued)

d. Calculated distance from crest to floor: h_0
$h_0 = h_2 - Y_3$
$h_0 = -2.64 - 5.12 = -7.76$ ft.

Therefore, the floor of the stilling basin is 1.76 feet below the grade line of the downstream channel.

Step 3 Calculate Minimum Length of Basin

$$L_b = L_1 + L_2 + L_3 \qquad \textbf{(9.9)}$$

L_1= drop length, Figure 9-13, Need ratios h_0/Y_C and h_2/Y_C.

$h_0/Y_C = -7.76/2.38 = -3.26$ $h_2/Y_C = -2.64/2.38 = -1.11$

from Figure 9-13,
$L_1/Y_C = 3.95$
$L_1 = 3.95*(2.38) = 9.4$ ft.

$L_2 = 0.8\ Y_c = 0.8(2.38) = 1.9$ ft

$L_3 = 1.75\ Y_c = 1.75(2.38) = 4.2$ ft

$L_b = L_1 + L_2 + L_3 = 9.4 + 1.9 + 4.2 = 15.5$ ft.

Step 4 Calculate Size of Blocks
a. Baffle blocks (h_b, b_B, N_B).

Height, $h_B = 0.8*Y_c = 0.8(2.38) = 1.90$ ft.

Width, $b_B = 0.4*Y_c = 0.4(2.38) = 0.95$ ft, use 1.0 ft.

Number of blocks = $N_B = W_B/2b_B$,
=20/2*1=10

spacing = (20-10*1)/10=1 ft.,
set spacing at wall to be one-half of normal spacing.

9.9 Straight Drop Stilling Basin (continued)

9.9.4 Design Example (continued)

Step 4 Calculate Size of Blocks (continued)

b. End sill (h_4).
$h_4 = 0.4*Y_c = 0.4(2.38) = 0.95$

Step 5 Sidewall Height
.
$= 0.85*Y_c$ above tailwater level

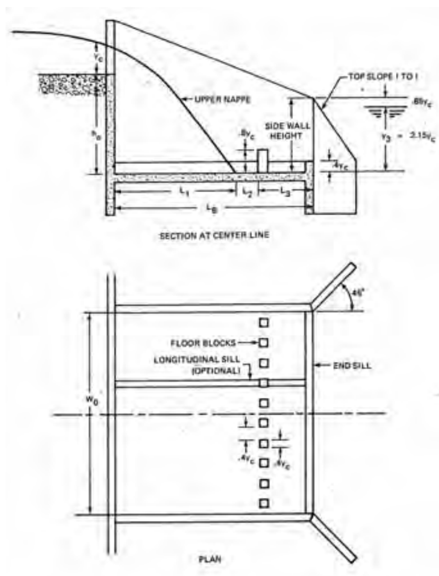
$=0.85(2.38)+3.36=1.9+3.36=5.26$ ft above channel invert.

9.9 Straight Drop Stilling Basin (continued)

9.9.4 Design Example (continued)

Straight Drop Stilling Basin
PROJECT:______________________________ PROJECT NO.:________________________
SUBJECT____ PAGE ____ OF ____
BY ____ DATE ____________
CHECKED BY ____ DATE ____________

Straight Drop Stilling Basin DESIGN VALUES	**TRIALS**	
	1	**2**
Discharge, cfs	250	
Approach slope	0.002	
Manning's n	0.03	
Approach Depth,(d_1)	3.36	
Approach Velocity, V_1	3.7	
Specific Head, $d_1+V_1^2/2g$	3.57	
Critical Depth, Y_c	2.38	
Approach Froude No, Fr_1	0.44	
Sequent Depth, Y_2		
Drop; Crest to channel Invert	6.0	
WIDTH (W_B)	30.16	
Minimum depth of Pool, Y_3	5.12	
Distance to tailwater, h_2	-2.64	
$h_o= h_2- Y_3$	-7.76	
Z, depth below channel invert, Drop+ h_o	-1.76	
h_o/ Y_c	-3.26	
h_2/ Y_c	-1.11	
L_1/ Y_c, Fig 9-19	3.95	
L_1	9.4	
$L_2 = 0.8\ Y_c$	1.9	
$L_3 >= 1.75\ Y_c$	4.2	
$L_B= L_1 + L_2 + L_3$	15.5	



DIMENSIONS OF ELEMENTS	TRIALS		
	1	2	FINAL
Baffle Bocks			
$h_B = 0.8\ Y_c$,	1.9		1.9
$b_B = 0.4\ Y_c$	0.95		1
Spacing	0.95		1
$NB = W_B/2\ b_B$	15.87		15
H_E, END SILL, $0.4Y_c$	0.95		1
H_{SW}, SIDE WALL, $3Y_c$	7.14		7

Figure 9-16 Straight Drop Stilling Basin Form, Example

9.10 References

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Appendix A.

Pipe Characteristics:

Diameter, inches	Area, Ft^2	Perimeter, Ft.	Hydraulic Radius, Ft.
24	3.14	6.28	0.500
30	4.91	7.85	0.625
36	7.07	9.42	0.750
42	9.62	11.00	0.875
48	12.57	12.57	1.00
54	15.90	14.14	1.125
60	19.63	15.71	1.25
66	23.76	17.28	1.375
72	28.27	18.85	1.50

CHAPTER 10

BRIDGE HYDRAULICS

Chapter 10 Bridge Hydraulics
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Chapter 10 Bridge Hydraulics
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10.1 Introduction

10.1.1 Overview

This chapter provides general design guidance for the design of stream crossing system utilizing bridges through:

- presentation of the appropriate design philosophy, goals, and considerations.

- discussion of the technical aspects of hydraulic design including a design procedure which emphasizes hydraulic analyses using the computer program HEC-RAS.

- presentation of equations and methodology for scour analysis.

Waterway bridges are structures that carry traffic over a waterway: the stream crossing system includes the approach roadway, relief openings, when present, and the bridge structure. A more in-depth discussion of the philosophy of Bridge Hydraulics is presented in the AASHTO Highway Drainage Guidelines, Chapter VII(1). Hydraulics of culverts used for stream crossings should be analyzed in accordance with Chapter 8, Culverts.

10.1.2 Objectives

The objective in performing a hydraulic design of a stream crossing system is to provide a cost-effective crossing that satisfies criteria regarding the desired level of hydraulic performance at an acceptable level of risk. The relevant areas for consideration are impact on the stream environment, hydraulic performance, and total economic costs for the stream crossing system: the total cost includes construction, maintenance and risk costs of traffic delay, repair and liability.

The desired level of hydraulic performance can be generally described as:

- to not adversely impact adjacent properties
- to not significantly adversely impact the stream and its environment outside the project
- to have the bridge withstand all flow events up to and including a “Superflood” event

These are further quantified in section 10.3. To meet the stated goals, a hydraulic analysis of the stream crossing system must be performed. The questions to be addressed are:

1.) Determine the changes in stream behavior at the project-crossing site. This includes both the naturally occurring changes as part of the stream morphology and those related to the construction of the project.

2.) The methods used to quantify the changes include a multi-level geomorphic analysis, a steady-state hydraulic analysis, and prediction of scour at the project site.

The design of a cost-effective stream crossing system requires a comprehensive engineering approach that includes data collection, formulation of alternatives, selection of the most cost effective alternative satisfying the established criteria, and documentation of the final design. Water surface elevations and profiles affect the highway and bridge design and are the mechanism for determining the effect of a bridge opening on upstream and downstream water levels.

10.2 Definitions, Symbols and Abbreviations

10.2.1 Definitions

Armoring is a natural process by which the stream removes only particles that are smaller than the transport size, leaving particles greater than the displaced size to "armor" the bed. Usually this layer is of only one particle thickness. It is **NOT** to be counted upon to resist scour during design storm events.

Bridge Modeling Cross-sections

Approach cross-section (Section 1). Section describing the conditions upstream of the effects of the roadway embankment.

BU, section describing the conditions at the upstream end of the roadway embankment.

BD, Section describing the conditions at the downstream end of the roadway embankment.

Exit cross-section, section describing the conditions of the effects of the roadway embankment

Backwater (h1) is the increase in water surface at section. For bridge modeling it is measured relative to the normal water surface elevation without the effect of the bridge at the approach cross-section (Section 1). It is the result of contraction and re-expansion head losses and head losses due to bridge piers. Backwater can also be the result of a "choking condition" in which critical depth is forced to occur in the contracted opening with a resultant increase in depth and specific energy upstream of the contraction. This is illustrated in Figure 10-2.

Contraction is the reduction in channel width.

Free surface flow is flow at atmospheric pressure. At a bridge opening, it is assumed until flow depth is 1.1 times the hydraulic depth of the opening.

Hydraulic depth is an equivalent depth of flow. It is determined as flow area divided by top width, A/TW.

Normal water surface is the water surface at a section that would occur if there was no impact from the project.

Pressure flow is flow at a pressure higher than atmospheric. This occurs when an opening is totally under water, when the flow comes in contact with the low chord of the bridge. Also called orifice flow.

Weir flow is flow that is under atmospheric pressure and is flowing over an obstruction.

Scour is the removal of material from the bed or bank of a stream.

Submergence is the ratio of the depth of water downstream side divided by the height of the energy grade line above the minimum weir elevation on the upstream side. The submergence must exceed 75% for any reduction in flow to take place.

10.2 Definitions, Symbols and Abbreviations (continued)

10.2.1 Definitions (continued)

Thalweg is the loci of points that describe the bottom of channel flow line.

Steady Flow Water Surface Profiles. The steady flow component in HEC-RAS is capable of modeling subcritical, supercritical, and mixed flow regime water surface profiles. The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situations where the water surface profile is rapidly varied. These situations include mixed flow regime calculations (hydraulic jumps), hydraulics of bridges, and evaluating profiles at river confluences (stream junctions).

10.2 Definitions, Symbols and Abbreviations (continued)

10.2.2 Symbols

Symbol	Description	Units
A	Area of cross-section of flow	ft^2
d	Depth of flow	ft.
L_{1-2}	Length between section 1 and section 2	ft.
L_{2-3}	Length between section 2 and section 3	ft.
L_{3-4}	Length between section 3 and section 4	ft.
n	Manning's roughness coefficient	-
P	Wetted perimeter	ft.
Q_{mc}	Flow in main channel	cfs
$Q_{lob,rob}$	Flow in overbank, either left or right	cfs
Q_w	Flow approaching scour hole within the scour width under consideration	cfs
R	Hydraulic radius, (A/P)	ft.
S_o	Slope of channel	ft./ft.
TW	Top width	ft.
WSE	Water surface elevation	ft.
Y_a	Depth of flow approaching scour hole, relates to Q_w	ft.
Y_o	Depth of flow	ft.

10.2.3 Abbreviations

FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
NFIP	National Flood Insurance Program

10.3 Hydraulic Design Goals

10.3.1 General Goals

The process to select the recommended stream crossing system uses the evaluation of alternatives for site-specific criteria including capital costs, traffic service, environmental and property impacts, hazard to human life, and hydraulic performance.

The hydraulic performance evaluation of an alternative is based on the success in meeting the goals for the project.

- **Backwater should not significantly increase flood impact to property.**
- **The existing flow distribution is maintained to the extent practicable.**
- **Increased velocities should not damage either the structure or significantly increase impact to adjacent property.**
- **Scour shall not cause the failure of the bridge.**
- **Life cycle costs for construction, maintenance, and operation are minimized.**

These goals, criteria and possible actions to address these goals are further discussed below.

Goal 1. Backwater should not significantly increase flood impact to property.

The interest of other property owners must be considered in the design of a proposed stream-crossing system. Not all stream crossing systems can be designed to economically pass all possible flows without backwater effects, therefore the effects of flows greater than the roadway operational flow should be evaluated. Overtopping of the roadway may be used to control backwater levels for flows greater than the operational flow. Embankment overtopping incorporated into the design should be located well away from the bridge abutments and superstructure.

In delineated floodplains, whenever practicable, the stream-crossing system shall avoid encroachment on the **floodway** within a flood plain. When this is not feasible, modification of the floodway itself shall be considered. If neither of these alternatives is feasible, FEMA regulations for "floodway encroachment where demonstrably appropriate" shall be met.

Backwater/Increases Over Existing Conditions

The effects of the bridge backwater changes in water surface profiles shall be evaluated at the right-of-way line and shall:

- consider the impact on adjacent properties during the passage of the 1% exceedence probability flood for sites not covered by NFIP.

- conform to FEMA regulations for sites covered by the National Flood Insurance Program (NFIP). See Chapter 2 Legal Aspects for discussion of interaction with FEMA.

10.3 Hydraulic Design Goals (continued)

10.3.1 General Goals (continued)

Waterway Enlargement

There are situations where the proposed roadway and structural designs regarding the vertical positioning of a bridge result in a small vertical clearance between the low chord and the ground. Significant increases in span length provide small increases in effective waterway opening in these cases. Although it may seem possible to increase the effective area by excavating a flood channel through the reach, the desired long-term hydraulic performance does not usually result. The design usually fails to address the issue of a stable channel in regards to erosion, scour or aggradation and deposition. The use of waterway enlargement is discouraged. Its use will require approval from ADOT Drainage Section.

Goal 2. The existing flow distribution is maintained to the extent practicable.

Flow Distribution

The conveyance of the proposed stream-crossing location shall be calculated to determine the flow distribution and to establish the location of bridge opening(s). The proposed facility shall not cause any significant change in the existing flow distribution. Relief openings in the approach roadway embankment should be evaluated when there are flow concentration points that would be blocked by a single bridge opening.

Auxiliary Openings

Stream-crossing systems on wide floodplains may result in the need for auxiliary waterway openings. The purpose of the auxiliary openings on the floodplain is to pass a portion of the flood flow in the floodplain when the stream reaches a certain stage. It does not provide relief for the principal waterway opening in the sense that an emergency spillway at a dam does, but has predictable capacity during flood events.

Basic objectives in choosing the location of auxiliary openings include:

- maintenance of flow distribution and flow patterns,
- accommodation of relatively large flow concentrations on the flood plain,
- avoidance of flood plain flow along the roadway embankment for long distances, and
- crossing of significant tributary channels.

The most complex factor in designing auxiliary openings is determining the division of flow between the two or more structures. If incorrectly proportioned, one or more of the structures may be overtaxed during a flood event. Auxiliary openings should usually be generously sized to guard against that inability to adequately determine the flow distribution. The technological weakness in modeling auxiliary openings is in the use of one-dimensional models to analyze two-dimensional flow.

Goal 3. Increased velocities should not damage either the structure or significantly increase impact to adjacent property.

Velocity

The velocity changes should be minimal. Velocities that result in predicted bed or bank erosion shall be addressed. Erosion protection in the form of bed or bank protection shall be included in the stream crossing system. Whenever possible, clearing of vegetation upstream and downstream of the toe of the embankment

10.3 Hydraulic Design Goals (continued)

10.3.1 General Goals (continued)

slope should be avoided. For the operational flow, wire-tied riprap, railbank, or soil cement bank protection is used to protect the roadway embankment or abutment fill slopes. Bank protection design information is presented in Chapter 11.

Spur dikes should be used, as necessary, to align flows, protect roadway embankments, and mitigate the effects of changes in the stream hydraulic behavior. Spur dikes are recommended to align the approach flow with the bridge opening and to mitigate scour for the operational flow at the abutments. They are usually elliptical shaped with a major to minor axis ratio of 2.5 to 1. Their length can be determined according to HDS-1 (2) or by 7 times the depth of abutment scour to be mitigated. Spur dikes and embankments shall be protected by bank protection for flows up to the operational flow.

Goal 4. Scour shall not cause the failure of the bridge.

Scour

Stream effects causing scour for flows up to and including the 500-year event shall be determined. Scour should not be aggravated due to excessive constriction nor improper alignment. If overtopping of the bridge highway system occurs at less than Q500, the overtopping flow and its impact shall also be evaluated. Debris build-up on foundation elements in the flow shall be included in the determination of design scour. The bridge shall withstand the scour effects for the greatest discharge passing through the bridge up-to and including the “superflood” event.

Goal 5. Costs for construction, maintenance, and operation are minimized.

Operational Frequency

Infringement of storm water into the desired freeboard of the roadway determines the operational level of traffic service provided by the facility. Desired minimum levels of operational frequency for travelway inundation of specific roadways are presented in Chapter 600 of The Highway Design Manual.

Bridge Freeboard Clearance

A minimum clearance of 3 ft for Level 1 bridges or 1 foot for Level 2,3 or 4 bridges shall be provided between the design approach water surface elevation and the low chord of the bridge for the selected design alternative.

10.4 General Considerations

10.4.1 Location Considerations

The primary drainage consideration for facility location in highway planning is the evaluation of the impact of floodplain encroachments of a stream crossing. Hydraulic and environmental considerations of highway river crossings and encroachments are presented in the FHWA Highways in the River Environment, Training and Design Manual (1990). The Manual identifies possible local, upstream and downstream effects of highway encroachments.

The principal hydraulic factors to be considered in locating a stream crossing that involves encroachment within a floodplain are:

- river type (straight or meandering),
- river characteristics (stable or unstable),
- river geometry and alignment,
- hydrology,
- hydraulics,
- floodplain flow, and
- economic and environmental concerns.

A detailed evaluation of these factors is part of the location hydraulics study. When a suitable crossing location has been selected, specific crossing components can then be determined. Exact information on these components is usually not developed until the final stage.
When necessary, these include:

- the geometry and length of the approaches to the crossing,
- probable type and approximate location of the abutments,
- probable number and approximate location of the piers,
- estimated depth to the footing supporting the piers (to protect against local scour),
- the location of the longitudinal encroachment in the floodplain,
- the amount of allowable longitudinal encroachment into the main channel, and
- the required river training works to ensure that river flows approach the crossing or the encroachment in a complementary way.

10.4.2 Morphology

A stream is a dynamic natural system that as a result of the encroachment caused by elements of a stream-crossing system will respond in ways that may be unexpected. Among the many hydraulic factors that affect and need to be considered in the design of a stream crossing system are: flood plain width and roughness, flow distribution and direction, bed slope, stream type (braided, straight, or meandering), stream regime (aggrading, degrading, or equilibrium), and stream controls. The hydraulics of a proposed location may affect environmental considerations such as aquatic life, wetlands, sedimentation, and stream stability. The history of the stream must be considered, including assessment of long-term trends in aggradation or degradation.

The inherent complexities of stream stability, further complicated by highway stream crossings, requires a multilevel procedure. The evaluation and design of a highway stream crossing or encroachment should begin with a qualitative assessment of stream stability. This involves application of geomorphic concepts to identify potential problems and alternative solutions. This analysis should be followed with quantitative

10.4 General Considerations (continued)

10.4.2 Morphology (continued)

analysis using basic hydrologic, hydraulic and sediment transport engineering concepts. Such analyses could include evaluation of flood history, channel hydraulic conditions (up to and including, for example, water surface profile analysis) and basic sediment transport analyses such as evaluation of watershed sediment yield, incipient motion analysis and scour calculations.

This analysis can be considered adequate for many locations if the problems are resolved and the relationships between different factors affecting stability are adequately explained. If not, a more complex quantitative analysis based on detailed mathematical modeling and/or physical hydraulic models should be considered. This multilevel approach is presented in FHWA HEC-20. See Chapter 8 for a more in-depth discussion of morphology.

10.4.3 Water Surface Profile Modeling

The hydraulic analysis of bridges includes the computation of the water surface profile, flow and velocity distribution. The results of the analysis are used to evaluate the adequacy of the alternative in meeting design criteria.

Water surface profiles are computed for a variety of technical uses including:

- drainage crossing analysis,
- flood hazard mitigation investigations,
- flood insurance studies, and
- evaluation of longitudinal encroachments.

Hydraulic computations performed for other agencies shall be carefully evaluated for appropriateness in bridge hydraulic analysis. The evaluation shall consider the completeness and level of detail used in the waterway model.

Water Surface Profile Determination

The water surface profile may be determined by using of mathematical one- or two- dimensional models or by physical models. Where flow is essentially two-dimensional in the horizontal plane, a one-dimension model might not provide an adequate analysis of the cross-stream water surface elevations, flow velocities, or flow distribution. A two-dimensional finite element model, FESWMS is available for the analysis of flows at bridge crossing where unusually complicated hydraulic conditions exist. For exceedingly complex sites that defy accurate or practicable mathematical modeling, physical modeling may be appropriate. The constraints on physical modeling are size, cost and time. **Use of modeling methods other than HEC-RAS shall be only with the prior approval of the ADOT Drainage Section.**

1-Dimensional Modeling

In practice, the water surface profile and velocities in a section of river are often predicted using one-dimensional methods such as the standard step method. The USACOE computer program HEC-RAS is the **recommended** method for performing step backwater water surface profile computations of bridges

10.4 General Considerations (continued)

10.4.3 Water surface profile modeling (continued)

and channels in non-prismatic channels where a one-dimensional model is acceptable. HEC-2 was often used to perform water surface profile modeling. HEC-RAS does not always replicate HEC-2 results. Therefore, prior studies performed using HEC-2 should be re-run with HEC-RAS to establish the current condition model.

2-Dimensional Modeling

Two-dimensional modeling may be necessary for many types of steady and unsteady flow problems including multiple opening bridge crossings, spur dikes, flood plain encroachments, multiple channels, and flow around islands. Two-dimensional models are more complex and require more time to set up and calibrate. They require essentially the same type of field data but at a greater density of information as a one-dimensional model. FESWMS is a finite element model that has been developed to analyze flow at bridge crossings where complicated hydraulic conditions exist.

Physical Modeling

Complex hydrodynamic situations defy accurate or practicable mathematical modeling. Physical models should be considered when:

- hydraulic performance data is needed that cannot be reliably obtained from mathematical modeling,
- risk of failure or excessive over-design is unacceptable, or
- research is needed.

The constraints on physical modeling are size (scale), cost, and time.

10.5 Design Procedures & Modeling Considerations

10.5.1 Outline of Procedure

An outline of a design procedure is presented below; this outline shall be modified as necessary to fit the individual site characteristics.

I. Data Collection

A. Survey.

1. Topography
2. Geology and Soil information
3. Highwater marks
4. History of debris accumulation and scour
5. Review of hydraulic performance of existing structures
6. Maps and aerial photographs for understanding of site and historical trends
7. Rainfall and stream gage records
8. Field reconnaissance

B. Hydrologic and hydraulic studies by other agencies.

1. Federal Flood Insurance Studies
2. Federal Flood Plain Studies by the COE, NRCS, etc.
3. Arizona Department of Water Resources and local flood plain studies
4. Hydraulic performance of existing bridges

C. Hydraulic features which influences site response.

1. Other streams, reservoirs, water intakes
2. Structures upstream or downstream
3. Natural features of stream and flood plain
4. Channel modifications upstream or downstream
5. Flood plain encroachments
6. Sediment types and bed forms

D. Environmental features which impact site.

1. Existing bed or bank instability
2. Flood plain land use and flow distribution
3. Environmentally sensitive areas (fisheries, wetlands, etc.)

E. Site-specific Design Criteria

1. Preliminary risk assessment
2. Application of design criteria

10.5 Design Procedures & Modeling Considerations (continued)

10.5.1 Outline of Procedure (continued)

II. Hydrologic Analysis

A. Watershed morphology

l. Drainage area size (attach map)
2. Watershed characterization (soil type, land use and vegetation)
3. Channel geometry and slope

B. Hydrologic computations

l. Discharge for historical flood that complements the high water marks used for calibration
2. Discharges for specified frequencies

III. Hydraulic Analysis

A. Computer model calibration and verification
B. Hydraulic performance for existing conditions
C. Hydraulic performance of proposed designs
D. Scour analysis of proposed designs

IV. Selection of Final Design

A. Cost effective alternative
B. Measure of compliance with established hydraulic criteria
C. Consideration of environmental and social criteria
D. Design details such as bank protection

V. Documentation

A. Complete project record
B. Complete correspondence and reports

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2 Water Surface Profile Analysis at Bridges

The water surface profile computations are interactive and are solved by application of the step-backwater method using a trial and error process of assuming the energy grade elevation, computing the discharge, and comparing with the specified discharge. It is impracticable to perform the hydraulic analysis for a bridge by manual calculations due to the interactive and complex nature of those computations.

The USACOE computer program HEC-RAS is the **accepted** method for performing step backwater water surface profile computations of bridges and channels in non-prismatic channels where a one-dimensional model is acceptable.

Accuracy of computed water surface profiles

Accuracy in computing water surface profiles with the step backwater profile method is affected by:

- data estimation errors resulting from incomplete or inaccurate data collection and inaccurate data estimation,
- errors in accuracy of energy loss calculations depending on the validity of the energy loss equation employed and the accuracy of the energy loss coefficients selected (Manning's n-value is the coefficient measuring boundary friction),
- inadequate length of stream reach investigated, and
- use of cross-section spacing that is too great. This results in inaccurate integration of the energy loss-distance relationship.

10.5.2.1 Classification of Flow and Computational Methods

The following discussion presents the methods and procedures used in HEC-RAS to perform water surface profiles. The user should review the information presented in the HEC-RAS Hydraulics Reference Manual, (Jan 01). The stream-crossing system is subject to either free-surface flow or pressure flow through one or more bridge openings with possible embankment overtopping. The bridge routines in HEC-RAS have the ability to model low flow (Class A, B, C), low flow and weir flow (with adjustments for submergence on the weir), pressure flow (orifice and sluice gate equations), pressure and weir flow, and highly submerged flows. In free-surface flow (low flow), there is no contact between the water surface and the low-girder (or low chord) elevation of the bridge (open channel flow). In orifice flow, only the upstream girder is submerged, while in submerged orifice flow both the upstream and downstream girders are submerged.

Combination Flow

Sometimes combinations of low flow or pressure flow occur with weir flow. In these cases the program uses an iterative procedure to determine the amount of each type of flow. The program continues to iterate until both the low flow method (or pressure flow) and the weir flow method have the same energy (within a specified tolerance) upstream of the bridge (section 3). The combination of low flow and weir flow can only be computed with the energy and Yarnell low flow method.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1 Classification of Flow and Computational Methods (continued)

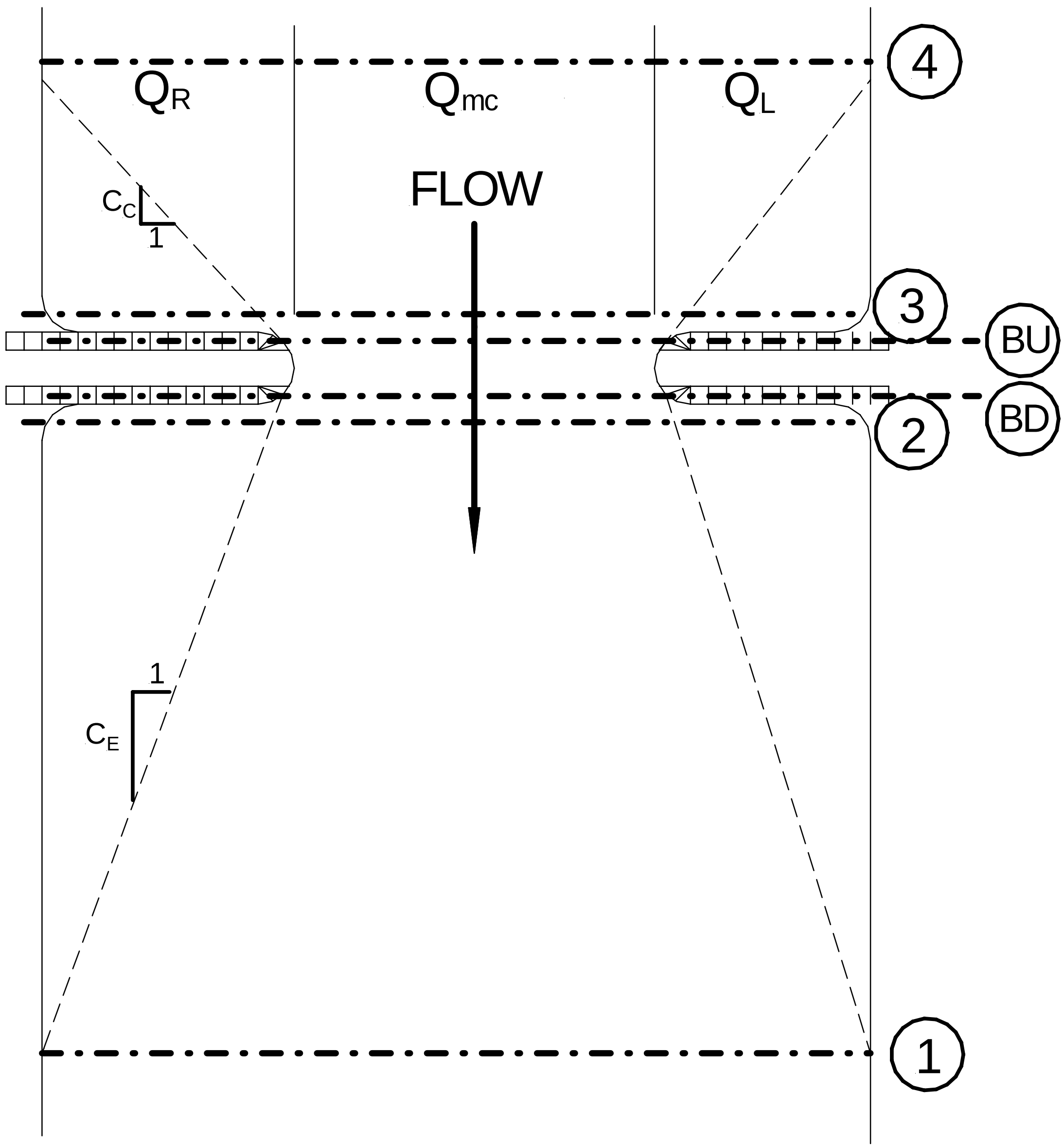


Figure 10.1a - Flow through a Bridge Opening

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1 Classification of Flow and Computational Methods (continued)

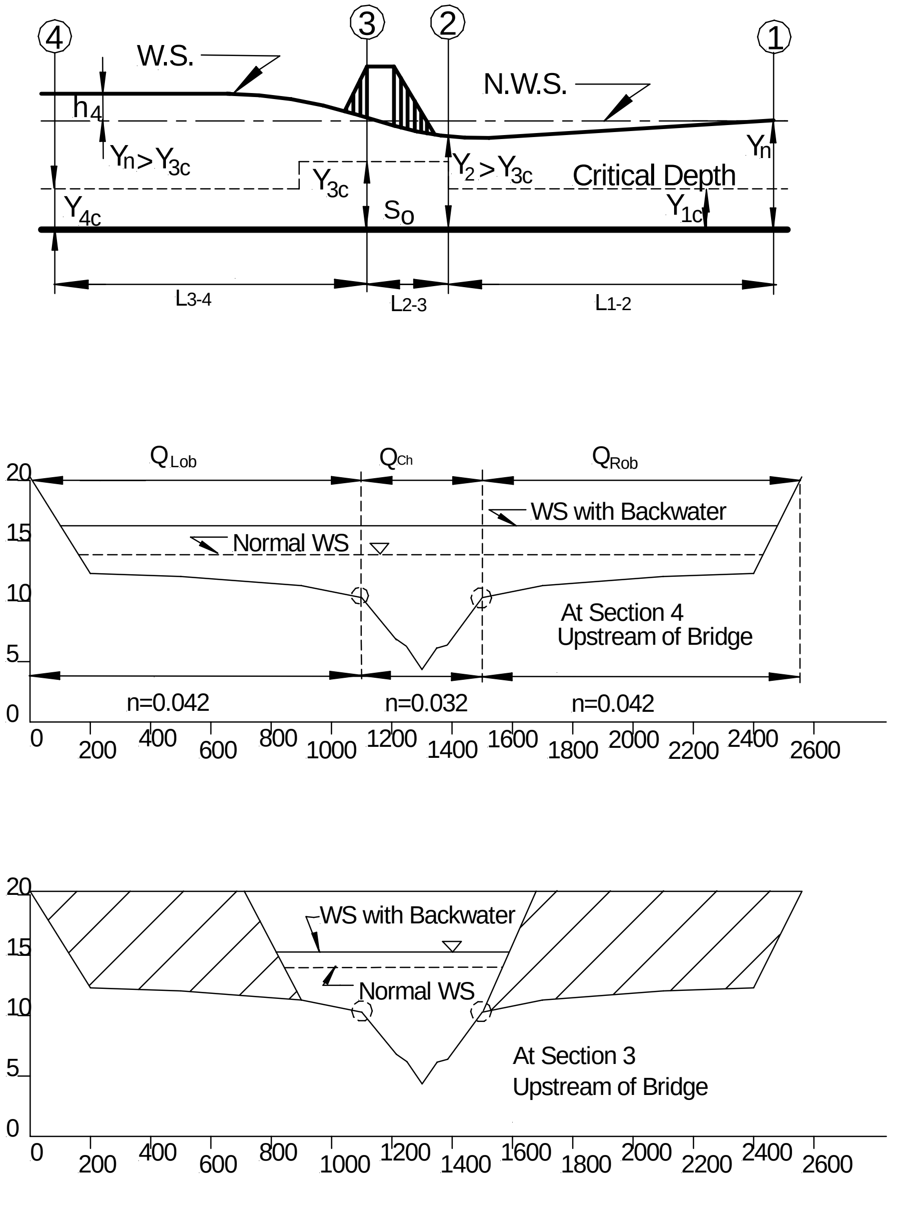


Fig 10.1b Backwater Effect of Bridge

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1 Classification of Flow and Computational Methods (continued)

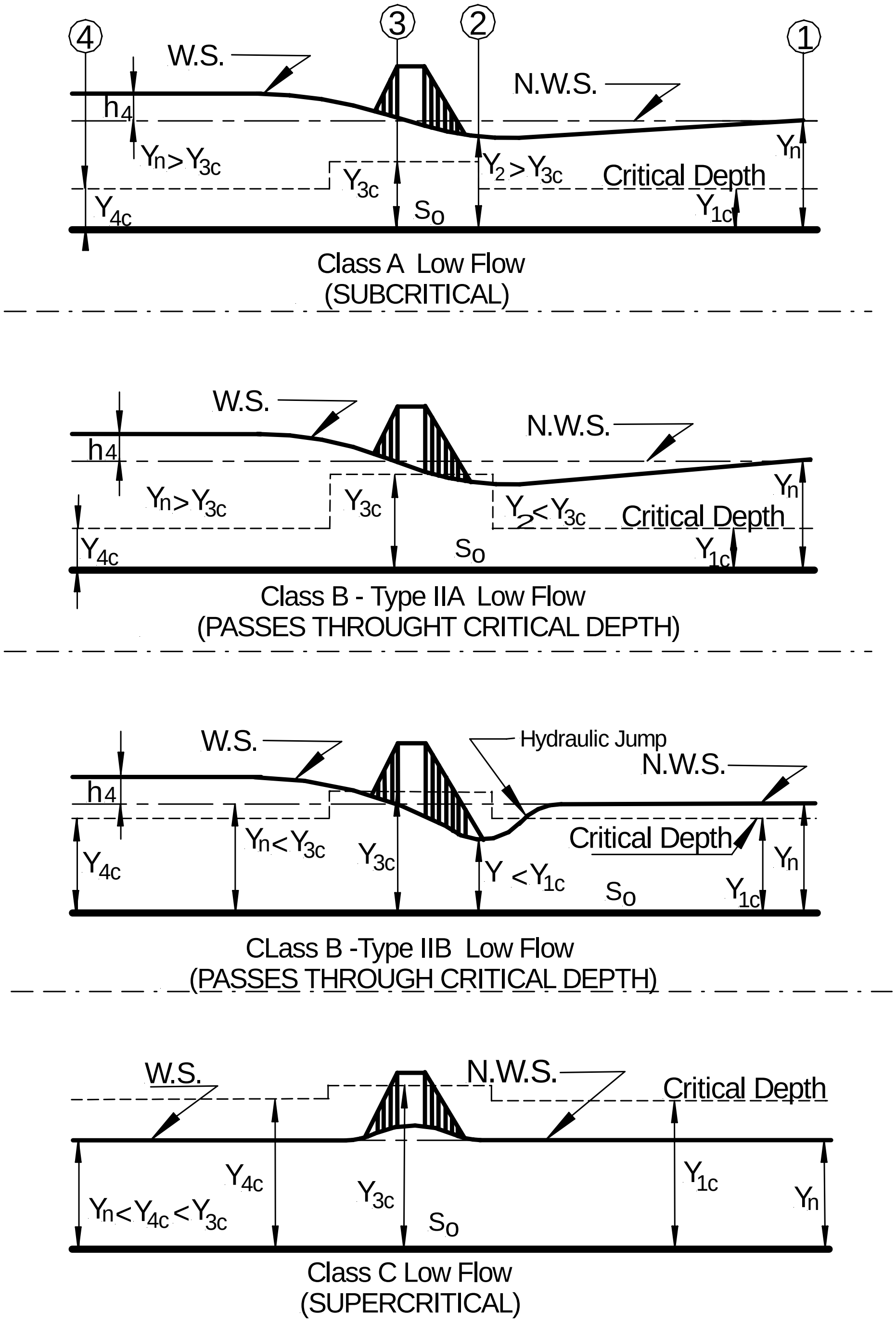


Figure 10-2 Bridge Flow Types

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1 Classification of Flow and Computational Methods (continued)

10.5.2.1.1. Modeling Cross Sections

The Bridge routines utilize four user-defined cross-sections in the computation of energy losses due to the structure. The cross sections are labeled 1 through 4 in Figure 10-1. During the hydraulic computations the program automatically formulates two additional cross sections inside the bridge (BD and BU).

Cross section 1 is located sufficiently downstream from the structure so that the flow is not affected by the structure. This is usually based on the expansion ratio chosen.

Cross section 2 is located a short distance downstream from the bridge (commonly placed at the downstream toe of the road embankment). This cross section should represent the effective flow area just downstream of the bridge. Cross section 3 is located a short distance upstream from the bridge (commonly placed at the upstream end toe of the road embankment). The distance between cross section 3 and the bridge should only reflect the length required for the abrupt acceleration and contraction of the flow that occurs in the immediate area of the opening. Cross section 3 represents the effective flow area just upstream of the bridge.

Both cross section 2 and 3 will have ineffective flow areas to either side of the bridge opening during low flow and pressure flow profiles. In order to model only the effective flow areas at these two sections, the user should use ineffective flow area options at both these cross sections.

Cross section 4 is an upstream cross section where the flow lines are approximately parallel and the cross section is fully effective.

During the hydraulic computations, the program automatically formulates two additional cross sections inside the bridge structure (BD and BU). The geometry inside the bridge is a combination of the bounding cross sections (sections 2 and 3) and the bridge geometry. The bridge geometry consists of the bridge deck and roadway, sloping abutments if necessary, and any piers that exist.

10.5.2.1.2 Low Flow Computations

Low flow exists when the flow going through the bridge opening is open channel flow (water surface below the highest point on the low chord of the bridge opening). For low flow computations, the program firsts uses the momentum equation to identify the class of flow. The program first calculates the momentum at critical depth inside the bridge at the upstream and downstream ends. The end with the higher momentum (therefore most constricted section) will be the controlling section in the bridge. If the two sections are identical, the program selects the upstream bridge section as the controlling section. The momentum at the controlling section is then compared to the momentum of the flow downstream of the bridge when performing a subcritical profile (upstream of the bridge for a supercritical profile). If the momentum downstream is greater than the critical depth momentum inside the bridge, the class of flow is considered to be completely subcritical. (Class A low flow). If the momentum downstream is less than the momentum at critical depth in the controlling section of the bridge, then it is assumed that the constriction will cause the flow to pass through critical depth and a hydraulic jump will occur at some distance downstream (Class B low flow). If the profile is completely supercritical through the bridge, then it is considered Class C low flow. See Figure 10-2.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1.2 Low Flow Computations (continued)

Class A Low Flow:
Class A low flow consists of subcritical flow throughout the approach, bridge, and exit cross sections. It is the most common condition encountered in practice. Energy losses through the expansions are calculated as friction losses and expansion losses. Friction losses are based on a weighted friction slope times a weighted reach length between section 1 and 2. The weighted friction slope is based on one of the four available alternatives in HEC-RAS, with the average conveyance method being the default. The average length used is based on a discharge-weighted reach length. Energy losses through the contraction (sections 3 and 4) are calculated as friction losses and contraction losses; these are calculated in the same way as between sections 1 and 2.

Losses through the bridge are calculated by one of four available user selected methods:

- Energy Equation (standard step method)
- Momentum Balance
- Yarnell Equation
- FHWA WSPRO method

The user can either select the method, or direct the program to use the method that computes the greatest energy loss through the bridge.

Energy Equation
The Energy Equation treats a bridge in the same manner as a natural river cross-section with the area of the bridge below the water surface is subtracted from the total area, and the wetted perimeter is increased where the water is in contact with the bridge structure. The program formulates the two cross sections inside the bridge by combining the ground information of sections 2 and 3 with the bridge geometry. The program begins with standard step calculation at the downstream section 2, then to the downstream section just inside of the bridge, BD, then to the upstream section just inside the bridge, BU, and finally to the upstream section 3. The energy-based method requires Manning’s n values for friction losses and contraction and expansion coefficients for transition losses. Detailed output is available for cross sections inside the bridge as well as the user entered cross sections (sections 2 and 3)

Momentum Balance Method
The momentum method is based on performing a momentum balance from cross section 2 to cross section 3. The momentum balance is performed in three steps. The first step is to perform a momentum balance from cross section 2 to the downstream section just inside of the bridge. The equation for this momentum balance is

$$(A*Y+Q\wedge^{2}/gA)=(A*Y+Q\wedge^{2})-A*Y+F-W$$

Area of flow times depth of flow plus Discharge squared divided by gravity times area of flow in the downstream bridge section is set equal to the same quantity in section 2, plus the external force due to friction minus the force due to weight of water in the direction of flow. The second step is a momentum balance between section BD and BU. This has the same form of equation as for the first step. The final step is a momentum balance between section BU and section 3. This step includes a term for the drag for flow going around the piers. The momentum balance requires the use of roughness coefficients of the friction

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1.2 Low Flow Computations (continued)

Class A Low Flow: (continued)
force and a drag coefficient for the force of drag on the piers. Drag coefficients are used to estimate the force due to the water moving around the piers, the separation of flow, and the resulting wake that occurs downstream. The drag coefficient for square nose piers is 2.0.

The momentum method provides detailed output for the cross sections inside the bridge (BU and BD) as well as outside the bridge (section 2 and 3). The user has the option of turning the friction and weight force components off. The default is to include the friction force but not the weight component. During the momentum calculations, if the water surface at section BD or BU comes in contact with the maximum low chord of the bridge, the momentum balance is assumed to be invalid and the results are not used.

Yarnell Equation

The Yarnell equation is an empirical equation that is used to predict changes in water surface from just downstream of the bridge (Section 2) to just upstream of the bridge (section 3). The drop in water surface from section 3 to section 2 is a function of the ratio of velocity head to depth at section 2, the obstructed area of the piers divide by the unobstructed area at section 2, and the velocity at section 2.

The computed water surface at section 3 is simply the downstream water surface plus the change calculated above. The Yarnell equation is very sensitive to pier shape, pier obstructed area, and velocity of flow. Because of these limitations, the Yarnell method should only be used at bridges where the majority of energy losses are associated with the piers. The pier coefficient for square nose pier is 1.25.

FHWA WSPRO Method

The WSPRO method computes the water surface profile through a bridge by solving the energy equations. The method is an iterative solution performed from the exit cross-section (1) to the approach cross section (4). The energy balance is performed in steps from the exit section (1) to the section just downstream of the bridge (2), from just downstream of the bridge (2) to just inside the downstream end (BD), from just inside downstream (BD) to just inside upstream (BU), from just inside upstream (BU) to just upstream (3), and from just upstream (3) to the approach section (4).

Losses from section 1 to section 2 are based on friction losses and an expansion loss. Friction losses are calculated using the geometric mean friction slope times the flow weighted distance between section 1 and 2. Losses from section 2 to section 3 are based on friction losses only. The energy balance is performed in three steps: from section 2 to section BD, from section BD to section BU, and BU to 3. Friction losses are calculated using geometric mean friction slope time the flow weighted distance between sections.

Losses from section 3 to section 4 are based on friction losses only. This is based on the effective flow length in the approach reach, computed as the average of 20 equal conveyance steam tubes.

Class B Low Flow
Class B low-flow can exist for either subcritical or supercritical profiles. For either profile, Class B flow represents the condition when "choking" of the flow by the bridge opening results in critical flow through the bridge opening. In Type IIA flow, supercritical downstream, the critical water surface elevation in the bridge opening is lower than the undisturbed normal water surface elevation. In the

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1.2 Low Flow Computations (continued)

Class B Low Flow (continued)

Type IIB flow, subcritical downstream, it is higher than the normal water surface elevation and a weak hydraulic jump immediately downstream of the bridge contraction is possible. For a subcritical profile, the momentum equation is used to compute an upstream water surface (Section 3) above critical depth and a downstream water surface (section 2) below critical depth. For a supercritical profile, the bridge is acting as a control and is as causing the upstream water surface elevation to be above critical depth. Momentum is used to calculate an upstream water surface above critical depth and a downstream water surface below critical depth. If the momentum equation fails to converge on an answer during Class B flow computations, the program will automatically switch to an energy-based method.

Whenever Class B flow is found to exist, the user should run the program in a mixed flow regime mode. If the mixed flow regime is run, the program will proceed with backwater calculations upstream, and later with forewater calculations downstream from the bridge. Any hydraulic jumps that occur either upstream or downstream of the bridge can be located.

Class C Low Flow

Class C flow is supercritical approach flow and remains supercritical through the bridge contraction. The program can use either the energy equation or the momentum equation to compute the water surface through the bridge. Such a flow condition is not subject to backwater unless it chokes and forces the occurrence of a hydraulic jump upstream of the contraction.

10.5.2.1.3 High Flow Computations

The program computes high flow conditions by either the Energy equation (standard step method) or by using separate equations for pressure and/or weir flow.

Energy Equation

The energy-based method is applied to high flows in the same manner as it is applied to low flows. Computations are based on balancing the energy equation in three steps through the bridge. Energy losses are based on friction and contraction and expansion losses. Friction losses are based on use of Manning's equation. Expansion and contraction losses are based on a coefficient times the change in velocity head. The energy-based method performs all computations as they are open channel flow. At the cross sections inside the bridge, the area obstructed by the bridge piers, abutments, and deck is subtracted from the flow area and additional wetted perimeter is added. Occasionally the resulting water surfaces inside the bridge sections (BD and BU) are computed to be at elevations inside the bridge deck. The water surface inside the bridge reflects the hydraulic grade line elevations, not necessarily the actual water surface elevations. The active flow area is limited to the open bridge area.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.1.3 High Flow Computations (continued)

Pressure Flow Computations

Pressure flow occurs when the flow comes into contact with the low chord of the bridge. Once the flow comes in contact with the upstream side of the bridge, a backwater occurs and orifice flow is established. The program will handle two cases of orifice flow; when only the upstream side of the bridge is in contact with the water, and when the bridge opening is completely full. The program automatically selects the appropriate equations. For the first case, a sluice gate type of equation with a discharge coefficient is used. The program will select the discharge coefficient based on the amount the inlet is submerged.

Free surface flow is assumed to occur until the depth just upstream of the bridge opening exceeds 1.1 times the hydraulic depth of the opening, then pressure flow is assumed. In the second case, when both the upstream and downstream side of the bridge are submerged, the full flowing orifice equation is used. Pressure flow is calculated as orifice flow with the discharge proportional to the square root of the effective head. Submerged orifice flow is treated similarly with the head redefined. HEC-RAS can also simultaneously consider embankment overflow as a weir discharge. The program will begin checking for the possibility of pressure flow when the computed low flow energy grade line is above the maximum low chord elevation at the upstream side of the bridge. Once pressure flow is computed, the pressure flow answer is compared to the low flow answer; the higher of the two is used. The user has the option to tell the program to use the water surface instead of energy to trigger the pressure flow calculation.

Weir Flow Computations

Flow over the bridge, and the roadway approaching the bridge is calculated using the standard weir equation.

$$Q=CLH^{1.5} \tag{10.1}$$

The approach velocity is included by using the energy grade line elevation in lieu of the upstream water surface elevation for computing the head, H. The coefficient C, is based on broad crested weir for either free flow conditions (discharge independent of tailwater). For rectangular weir flow over the bridge deck, a coefficient of 2.6 is reasonable. If the weir flow is over the roadway approach, a coefficient of 3.0 is reasonable. If weir flow occurs as a combination of bridge and roadway overflow, then an average coefficient (weighted by weir length) could be used.

For high tailwater elevations, the program will automatically reduce the amount of weir flow to account for submergence on the weir. Submergence is defined as the depth of water above the minimum weir elevation on the downstream side divided by the height of the energy grade line above the minimum weir elevation on the upstream side. The submergence must exceed 75% for any reduction to take place. The reduction of weir flow is accomplished by reducing the weir coefficient based on the amount of submergence. The total weir flow is computed by subdividing the weir crest into segments, computing L, H, a submergence correction, and a Q for each section, them summing the incremental discharges.

When the weir becomes highly submerged the program will automatically switch to calculating the upstream water surface by the energy equation (standard step backwater) instead of using the pressure and weir flow equations. The default submergence is 0.95.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.2 HEC-RAS Modeling

10.5.2.2.1 General Considerations

HEC-RAS uses some methodology that is different than HEC-2. Studies performed using HEC-2 will result in slightly different water surface profile elevations when recalculated using HEC-RAS. For such sites, the first run should be for the HEC-2 data to establish the base condition prior to making changes that reflect the proposed project.

Several models may need to be developed demonstrating the sequence of events that impact the site. A suggested order is:

A. Existing condition
B. Embankment Encroachment
C. Structure effects

In order to understand the effects of the stream crossing system on the hydraulics of the site, the following discussion is presented. A user's instruction manual for HEC-RAS is available and should be used for information on using the computer model. Two specific examples are given in Appendix B. Only sufficient information to understand the examples is given.

Starting Water Surface

For subcritical flow, it is normal practice to use Manning’s equation to compute normal depth as the starting water surface. The actual water surface may be higher or lower than normal depth. The use of normal depth at the boundary will introduce an error. In general the error at the boundary will diminish as the computations proceed upstream.

The water surface profile used in a subcritical hydraulic analysis should begin far enough downstream, if there is no obvious hydraulic control section, to ensure that the computed water surface elevation has converged to a consistent answer by the downstream limits of the study reach, i.e. reached the “true” depth downstream of the influence of the bridge constriction. The water surface profile shall extend upstream beyond the extent of the bridge backwater caused by the bridge (increase in water surface profile resulting from a channel modification converges with the existing conditions profile).

Conversely, in modeling supercritical flow, the analysis should begin far enough upstream to insure that the water surface elevation has reached the normal depth stage upstream of the influence of the bridge constriction and extends downstream far enough to be beyond the influence of the bridge.

Length of reach

The distances between cross sections are referred to as reach lengths. Cross-sections should be situated such that they represent the channel conditions, and provide for computational stability. Channel reach lengths are typically specified along the thalweg. Overbank reach lengths should be measured along the anticipated path of the center of mass of the overbank flow. In steep channels with shallow flow they should be situated sufficiently close to preclude that the fall exceeds the flow depth. A guide for the spacing of cross-sections is the distance between cross-sections does not exceed 10% of the Average Depth/ Streambed Slope.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.2.1 General Considerations (continued)

Cross-sections for Modeling

The cross sections that are desired for the energy analysis through the bridge opening for a single opening bridge without spur dikes are shown in Figure 10-1. The additional cross sections that are necessary for computing the entire stream water surface profile are not shown in this figure. Cross sections 1, 2,3, and 4 are required to define the flow approaching and departing the roadway embankment. In addition, sections which define the bridge are needed for the energy loss computations of the bridge structure. The bridge and attendant embankment encroachment should be modeled using 4 sections. There should be an entrance(3) and exit(2) section that describes the embankment encroachment using the geometry of the embankment and bank protections, and bridge sections as appropriate for the upstream(BU) and downstream(BD) faces of the bridge.

Ineffective Flow Modeling

The cross-sections may need to be modified to identify cross section areas that contain water that is not actively conveyed (ineffective flow). The program has the capability to modify cross sections so as to simulate sediment deposition, confine flows to leveed channels, block out road fills and bridge decks, and floodplain encroachments. Ineffective flow areas can be defined using a.) ineffective flow , b.) levees, or c.) blocked areas. As encroachments are changed at a section, other sections must be reviewed for the specification of effective flow areas. Modeled ineffective flow areas may need to be changed with changes in discharge.

Ineffective flow areas are used to describe portions of a cross section in which water will pond, but the velocity of that water, in the downstream direction, is close to zero. The water is included in the storage calculations and other wetted cross section parameters but is not included as part of the active flow area. When using ineffective flow areas, no additional wetted perimeter is added to the active flow area. Ineffective flow areas are modeled by use of levees, or blocked flow. For leveed flow, once the water surface goes above the established elevation, then that specific area is no longer considered ineffective. For blocked flow, the water will not flow within the limits that describe the block, however, a blocked obstruction does not prevent water from going outside of the obstruction. Up to 20 blocks may be entered.

Ineffective Flow at an Encroaching Embankment

Ineffective flow areas are used for the channel sections adjacent to a roadway embankment where there are ponded areas outside the contraction or expansion limits. Use of ineffective flow will allow for considering this area as effective when the roadway is overtopped. The ineffective flow areas should be set at stations that will adequately describe the active flow area at cross section 2 and 3. In general, these stations should be placed outside the edges of the bridge opening to allow for the contraction and expansion of flow that occurs in the immediate vicinity of the bridge. On the upstream side, Section 3, one may use a contraction ratio of 1:1. If section 3 is 10 feet upstream, begin the limits of ineffective flow 10 feet wider than the bridge opening on each side. On the downstream side, Section 2, a similar assumption may be used with an expansion ratio in the range of $0.5 < ER < 4$. See Appendix 10-A for guidance on selecting an expansion ratio.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.2.1 General Considerations (continued)

The elevations specified for ineffective flow should correspond to elevations where significant weir flow passes over the roadway. This may need to be the elevation at the top of the guard/bridge rail as they are usually blocked with debris. For the downstream cross section, the threshold water surface elevation for weir flow is not usually known for the initial run, it may be estimated using the average elevation between the low chord and the minimum top of road.

The user should check to ensure that the computer solutions are consistent with the type of flow described. For low flow or pressure flow, the output should show the effective areas restricted to the bridge opening. When the output indicates weir flow, the solution should show the entire cross section is effective. The overbank flow around the bridge should be consistent with weir flow.

Levees

Cross sections with low overbank areas that are not part of the main channel require special consideration in computing water surface profiles. Normally the computations are based on the assumption that all area below the water surface elevation is effective in passing the discharge. However, if the water surface elevation at a particular cross section is less than the top of "levee" elevations, and if the water cannot enter or leave the overbanks upstream of that cross section, then the flow areas in these overbanks should not be used in the computations. These areas should only be considered when the flow in the channel is higher than the "levee" elevations. The user excludes these areas by using levees at locations where the water is contained in the main channel until the levee elevation is exceeded. The program includes additional wetted perimeter when water comes in contact with the levee wall. If the water surface elevation is close to the top of a levee, the program may experience some difficulty in balancing the water surface elevations due to changing assumptions of flow area when just above or below the levee top. The designer must review the appropriateness of the assumed water surface elevations and revise the model as necessary. Also, assumptions regarding effective flow areas may change with changes in flow magnitude. Where cross section elevations outside the levee are considerably lower than the channel bottom, this may require adjustment of the cross section to be sure that effective flow areas are properly described.

It is important for the user to study carefully the flow pattern of the river where levees exist. If, for example, a levee were open at both ends and flow passed behind the levee without overtopping it, it should be modeled as a berm rather than described as a "levee".

Blocked Areas

Areas of the cross section from which flow is permanently obstructed are described using blocked area. The areas may be defined using total blocked areas for each side or up to 20 multiple blocks. In either case the blockage is described using stations and elevations of the blocked area.

Energy Losses

Energy losses in HEC-RAS are normally computed using standard step procedures. Friction losses are evaluated using Manning's n values. The accuracy of the computed surface profiles is significantly affected by the appropriateness of the chosen Manning's n. See Chapter 8 for discussion on selection of Manning's n.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.2.1 General Considerations (continued)

Expansion and Contraction

All models must have the appropriate expansion and contraction encroachment applied at each step. Expansion and contractions are handled by using taper ratios to limit effective flow areas and coefficients. The coefficients are multiplied by the absolute difference in velocity heads between the current cross section and the next cross section downstream. Where the change in river cross section is small, and the flow is subcritical, contraction and expansion coefficients of 0.1 and 0.3 are often used.

Table 3.3 Subcritical Flow Contraction and Expansion Coefficients

	Contraction	Expansion
No transition loss computed	0.0	0.0
Gradual transitions	0.1	0.3
Typical Bridge Sections	0.3	0.5
Abrupt Transitions	0.6	0.8

Flow Distribution Calculations

The program computes the flow distribution based on three flow subdivisions, (left overbank, main channel, and right overbank), and balancing the energy equation. The user can request additional output showing the distribution of flow for multiple subdivisions on each of these areas. The user can select to have this information for specific cross-sections or all cross-sections. Up to 45 slices can be specified. Each flow element (left overbank, main channel, right overbank) must have at least one slice. Where a flow distribution is requested, the program will calculate the flow (discharge), area, wetted perimeter, percentage of conveyance, hydraulic depth, and average velocity for each slice.

10.5 Design Procedures & Modeling Considerations (continued)

10.5.2.2. HEC-RAS Modeling (continued)

10.5.2.2.2 Bridge Modeling Approach

Low Flow Methods
For low flow conditions (water surface below the highest point on the low chord of the bridge opening) the Energy and Momentum methods are the most physically based and in general are applicable to the widest range of bridges and flow situations. Both methods account for friction losses and changes in geometry through the bridge. The energy method accounts for additional losses due to flow transitions and turbulence through the use of contraction and expansion losses. The momentum method can account for additional losses due to pier drag.

1. In cases where the bridge piers are a small obstruction to the flow, and fiction losses are the predominate considerations the energy based method or the momentum method should be used.

2. In cases where pier losses and friction losses are predominant, the momentum method is most applicable.

3. Whenever the flow passes through critical depth within the vicinity of the bridge, both the momentum and energy methods are capable of modeling this type of flow transition.

4. For supercritical flow both the energy and the momentum method can be used. The momentum-based method may be better at locations that have a substantial amount of pier impact and drag losses.

5. For bridges in which the piers are the dominant contributor to energy losses and the change in water surface, the momentum method is applicable.

6. For long culverts under low flow conditions, the energy based standard step method is the most suitable approach. However, if the culvert flows full or in inlet control conditions, the culvert routines are the best approach.

High Flow Methods
For high flow (flows that come in contact with the maximum low chord of the bridge deck) the energy based method is applicable to the widest range of conditions.

1. When the bridge deck is a small obstruction to the flow, and the bridge opening is not acting like a pressurized orifice, the energy method should be used.

2. When the bridge deck and road embankment are a large obstruction to the flow, and a backwater created due to the construction of the flow, the pressure and weir method should be used.

3. When the bridge and/or road embankment is overtopped, and the water going over the top of the bridge is not highly submerged by the downstream tailwater, the pressure and weir method should be used. The pressure and weir method will automatically switch to the energy method if the bridge 10.5 Design Procedures & Modeling Considerations (continued)

10.5.2 Water Surface Profile Analysis at Bridges (continued)

10.5.2.2.2 Bridge Modeling Approach (continued)

becomes 95 percent submerged. The user can change the percent submergence at witch the program switches from the pressure and weir method to the energy method.

4. When the bridge is highly submerged, and flow over the road is not acting like weir flow, the energy based method should be used.

5. When describing the bridge superstructure for overtopping, include the guardrail and bridge rail.

Unique Challenges and Suggested Approaches

Perched Bridge

A perched bridge is one for which the road approaching the bridge is at the floodplain ground level and in the immediate vicinity of the bridge the road rises above the ground level to span the watercourse. A typical low-flow situation is low flow under the bridge and overbank flow around the bridge. If the road approaching the bridge is not much higher than the surrounding ground, the assumption of weir flow is not justified. A solution based on the energy method is a better solution, especially when a large percentage of the total discharge is in the overbank areas.

High Submergence Bridge (Low flow bridge)

A low flow bridge is designed to carry only low flows under the bridge. Flood flows are carried over the bridge and road. When modeling this condition for flood flows, the solution may be by either a combination of pressure and weir flow method or energy flow method. If the tailwater is high, it may be better to use the energy-based method.

Bridges on a Skew

Skewed bridge crossings are generally handled by making adjustments to the bridge dimensions to define an equivalent cross section perpendicular to the flow lines. Skewed crossing with angles up to 20 degrees show no objectionable flow patterns.

Multiple Openings

Multiple openings of either bridge and/or culvert can be modeled using HEC-RAS. One should consider using flow distribution output to review the hydraulic modeling.

Multiple/Parallel Bridges

The hydraulic loss through multiple bridges is between one to two times for one bridge. If the bridges are far enough apart, the loss for the multiple bridges is equal to sum of the losses for each bridge. If the bridges are very close together and the flow is not able to expand between the bridges, the contraction should occur only at the upstream bridge with only pier and friction losses at the downstream bridges, the bridges can be modeled as a single wide bridge. If there is sufficient distance between the bridges in which the flow has room to expand and contract, the bridges should be modeled as separate bridges. If separate bridges are modeled, the expansion and contraction rates should be based on the same procedure as for a single bridge.

10.6 Bridge Scour

10.6.1 Introduction--Philosophy

Reasonable and prudent hydraulic analysis of a bridge design requires that an assessment be made of the proposed bridge's vulnerability to undermining due to potential scour.

It is ADOT's Goal that the bridge does not fail during its lifetime due to scour for flow events up-to and including the 500-year event. In this context, lifetime means the period of time that the bridge is in use. It is not limited to the economic life or the design life of the highway.

10.6.2 Scour Types

Scour is the manifestation of the sediment transport process as affected by the presence of objects that change or disturb the approach flow conditions. The object that causes disturbance can be a bend in the bank, an encroaching embankment or a pier. Any of these objects causes a change in the sediment transport capacity of the flow, and a corresponding scour.

Bridge scour is evaluated as interrelated components:

- long term profile changes (aggradation/degradation),
- plan form change (lateral channel movement, bank widening),
- contraction scour/deposition, and
- local scour.

Presented are the generally recommended procedures for use in most situations. In line with the goal presented above, the designer is encouraged to apply all the information available and make judgments that **ensure the bridge will NOT fail**.

Long Term Profile Changes

Aggradation or degradation long-term profile changes can result from streambed gradient or sediment transport changes.

- Aggradation is the deposition of bedload due to a decrease in the local stream sediment transport capacity. This may be due to a decrease in the local energy gradient.

- Degradation is the scouring of bed material due to an increase in the local stream sediment transport capacity. This may result from an increase in the energy gradient or from removal of sediment at an upstream section due to gravel mining or a reservoir.

10.6 Bridge Scour (continued)

Long Term Profile Changes (continued)

Where gravel mining is an on-going or to be expected activity, the impacts of the change in the sediment transported may result in long-term profile changes.

- Gravel mining can cause an increase in the energy gradient at the upstream end of the operation that may result in degradation of the streambed profile.

- Gravel mining can reduce the sediment transported to downstream reaches that may result in degradation of the streambed profile.

Plan Form Changes

Plan form changes are morphological changes such as meander migration or bank widening. The lateral movement of meanders can threaten bridge approaches as well as increase scour by changing flow patterns approaching a bridge opening. Bank widening can cause significant changes in the flow distribution and thus the bridge's flow contraction ratio.

Constriction & Expansion

Contraction scour results from a constriction of the flow area that may, in part, be caused by bridge piers in the waterway in addition to the encroaching embankment/abutment. Deposition results from an expansion of the channel or the bridge site being positioned immediately at the beginning of a flatter reach of stream.

Highways, bridges, and natural channel constrictions are the most commonly encountered cause of contraction scour. The scour is considered as either live-bed or clear water contraction scour. Live-bed scour occurs when bed material is already being transported into the contracted opening from upstream of the approach section. Clear water contraction scour occurs when the bed material transport in the uncontracted approach section is negligible or less than the carrying capacity of the flow. The two most common occurrences of contraction scour for bridges in Arizona are

Case I. Overbank flow on a floodplain being forced back into the main channel. Case I occurrences include the following conditions:

a. The river channel width becomes narrower either due to the bridge abutments projecting into the channel or the bridge being located at a narrowing reach of the river.

b. Overbank flow area is obstructed either partially or completely by the road embankment/abutment with no constriction of the main channel

Case II. Flow is confined to the main channel (i.e. there is no overbank flow). The normal river channel becomes narrower due to the bridge itself or the bridge site is located at a narrowing reach of the river.

10.6 Bridge Scour (continued)

10.6.2 Scour Types (continued)

Local Scour

Exacerbating the potential scour hazard at a bridge site are any abutments or piers located within the flood flow prism. The amount of potential scour caused by these features is termed local scour. Local scour is a function of the geometry of these features as they relate to the flow geometry.

10.6.3 Armoring & Scour Resistant Materials

Armoring

Armoring occurs because a stream or river is unable, during a particular flood event, to move the more coarse material comprising either the bed or, if some bed scour occurs, its underlying material. A review of the armored material may reveal well-rounded material that has been transported; not a coarse resident bed material. Scour may occur initially but later become arrested by armoring before the full scour potential is reached again for a given flood magnitude. When armoring does occur, the coarser bed material will tend to remain in place or quickly redeposit so as to form a layer of riprap-like armor on the streambed or in the scour holes and thus limit further scour for a particular discharge. When a larger flood occurs than the flood that created the armoring, scour will probably penetrate deeper until armoring again occurs at some deeper threshold.

Armoring may result in the stream being unable to satisfy it desired sediment transport capacity, this may cause bank widening. Bank widening encourages rivers or streams to seek a more unstable, braided regime. Such instabilities may pose serious problems for bridges as they encourage further, difficult to assess plan form changes. Bank widening also spreads the approach flow distribution that in turn results in a more severe bridge opening contraction.

Scour Resistant Materials

Caution is necessary in determining the scour resistance of bed materials and the underlying strata. Serious scour has been observed to occur in materials commonly perceived to be scour resistant such as consolidated soils and glacial till, as well as so-called bedrock streams and streams with gravel and boulder beds. With sand size material, the passage of a single flood may result in the predicted scour depths. Conversely, in scour resistant material the maximum predicted depth of scour might not be realized during the passage of a particular flood; however, some scour resistant material may be lost. Commonly, the stream will replace this material with transported material that is more easily scoured. Thus, at some later date another flood may reach the predicted scour depth.

10.6.4 Pressure Flow Scour

Pressure flow, which is also denoted as orifice flow, results from a pile up of water on the upstream bridge face, and a plunging of the flow downward and under the bridge. This occurs when the water surface elevation at the upstream face of the bridge is greater than or equal to the low chord of the bridge superstructure. At higher approach flow depths, the bridge can be entirely submerged with the resulting flow being a complex combination of the plunging flow under the bridge and the flow over the bridge.

With pressure flow, the local scour depths at a pier or abutment are larger than for free surface flow with similar depths and approach velocities. The increase in local scour at a pier subject to pressure flow results from the flow being directed downwards toward the bed by the superstructure and by increasing the

10.6 Bridge Scour (continued)

10.6.4 Pressure Flow Scour (continued)

intensity of the horseshoe vortex. The vertical contraction of the flow is a more significant cause of the increase in scour depth. However, in many cases, when a bridge becomes submerged, the average velocity under it is reduced due to a combination of additional backwater caused by the bridge super-structure impeding the flow and a reduction of discharge which must pass under the bridge due to weir flow over the bridge and approach embankments. As a consequence of this, increases in local scour attributed to pressure flow scour at a particular site, may be offset to a degree by lesser velocities through the bridge opening due to increased backwater and a reduction in discharge under the bridge due to over-topping.

HEC-RAS can be used to determine the discharge through the bridge and the velocity of approach and depth upstream of the piers when flow impacts the bridge superstructure. These values should be used to calculate local pier scour. Engineering judgment will then be exercised to determine the appropriate multiplier times the calculated pier scour depth for the pressure flow scour depth. This multiplier ranges from 1.0 for a low approach Froude numbers ($Fr = 0.1$) to 1.6 for high approach Froude numbers ($Fr = 0.6$). If the bridge is over- topped, the depth to be used in the pier scour equations and for computing the Froude number is the depth to the top of the bridge deck or guardrail obstructing the flow.

10.6.5 Scour Prediction Methodology

Bridge scour assessment shall be accomplished by collecting the data and applying the general procedure outlined in this section.

10.6.5.1 Site Data

Geometry

Obtain existing stream and flood plain cross sections and profile, site plan and the stream's present, and where possible, historic geomorphic plan and profile form. Also, locate the bridge site with respect to such things as other bridges in the area, tributaries to the stream or close to the site, bedrock controls, manmade controls (dams, old check structures, river training works, etc.), and downstream confluence with other streams. Locate (distance and height) any "headcuts" due to natural causes or activities such as gravel mining operations. Data related to plan form changes such as meander migration and the rate at which they may be occurring are useful.

When gravel mining is an on-going activity or should be expected, the impacts of gravel mining on the stream shall be evaluated. Upstream gravel mining operations may “capture” the bed material discharge resulting in the more adverse clear water scour case discussed later. Current practice is to include an allowance for future degradation at the bridge site where extensive mining is occurring, such as the Salt River in Phoenix or the Santa Cruz River in Tucson. For isolated gravel mining, an estimate of the degradation depth may be made using the procedures in “Gravel Mining Guidelines” reference “Effects of In-stream Mining on Channel Stability”

10.6 Bridge Scour (continued)

10.6.5.1 Site Data (continued)

Bed Material

The bed material should be observed. Look for evidence of bedrock outcrops, grain size for determination of bed forms and type of scour, and other indicators of the morphology of the stream. It is ADOT practice to consider all material to be scour susceptible unless proven otherwise. Grain size is not considered for the purpose of decreasing the predicted scour.

Geomorphology

Classify the geomorphology of the site; i.e., such things as whether it is a flood plain stream or crosses an alluvial fan; youthful, mature or old age, presence of headcuts, and meanders.

Historic Scour

Review available information such as as-builts, bridge inspection reports, old contour mapping, and aerial photographs to evaluate scour data on other bridges or similar facilities along the stream.

Debris

A build up of debris on the pier shall be considered. In the absence of additional site information, the **debris shall be assumed to extend 2 feet on each side of the pier and have a depth of 12 feet from the water surface**.

Hydrology

Identify the character of the stream hydrology; i.e., perennial, ephemeral, intermittent as well as whether it is "flashy" or subject to broad hydrograph peaks resulting from gradual flow increases such as occur with general thunderstorms or snowmelt and dam releases. The operational design frequency flood, the superflood and the greatest discharge passing through the structure, if less than the superflood, will be required.

10.6.5.2 Scour Prediction Practice

It is ADOT's practice to calculate the scour components as if they develop independently. Thus, the potential local scour is added to the contraction scour without considering the effects of contraction scour on the channel and bridge hydraulics. The general approach with this method is as follows. No reduction in the predicted scour is considered for armoring.

- The ADOT bridge manual specifies different load cases for analysis of the bridge capacity. These load cases must be considered in calculating the predicted scour.
- Estimate the natural channel's hydraulics for a fixed bed condition based on existing conditions.
- Assess the expected profile and plan form changes.

10.6 Bridge Scour (continued)

10.6.5. 2 Scour Prediction Practice (continued)

- Adjust the fixed bed hydraulics to reflect any expected profile or plan form changes.
- Estimate contraction scour using the empirical contraction formula.
- Estimate local scour using the channel and bridge hydraulics assuming no bed armoring. Abutment scour is estimated using Froelich's method as demonstrated in Appendix D. Pier scour is estimated using the CSU equations as demonstrated in Appendix E.
- Add the contraction scour to the local pier scour to obtain the total scour. The Abutment scour predicted includes the Contraction scour.
- The resultant predicted scour needs to be discussed with the geotechnical and bridge designers to confirm the assumptions made and to verify the erodibility of the bed material. As the goal is for the bridge not to fail; that the bed material is non-erodible is the case that must be proven.

10.6.5.3 General Procedure

Step 1

Determine the magnitude of the design flood and the superflood as well as the magnitude of the incipient overtopping flood, or relief-opening flood. Accomplish steps 2 through 10 using the discharges computed.

Review the site to identify the discharge that places the greatest stress on the bed material in the bridge opening. Where there are relief structures on the flood plain or overtopping occurs, some flood other than the base flood or "super flood" may cause the worse case bridge opening scour. This situation occurs where the bridge opening will pass the greatest discharge just prior to incurring a discharge relief from overtopping or a flood plain relief opening. Conversely care must be exercised in that a discharge relief at the bridge due to overtopping or relief openings might not result in reduction in the bridge opening discharge. Should a reduction in the bridge opening discharge occur, the incipient overtopping flood or the overtopping flood corresponding to the base flood or "super flood" is to be used to evaluate the bridge scour.

Step 2

Assess the bridge crossing reach of the stream for <u>profile bed</u> scour changes to be expected from degradation or aggradation. Take into account past, present and future conditions of the stream and watershed in order to forecast what the elevation of the bed might be in the future. Certain plan form changes such as migrating meanders causing channel cutoffs would be important in assessing future streambed profile elevations. The possibility of downstream mining operations inducing "headcuts" shall be considered. The quickest way to assess streambed elevation changes due to "headcuts" (degradation) is by obtaining a vertical measurement of the downstream "headcut(s)" and projecting that measurement(s) to the bridge site using the existing stream slopes if it is acceptable to assume the stream is in regime conditions; if it is not, then it may be necessary to estimate the regime slope. A more time consuming way to assess elevation changes would be to use a sediment routing practice in conjunction with a synthetic flood history.

10.6 Bridge Scour (continued)

10.6.5.3 General Procedure (continued)

Step 3

Assess the bridge crossing reach of the stream for plan form scour changes. Attempt to forecast whether an encroaching meander will cause future problems within the expected life of the bridge. Take into account past, present and expected future conditions of the stream and watershed in order to forecast how such meanders might influence the approach flow direction in the future. The sediment routing practice discussed later for computing channel contraction scour or aggradation may prove useful in making such assessments. This forensic analysis on a site's past geomorphological history may prove useful to forecast the future. Otherwise this assessment has to be largely subjective in nature.

Step 4

Develop a water surface profile for the discharges to be considered through the site's reach for fixed bed conditions using HEC-RAS.

Step 5

Assess the magnitude of contraction scour based on the fixed bed hydraulics.

Step 6

Assess the magnitude of local scour at abutments and piers. See section 10.6.5.4 and 10.6.5.5.

Step 7

For each discharge under consideration, plot the scour and aggradation depths from foregoing steps on a cross section of the stream channel and flood plain at the bridge site. Using judgment, enlarge any overlapping scour holes (discussed later).

Step 8

Discuss the predicted condition with the bridge and geotechnical design staff.

10.6.5.4 Contraction Scour

Contraction scour is an application of the principle of conservation of sediment transport. Live-bed contraction scour occurs at a bridge when there is transport of bed material in the upstream reach in to the bridge cross-section. In live-bed scour, the area of the contracted section increases until the fully developed scour in the bridge cross-section reaches equilibrium when sediment transported into the contracted section equals sediment transported out. As scour develops, the shear stress in the contracted section decreases as a result of a larger flow area and decreasing average velocity. For live-bed scour, maximum scour occurs when the shear stress reduces to the point that sediment transported in equals the bed sediment transported out and the conditions for sediment continuity are in balance.

10.6 Bridge Scour (continued)

10.6.5.4 Contraction Scour (continued)

Clear water contraction scour occurs when (1) there is no bed material transport from the upstream reach into the downstream reach, or (2) the material being transported in the upstream reach is transported through the downstream reach mostly in suspension and at less than the capacity of the flow. For clear-water scour, the transport into the contracted section is essentially zero and maximum scour occurs when the shear stress reduces to the critical shear stress of the bed material in the section. With clear-water contraction scour the area of the contracted section increases until, in the limit, the velocity of the flow (v) or the shear stress (t_c) on the bed is equal to the critical velocity (v_c) or the critical shear stress (t_c) for the representative particle size D, in the bed material.

There are four cases of contraction scour at bridge sites depending on the type of contraction, and whether there is overbank flow or relief bridges. The two most common occurrences of contraction scour for bridges in Arizona are presented on page 10-31. For any condition, it is only necessary to determine if the flow is transporting bed material (live-bed) or is not (clear water), and then apply the equation appropriate for the case with the variables defined according to the location of contraction scour (channel or overbank).

To determine if the flow upstream of the bridge is transporting bed material, calculate the critical velocity for beginning of motion V_c of the D50 size of the bed material being considered for movement and compare with the mean velocity of the flow in the main channel or the overbank area upstream of the bridge opening. If the critical velocity of the bed material is larger than the mean velocity ($V_c > V_0$), then clear water scour will exist. If the critical velocity is less than the mean velocity ($V_c < V_0$), then live-bed contraction scour will exist. The equation for the critical velocity is

$$V_c = 11.17\, y^{1/6} D_{50}^{1/3} \qquad \textbf{(10.2)}$$

Where:

V_c = Critical velocity above which bed material of size D and smaller will be transported, ft/sec
$y^{1/6}$= Average depth of flow upstream of the bridge, ft
$D_{50}^{1/3}$= Particle size in a mixture of which 50 percent are smaller, ft.

The D_{50} is taken as an average of the bed material size in the reach of the stream upstream of the bridge. It is a characteristic size of the material that will be transported by the stream. Normally this would be the bed material in the surface 1 foot of the streambed.

10.6 Bridge Scour (continued)

10.6.5 Scour Prediction methods (continued)

10.6.5.4 Contraction Scour (continued)

Laursen (1980) in **Predicting Scour at Bridge Piers and Abutments** made the following comments:

- Scour is first of all a result of the nonuniform flow pattern whereby the capacity (or competence) for sediment movement is different in one area than in the immediately upstream area. Therefore, scour is first of all a function of geometry which determines flow pattern

- Scour at abutments is the same as scour in a long contraction in that it is a consequence of an imbalance of the supply of sediment to an area and the capacity to move sediment out of that area.

- The limit of scour at an abutment is either a balance of supply and capacity (scour by sediment-transporting flow) or a boundary shear equal to the critical tractive force (clear-water scour).

- Abutment scour is caused by the flow that is forced to be turned by the embankment. The scour hole occurs within the width of the obstructed flow at the point of joining the main flow.

On page 57 Laursen makes these observations:

- The flow and sediment transport in the channel beyond the lateral limits of a scour hole are not noticeably affected by the obstruction or the scour hole.

- The flow approaching a scour hole, but outside the lateral limits of the actual obstruction, is virtually unaffected by the obstruction although the sediment being transported, of course, falls into the hole.

- The flow that approaches the obstruction dives down into the scour hole, takes the form of a spiral roller within the scour hole which bends around the sides of the obstruction and out of the scour hole in a flat tail as it mixes with the flow above. The spiral roller is the agent which moves the sediment on out of the scour hole.

There are many procedures available for computing predicted abutment scour. ADOT recommends the use of Froehlich's equation for calculating abutment scour as described below. The predicted scour should be checked with the HIRE equation.

Froehlich's equation for calculation of predicted abutment scour is

$$y_s / y_a = 2.27 * K_1 * K_2 * (L'/y_a)^{0.43} * Fr^{0.61} + 1 \qquad \textbf{(10.3)}$$

where:

Y_S= predicted abutment scour, ft.

y_a = Average flow depth on the floodplain, ft.

L' = Length of active flow obstructed by the abutment and approach embankment, ft

Fr = Froude number based on the velocity just upstream of the abutment.

10.6 Bridge Scour (continued)

10.6.5.4 Abutment Scour (continued)

Factor Values:

K_1= factor for abutment shape.
K_1= 1.0, vertical wall abutment, to be used when evaluating the capacity of the abutment foundation with an eroded embankment slope.
K_1= 0.55, spill through abutment, can be used for the design depth of bank protection when failure of the bank protection does not endanger the safety of the bridge.

K_2= factor for angle between flow direction and embankment alignment.
$K_2 = (Q/90)^{0.13}$

$Q<90$ if embankment points downstream
$Q>90$ if embankment points upstream

Based on the bridge crossing layout, determine the right and left encroachment lengths. As mentioned in HEC-18, abutment scour depends on the interaction of the flow obstructed by the abutment and roadway approach and the flow in the main channel. To apply these equations, the obstructed flow and the approach flow must be determined. Where Q_{ob} is the obstructed flow and q_w is the unit flow adjacent to the abutment.

y_a = Average flow depth on the floodplain, ft.
$y_a = A_{ob}/T_{ob}$, where A_{ob} is the area of the obstructed flow and T_{ob} is the top width for the obstructed flow

HIRE: For HIRE, scour is a function of the depth and Froude number for the flow adjacent to the abutment.

$y_s = 4.0*K_1*K_2 *Fr^{0.33}\, y_a$

with K_1, K_2, and y_a the same as for Froelich.

The abutment scour prediction should follow the following steps:

1. Run a hydraulic model to determine the main channel, left and right overbank discharges

2. Determine Q_o, q_w, and A_{ob}.

3. Determine $L' = Q_{ob}/q_w$ and $y_a = A_{ob}/T_{ob}$.

4. Apply the values for L' and y_a in equation 10.3. ADOT makes the following adjustment in the application of equation 10.3. **If L'/y_a is greater than 25, set L'/y_a to 25 and solve for Y_s. Compare with the scour resulting from use of HIRE. Use the larger scour value.**

10.6 Bridge Scour (continued)

10.6.5.4 Abutment Scour (continued)

5. The datum for the predicted depth is the channel bottom without contraction scour. The predicted depth includes both abutment scour and contraction scour.

See Appendix D for an example and comparison of abutment scour computations.

10.6.5.5 Pier Scour

At bridge piers, the flow that is obstructed changes into a “horseshoe” pattern around the pier. There is also a component that flows downward along the pier. This flow continues downward until either the resistance of the water consumes the energy or another obstruction is encountered. If the obstruction is the bed of the stream, the flow will, depending on the grain size, loosen the bed material creating a hole. The flow will carry the displaced material out of the hole until the combination of grain size, stream force, and depth of hole result in an equilibrium condition with no additional material being carried out of the hole for clear water scour, or a balance of sediment inflow and outflow transport for live-bed scour.

The scour is related to the affected discharge, ($a*Y_O$), and the relative velocity of flow.

The recommended equation for calculation of predicted pier scour is

$$(Y_S/Y_o)=2.0* K_1*K_2*K_3*K_4*(a/Y_o)^{0.65}*Fr^{0.43} \qquad \textbf{(10.4)}$$

where:

Y_S= predicted pier scour, ft.

Y_o=flow depth upstream of pier, ft.

a = pier width, including adjustment for multiple piers and debris, ft

Fr=Froude number based on the velocity just upstream of the pier.

Factor Values:
K_1= factor for pier nose shape.
K_1=1.1, ADOT policy is that the pier shape is affected by debris, consider K_1 as 1.1.

K_2= factor for angle between flow direction and pier alignment.
Use the projected width of the pier, determined from $K_2=(\cos \theta+(L/a)*\sin \theta)^{0.65}$
with a limiting value of 5.0. See Table 10.6.1.

K_3= factor for bed condition.
K_3= 1.1 for most cases where plane bed conditions exist, dune height <= 10 feet.

K_4=factor for armoring by bed material.
K_4 = 1.0, ADOT policy is for no decrease due to bed armoring.

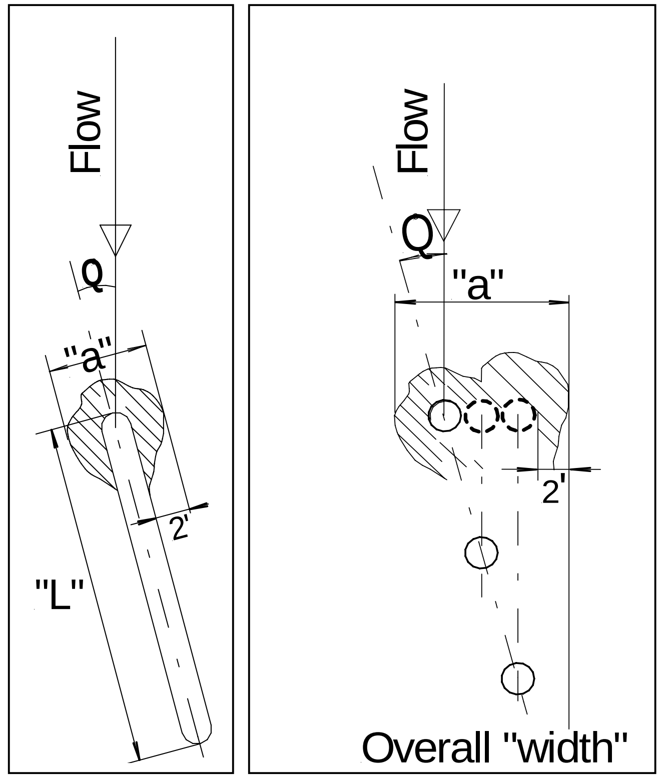
For these equations, it is assumed that the bed material is sufficiently fine-grained so that the material gradation does not affect the predicted scour.

10.6 Bridge Scour (continued)

10.6.5.5 Pier Scour (continued)

Table 10.6.1
Correction factor, K_2
for angle of attack of the flow
$K_2=(Cos(Q)+(L/a)*Sin(Q))^{0.65}$

Angle	L/a=4	L/a=8	L/a=12
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0



Where
Q= skew angle of flow,
ADOT requires a minimum value of 15 degrees,
site conditions may require a greater value.
L = length of pier, ft
a = effective width of pier including debris, ft.
If L/a is greater than 12, use the values for L/a =12.

In the absence of additional site information, for all piers, a debris width of 4 feet will be added to the normal width of pier. The debris shall be assumed to extend to a depth of 12 feet from the water surface or flow depth, whichever is less.

For pier frame with multiple columns, if the clear opening width between columns is greater than 16 feet, they shall be evaluated as single columns. If the clear opening width between columns is less than 16 feet, the overall width over the exterior columns shall be used as the obstructed width, a. Debris width shall be included in the width determined above. For this condition, use K_2=1.

Piers that are located close to an abutment, (such as at the toe of a spill through abutment), must be carefully evaluated. They may be exposed to a much higher angle of attack and velocity. Also they may be within the influence of the predicted “abutment scour”.

See Appendix E for an example of pier scour computation.

10.7 References

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Appendix A Flow Transitions in Bridge Backwater Analysis

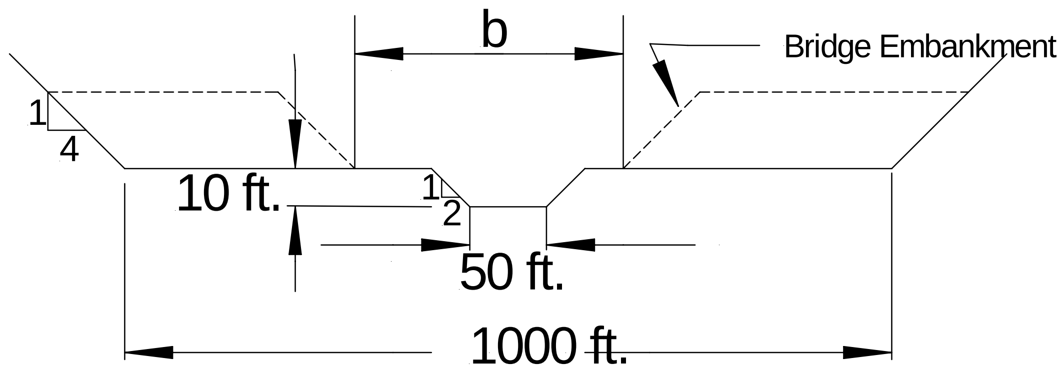
Bridges across floodplains, if they cause severe contraction and expansion of the flow, require special attention in one-dimensional modeling. The accurate prediction of the energy losses in the contraction reach and expansion reach requires the accurate evaluation of four parameters: the expansion reach length, L_e, the contraction reach length, L_c; the expansion coefficient, C_e; and the contraction coefficient, C_c. Research was conducted to investigate these four parameters through the use of field data, two-dimensional modeling, and one-dimensional modeling. The data consisted of 3 actual bridge sites and 76 idealized bridge sites. The field data had the following characteristics: wide, heavily vegetated overbanks, with Manning's n values from 0.07 to 0.24, and slopes between 2.5 feet/mile and 8.0 feet/mile. The idealized bridge sites used the following data:

B=1000 Feet
Bridge opening: 100, 250, and 500 feet
Discharge: 5000, 10000, 20000, and 30000 cfs

Main channel Manning's n = 0.04
Overbank Manning's n: 0.04, 0.08, and 0.16.

Bed slope: 1,5, and 10 feet/mile.

The idealized cross section had the following shape:



The summary statistics for the idealized cases had the following results:

Table 10.A.2
Summary Statistics

Variable	L_e	L_c	C_e	C_c
Sample Size	76	76	76	76
Average	564	386	0.27	0.44
Median	500	360	0.30	0.10
Standard Deviation	249	86	0.15	0.06
Minimum	260	275	0.10	0.10
Maximum	1600	655	0.65	0.50
Range	1340	380	0.55	0.40

Appendix A Flow Transitions in Bridge Backwater Analysis (continued)

CONCLUSIONS

Expansion Reach Length (L_e)

None of the two dimensional cases created for the study had an expansion ratio as great at 4:1. Most of the cases had expansion ratios between 1:1 and 2:1. The use of a 4:1 leads to a consistent over prediction of the energy losses in the expansion reach in most cases. The accompanying over prediction of the water surface elevation at the downstream face of the bridge may be conservative for flood stage prediction studies. For bridge scour studies, this overestimation of the tailwater elevation could in some circumstances lead to an underestimation of the scour potential.

It was found that the ratio of the channel Froude number at Section 2 to that at Section 1, (F_{c2}/F_{c1}), correlated strongly with the length of the expansion reach. Regression equations were developed for both the expansion reach length and ratio. The equation for expansion length also includes the average obstruction length. To use these regression equations in the application of one-dimensional model will usually require an iterative process since the hydraulic properties at section 2 will not be know in advance.

The value of the Froude number ratio reveals important information about the relationship between the constricted flow and the normal flow conditions. It is in effect a measure of the degree of flow constriction since it compares the intensity of flow at the two locations. Since these Froude numbers are for the main channel only, the value of F_{c1} also happens to reflect to some extent the distribution of flow between the overbanks and main channel.

Expansion Coefficients

Regression analysis for this parameter was only marginally successful. The resulting relationship is a function of the ration of hydraulic depth in the overbank to than in the main channel for undisturbed conditions (evaluated at Section 1).

Contraction reach length (L_c)

The contraction ratios for the idealized cases ranged from 0.7:1 to 2.3:1. As with the expansion reach length, these values correlated strongly with the same Froude number ratio. A more important independent variable, however, is the decimal fraction of the total discharge conveyed in the overbanks (Q_{ob}/Q) at the approach section. A strong regression equation was developed for the contraction length.

Contraction Coefficients

69 out of 76 cases had the minimum value of 0.10, making a regression analysis not fruitful.

Asymmetric Bridge Openings

Six idealized cases were developed which had asymmetric bridge obstructions. For these data, the averages of the reach length for the two corresponding symmetric cases closely approximated the values determined for the asymmetric cases. When the regression equations for L_e, ER and L_c were applied to the asymmetric cases, the predicted values were near the observed values.

Appendix A Flow Transitions in Bridge Backwater Analysis (continued)

RECOMMENDATIONS

In applying these recommendations, the modeler should always consider the range of hydraulic and geometric conditions included in the data.

Expansion reach length

Table B.2 offers ranges of expansion ratios that can be used for different degrees of constriction, slopes, and different ratios of overbank roughness to main channel roughness. Once an expansion ratio is selected, the distance to the downstream end of the expansion reach (distance L_e) is found by multiplying the expansion ratio by the average obstruction length (the average of the distances A to B and C to D) The average obstruction length is half of the total reduction in the floodplain width caused by the two bridge approach embankments. In table B.2, for each range, the higher the value is associated with a higher discharge. The ranges in Table B.2 capture the ranges of the idealized model data.

Table 10.A.2
Expansion Ratio

"b/B"	S, ft/mile	n_{ob}/n_{mc}=1	n_{ob}/n_{mc} =2	n_{ob}/n_{mc} =4
b/B=0.1	1 ft/mile	1.4-3.6	1.3-3.0	1.2-2.1
	5 ft/mile	1.0-2.5	0.8-2.0	0.8-2.0
	10 ft/mile	1.0-2.2	0.8-2.0	0.8-2.0
b/B=0.25	1 ft/mile	1.6-3.0	1.4-2.5	1.2-2.0
	5 ft/mile	1.5-2.5	1.3-2.0	1.3-2.0
	10 ft./mile	1.5-2.0	1.3-2.0	1.3-2.0
b/B=0.50	1 ft/mile	1.4-2.6	1.3-2.9	1.2-1.4
	5 ft/mile	1.3-2.1	1.2-1.6	1.0-1.4
	10 ft./mile	1.3-2.0	1.2-1.5	1.0-1.4

The b/B is the ratio bridge opening width to total floodplain width, n_{ob} is the Manning n value for the overbank, n_{mc} is the Manning n value for the main channel and S is the longitudinal slope.

Extrapolation of expansion ratios, slopes or roughness rations outside the range used in this table should be done with care. The expansion ratio should not exceed 4:1, nor should it be less than 0.5:1. The data used to develop the recommendations had a main channel n value of 0.04. For significantly higher or lower main channel n values, the n value ratio will have a different meaning with respect to the overbank roughness.

The regression equation for the expansion reach length is as follows:

$$L_e = -298 + 257\,(F_{c2}/F_{c1}) + 0.918\,L_{obs} + 0.00479Q \qquad \textbf{(10.A.1)}$$

Appendix A Flow Transitions in Bridge Backwater Analysis (continued)

Expansion reach length (continued)

Where : L_e = length of expansion reach, in feet
F_{c2} = main channel Froude number at Section 2
F_{c1} = main channel Froude number at Section 1
L_{obs} = average length of obstruction caused by the two bridge approaches, in feet, and
Q = total discharge, cfs.

When the width of the floodplain and the discharge are smaller than those of the regression data (100 ft wide and 5000 cfs discharge) the expansion ratio can be estimated by equation 10.A.2.

$$ER = L_e/L_{obs} = 0.421 + 0.485*(F_{c2}/F_{c1}) + 0.000\ 018\ Q \qquad \textbf{(10.A.2)}$$

When the scale of the floodplain is significantly larger than that of the data, particularly when the discharge is much higher than 30,000 cfs, Equation 10.A.1 and 10.A 2 will over estimate the expansion reach length. Equation 10.A.3 should be used:

$$ER = L_e/L_{obs} = 0.489 + 0.608*(F_{c2}/F_{c1}) \qquad \textbf{(10.A.3)}$$

The depth at Section 2 is dependent upon the reach length, and the Froude number at the same section is a function of the depth. This means that an iterative process is required to use the three equations above. It is recommended that the user start with an expansion ratio from Table 10.A.1, locate section 1 according to that expansion ratio, set the main channel and overbank reach lengths as appropriate, and limit the effective flow area at section 2 to the approximate bridge opening width. The program should then be run and the main channel Froude number values at Section 2 and Section 1 read from the model output. Use these Froude numbers to determine a new expansion length from the appropriate equations, move section 1 and re-compute. When the expansion ratio is large, the resulting reach length may require intermediate cross sections, which reflect the changing width of the effective flow area.

Expansion Coefficient

The analysis of the data with regard to the expansion coefficients did not yield a regression equation that fit the data will. Equation 10.A.6 was the best equation obtained. It is recommended that the modeler use equation 10.A.6 to find an initial value, then perform a sensitivity analysis using values of the coefficient that are 0.2 higher and lower than the value from equation 10.A.6. The plus or minus 0.2 range defines the 95% confidence limit for equation 10.A.6 within the domain of the regression data. If the difference between the two ends of this range is substantial, then the conservative value should be used. The expansion coefficient should not be higher than 0.8.

Contraction reach length

Ranges of contraction ratios (CR) for different conditions are presented in Table 10.A.3. These values should be used as starting values. Note that this table does not differentiate on the basis of the degree of constriction. For each range, the higher values are typically associated with higher discharges.

Appendix A Flow Transitions in Bridge Backwater Analysis (continued)

Contraction reach length (continued)

Table 10.A.3
Contraction Ratio

S, ft/mile	n_{ob}/n_{mc}= 1	n_{ob}/n_{mc}= 2	n_{ob}/n_{mc}= 4
1 ft/mile	1.0--2.3	0.8--1.7	0.7--1.3
5 ft/mile	1.0--1.9	0.8--1.5	0.7--1.2
10 ft/mile	1.0--1.9	0.8--1.4	0.7--1.2

When the conditions are within or near those of the data, the contraction reach length can be estimated by

$$L_C = 263 + 38.8\,(F_{c4}/F_{c3}) + 257\,(Q_{ob}/Q)^2 - 58.7\,(n_{ob}/n_c)^{0.5} + 0.161 L_{obs} \qquad \textbf{(10.A.4)}$$

Where:

L_{obs} = average length of obstruction caused by the two bridge approaches, in feet, and
Q_{ob} = the discharge conveyed by the two overbanks, at the approach section (Section 4)
n_{ob} = the Manning's n for the overbanks at Section 4
n_C = the Manning's n for the main channel at Section 4

In cases where the floodplain scale and discharge are significantly larger or smaller than those used in developing the regression formulae, equation 10.A.4 should not be used. The recommended approach is to compute a value from equation 10-A-5 and check it against the values in Table 10.A.3. The contraction ratio should not exceed 2.5:1 nor should it be less than 0.3:1.

$$CR=1.4-0.333(F_{c4}/F_{c3})+1.86(Q_{ob}/Q)^2-0.19(n_{ob}/n_C)^{0.5} \qquad \textbf{(10.A.4)}$$

Contraction Coefficient
The data in the study did not lend itself to regression analysis of the contraction coefficient as for nearly all the data, the value determined was 0.1, which is considered the minimum acceptable value.

Table 10.A.4
Contraction Coefficient Values

Degree of Constriction	Recommended Contraction Coefficient
0.0<b/B<0.25	0.3-0.5
0.25<b/B<0.5	0.1-0.3
0.50 <b/B<1.0	0.1

Appendix B Hydraulics of Bridge Waterways

10.B.1 Introduction

This appendix addresses the manual calculation of bridge backwater as presented in FHWA HDS-1. The information presented in this appendix covers the necessary calculations. The user should refer to the referenced publication for a more complete coverage of the subject.

10.B.2 Hydraulics Of Bridge Waterways

Backwater

The expression for backwater has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge, section 4, and a point downstream from the bridge at which normal stage has been reestablished, section 1 (Figure 10-1a). The expression is reasonably valid if the channel in the vicinity of the bridge is essentially straight, the cross sectional area of the stream is fairly uniform, the gradient of the bottom is approximately constant between sections 1 and 4, the flow is free to contract and expand, there is no appreciable scour of the bed in the constriction and the flow is in the subcritical range.

The expression for computation of backwater upstream from a bridge constricting the flow is as follows:

$$h_4^* = K^* \alpha_3 V_{n3}^2/2g + \alpha_4 [(A_{n3}/A_1)^2 - (A_{n3}/A_4)^2]\ V_{n3}^2/2g$$

h_4^* = total backwater, ft
K^* = total backwater coefficient
α_4 & α_3 = kinetic energy coefficient, as defined below
A_{n3} = gross water area in constriction measured below normal stage, ft^2
V_{n3} = average velocity in constriction* or Q/A_{n2}, ft/s
A_1 = water area at section 4 where normal stage is reestablished, ft^2
A_4 = total water area at section 1, including that produced by the backwater, ft^2

To compute backwater, it is necessary to obtain the approximate value of h_4^* by using the first part of the expression:

$$h_4^* = [K^* \alpha_3 (V_{n3}^2)]/2g$$

The value of A_4 in the second part of expression, which depends on h_4^*, can then be determined and the second term of the expression evaluated:

$$\alpha_4 [(A_{n3}/A_1)^2 - (A_{n3}/A_4)^2]\ V_{n3}^2/2g$$

This part of the expression represents the difference in kinetic energy between sections 1 and 4, expressed in terms of the velocity head, $V_{n3}^2/2g$.

Appendix B Hydraulics of Bridge Waterways (continued)

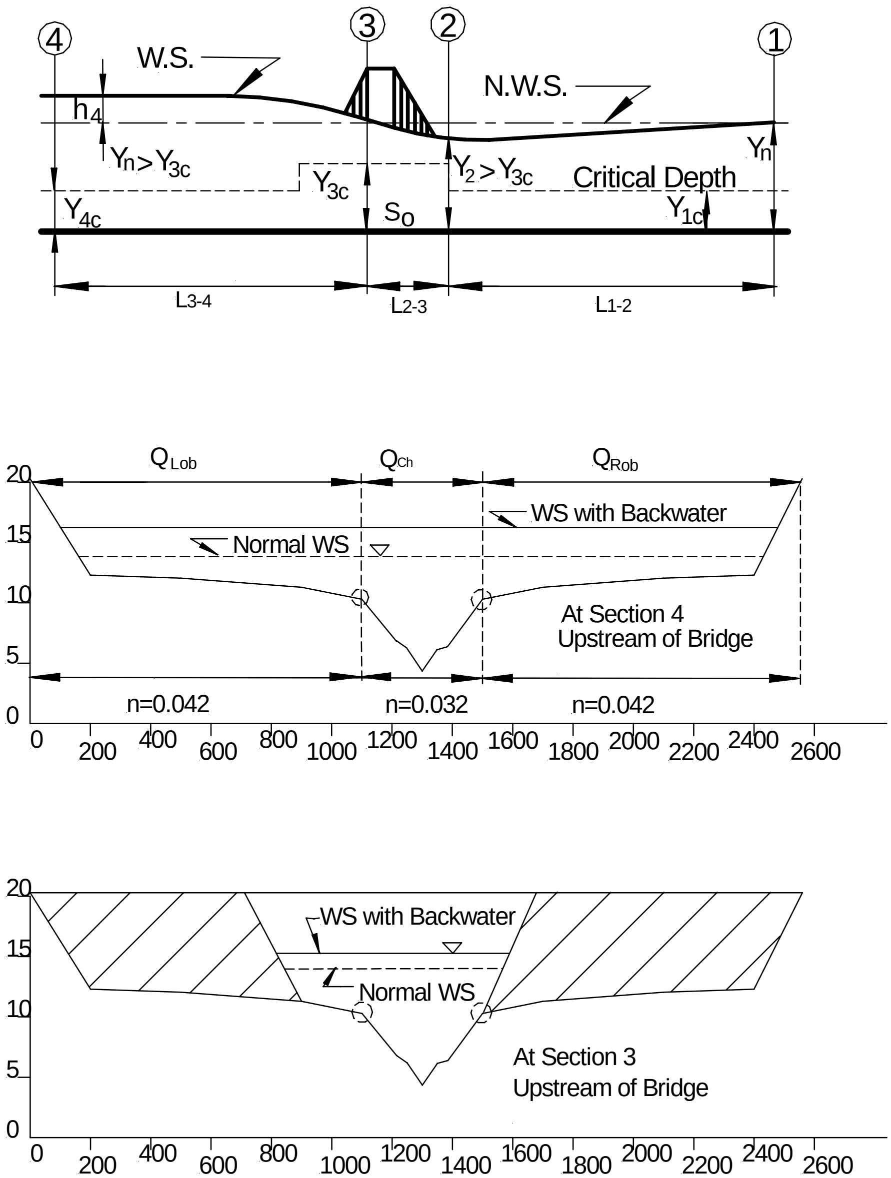


Figure 10-B-1 Normal Crossing: Spillthrough Abutments

Appendix B Hydraulics of Bridge Waterways (continued)

Bridge Opening Ratio

$M = Q_{MC}/(Q_a + Q_{MC} + Q_c)$

Kinetic Energy Coefficient

$\alpha_4 = (qv^2)/QV_4^2$

Where: v = average velocity in a subsection
q = discharge in same subsection
Q = total discharge in river
V_4 = average velocity in river at section 4 or Q/A_4

Width of Constriction

$b = A_{n3}/y$ (Figure 10-B-1)

Backwater Coefficient

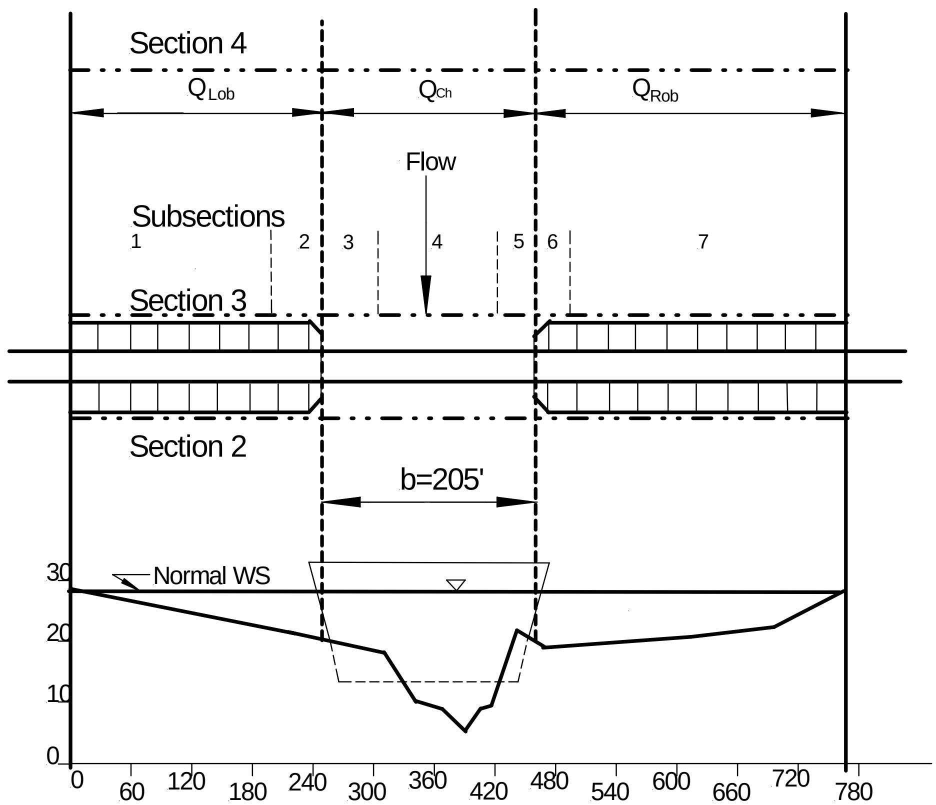
$K^* = K_b + \Delta K_p + \Delta K_s + \Delta K_e$

Where: K_b is the base constriction coefficient
ΔK_p is the pier coefficient
ΔK_s is the skew coefficient
ΔK_e is the eccentricity coefficient
Individual coefficient values are obtained from figures in HDS-1.

Appendix B Hydraulics of Bridge Waterways (continued)

Example 1 The channel crossing is shown in Figure 10-B-2 with the following information: Cross section of river at bridge site (areas, wetted perimeters, and values of Manning's n are given); normal water surface for design = El 28.0 ft at bridge; average slope of river in vicinity of bridge So=2.6 ft/mile or 0.00049 ft/ft; cross section under bridge showing area below normal water surface and width of roadway = 40 ft. The stream is essentially straight, the cross section relatively constant in the vicinity of the bridge, and the crossing is normal to the general direction of flow.

Find the Bridge Backwater caused by this roadway crossing



Subsection	n	a	p
1	0.045	627.4	200.2
2	0.070	285.2	40.1
3	0.070	324.5	40.1
4	0.035	2004.0	145.0
5	0.050	205.8	25.1
6	0.050	539.4	55.1
7	0.045	1677.4	251.0

Figure 10-B-2 Channel Crossing

Appendix B Hydraulics of Bridge Waterways (continued)

Solution

Under the conditions stated, it is permissible to assume that the cross sectional area of the stream at section 1 is the same as that at the bridge. The approach section is then divided into subsections at abrupt changes in depth or channel roughness as shown in Figure 10-B-2. The conveyance of each subsection is computed as shown in columns 1 through 8 of Table 10-B-l. The summation of the individual values in column 8 represents the overall conveyance of the stream at section 1 or K_1 = 879 489. Note that the water interface between subsections is not included in the wetted perimeter. Table 10-B-1 is set up in short form to better demonstrate the method. The actual computation would involve many subsections corresponding to breaks in grade or changes in channel roughness.

Since the slope of the stream is known (0.00049 ft/ft) and the cross sectional area is essentially constant throughout the reach under consideration, it is permissible to solve for the discharge by what is known as the slope-area method or;

$Q = K_1 S_o^{1/2} = 879{,}489 * (0.000\ 49)^{1/2} = 19{,}500\ ft^{3/s}$

To compute the kinetic energy coefficient, it is first necessary to complete columns 9, 10, 11 of Table 10-B-1; then:

$$\alpha_4 = \frac{374.895}{19{,}500\ (19{,}500/5{,}664)^2} = 1.62$$

The sum of the individual discharges in column 9 must equal 19,500 ft^3/s. The factor M is the ratio of that portion of the discharge approaching the bridge in width b, to the total discharge of the river:

$M = Q_{MC}/Q \quad M = 12{,}040/19{,}500 = 0.62$

Entering Figure 5 in HDS-1 with α_4 = 1.61 and M = 0.62, the value of α_3 is estimated as 1.40.

Entering Figure 6 in HDS-1 with M = 0.62, the base curve coefficient is K_b = 0.72 for bridge waterway of 205 ft.

As the bridge is supported by five solid piers, the incremental coefficient (ΔK_p) for this effect is determined. Referring to Figure 10-B-2 and Table 10-B-1: the gross water area under the bridge for normal stage, A_{n3} is 2,534 ft^2 and the area obstructed by the piers, $A_{p,}$ is 180 ft^2; so:

$J = 180/2{,}534 = 0.071$

Entering Figure 7A in HDS-1 with J = 0.071 for solid piers, the reading from the ordinate is ΔK = 0.13. This value is for M = 1.0. Now enter Figure 7B in HDS-1 and obtain the correction factor σ, for M = 0.62 which is 0.84. The incremental backwater coefficient for the five piers, $\Delta K_p = \Delta K\sigma = 0.13 \times 0.84 = 0.11$

Appendix B Hydraulics of Bridge Waterways (continued)

The overall backwater coefficient:

$K^* = K_b + \Delta K_p = 0.72 + 0.11 = 0.83,$

$$Vn_3 = \frac{Q}{An_3} = \frac{19{,}500}{2{,}534} = 7.70 \text{ ft/s}$$

and

$Vn_3^2/2g = (7.70)^2/2(32.2) = 0.92$ ft

The approximate backwater will be:

$K^*\alpha_3\,(Vn_3^2/2g) = 0.83 \times 1.40 \times 0.92 = 1.07$ ft.

Substituting values in the second half of expression for difference in kinetic energy between sections 1 and 4 where $A_{n4} = 5664\ ft^2 = A_1$.

$A_4 = 6384\ ft^2$ and $A_{n3} = 2534\ ft^2$

$\alpha_4\,[(A_{n3}/A_1)^2 - (A_{n3}/A_4)^2]\ \ V_{n3}^2/2g$

$1.61\,[(2534/5664)^2 - (2534/6384)^2] \times 0.92$
$= (1.61)(0.042)(0.92)$
$= 0.06$

Then total backwater produced by the bridge is

$h_4^* = 1.07 + 0.06 = 1.13$ ft.

Appendix B Hydraulics of Bridge Waterways (continued)

Table 10-B-1 Calculation Summary

	SUB-SECTION	n	A Ft^2	p Ft	$r = \frac{a}{p_{(ft)}}$ ft	$r^{2/3}$	$k=\frac{1.49}{n}ar^{2/3}$	$q=Q\frac{k}{k_1}$ cfs	$V=\frac{q}{a}$ fps	qv^2
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Q_c	0-200	0.045	627.4	200.2	3.134	2.142	44,349	983.3	1.57	2,424
	200-240	0.070	285.2	40.1	7.112	3.698	22,359	495.7	1.74	1,501
Q_b	240-280	0.070	324.5	40.1	8.092	4.031	27,732	614.8	1.89	2,196
	280-420	0.035	2004.0	145.0	13.821	5.759	490,492	10,875.2	5.43	320,654
	420-445	0.050	205.8	25.1	8.199	4.066	24,852	551.0	2.68	3,958
Q_a	445-500	0.050	539.4	55.1	9.789	4.576	73,309	1,625.4	3.01	14,726
	500-750	0.045	1677.4	251.0	6.683	3.548	196,396	4,354.6	2.60	29,436
			A_n = 5663.77 ft^2				k_1=879,489	Q=19,500 cfs		Σqv^2= 374,895
			An_3=2534 ft^2					Q_{mc}=12,040 cfs		

$\int_o$ = 0.00049

Appendix C HEC-RAS Examples

Existing - no bridge:

```
            X     X  XXXXXX    XXXX        XXXX       XX      XXXX
            X     X  X        X    X       X   X     X  X    X
            X     X  X        X            X   X    X    X   X
            XXXXXXX  XXXX     X       XXX  XXXX     XXXXXX    XXXX
            X     X  X        X            X  X     X    X        X
            X     X  X        X    X       X   X    X    X        X
            X     X  XXXXXX    XXXX        X    X   X    X   XXXXX


PROJECT DATA
Project Title: HEC18_existing_test
Project File : HEC18EXS_T.prj
Run Date and Time: 3/29/2004 1:41:36 PM

Project in English units

Project Description:
Hydraulics Manual

PLAN DATA

Plan Title: Plan 23
Plan File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.p23

           Geometry Title: HEC18_Existing_nobridge
           Geometry File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.g01
           Flow Title    : Flow 01
           Flow File     : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.f01

Plan Summary Information:
Number of:  Cross Sections =    4    Multiple Openings  =    0
            Culverts       =    0    Inline Structures  =    0
            Bridges        =    0    Lateral Structures =    0

Computational Information
    Water surface calculation tolerance  =  0.01
    Critical depth calculation tolerance =  0.01
    Maximum number of iterations         =  20
    Maximum difference tolerance         =  0.3
    Flow tolerance factor                =  0.001

Computation Options
    Critical depth computed only where necessary
    Conveyance Calculation Method: At breaks in n values only
    Friction Slope Method: Average Conveyance
    Computational Flow Regime: Subcritical Flow
```

Appendix C HEC-RAS Examples (continued)

Existing - no bridge:

```
FLOW DATA
Flow Title: Flow 01
Flow File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.f01
Flow Data (cfs)

  River           Reach           RS                    PF 1
  Test 1          1               21                   30000

Boundary Conditions
  River           Reach           Profile          Upstream        Downstream
  Test 1          1               PF 1
        Normal S = 0.002

GEOMETRY DATA

Geometry Title: HEC18_Existing_nobridge
Geometry File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.g01

CROSS SECTION
RIVER: Test 1
REACH: 1                  RS: 21
INPUT
Description:
Station Elevation Data    num= 18
Sta     Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    21.7     100    17.7     200    13.7     500   13.45     850   12.79
 900    12.7    1100    11.7    1215     8.2    1250     7.6    1300    5.75
1350    7.55    1385     7.8    1500    11.7    1700    12.7    2100   13.45
2400    13.7    2500    17.7    2600    21.7

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042

Bank Sta:         Lengths:                     Coeff
Left   Right    Left   Channel   Right     Contr.   Expan.
1100    1500    650     650        650        .1       .3

  Profile #PF 1
            Pos          Left Sta    Right Sta       Flow       Area
W.P.         Percent       Hydr   Velocity
             Conv         Depth
                         (ft)        (ft)          (cfs)    (sq ft)
(ft)                     (ft)       (ft/s)
  1           LOB          0.00        220.00         99.01      62.70
 59.42       0.33       1.06       1.58

  2           LOB          220.00      440.00        798.73     370.44
220.00       2.66       1.68       2.16

  3           LOB          440.00      660.00       1001.30     424.25
220.00       3.34       1.93       2.36

  4           LOB          660.00      880.00       1376.84     513.58
220.00       4.59       2.33       2.68
```

Appendix C HEC-RAS Examples (continued)

Existing - no bridge:

Profile #PF 1

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth (ft)	Velocity (ft/s)
5	LOB	880.00	1100.00	2124.50	666.25	220.00	7.08	3.03	3.19
6	Chan	1100.00	1200.00	3057.04	509.72	100.05	10.19	5.10	6.00
7	Chan	1200.00	1300.00	6357.06	790.88	100.05	21.19	7.91	8.04
8	Chan	1300.00	1400.00	6555.86	805.61	100.04	21.85	8.06	8.14
9	Chan	1400.00	1500.00	3232.60	527.12	100.06	10.78	5.27	6.13
10	ROB	1500.00	1720.00	2124.41	666.24	220.00	7.08	3.03	3.19
11	ROB	1720.00	1940.00	1374.16	512.99	220.00	4.58	2.33	2.68
12	ROB	1940.00	2160.00	1000.75	424.11	220.00	3.34	1.93	2.36
13	ROB	2160.00	2380.00	798.73	370.44	220.00	2.66	1.68	2.16
14	ROB	2380.00	2600.00	99.01	62.70	59.42	0.33	1.06	1.58

Warning: The energy loss was greater than 1.0 ft (0.3 m). between the current and previous cross section.
This may indicate the need for additional cross sections.

CROSS SECTION
RIVER: Test 1
REACH: 1 RS: 14.5
INPUT
Description: Downstream Section
Station Elevation Data num = 18

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
0	20.4	100	16.4	200	12.4	500	12.15	850	11.49
900	11.4	1100	10.4	1215	6.9	1250	6.3	1300	4.45
1350	6.25	1385	6.5	1500	10.4	1700	11.4	2100	12.15
2400	12.4	2500	16.4	2600	20.4				

Appendix C HEC-RAS Examples (continued)

Existing - no bridge

Manning's n Values num= 3

Sta	n Val	Sta	n Val	Sta	n Val
0	.042	1100	.032	1500	.042

Bank Sta: Left	Bank Sta: Right	Lengths: Left	Lengths: Channel	Lengths: Right	Coeff Contr.	Coeff Expan.
1100	1500	100	100	100	.1	.3

Profile #PF 1

	Pos	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P. (ft)	Percent Conv	Hydr Depth (ft)	Velocity (ft/s)
1	LOB	0.00	220.00	99.03	62.71	59.42	0.33	1.06	1.58
2	LOB	220.00	440.00	798.81	370.48	220.00	2.66	1.68	2.16
3	LOB	440.00	660.00	1001.37	424.28	220.00	3.34	1.93	2.36
4	LOB	660.00	880.00	1376.90	513.62	220.00	4.59	2.33	2.68
5	LOB	880.00	1100.00	2124.55	666.29	220.00	7.08	3.03	3.19
6	Chan	1100.00	1200.00	3056.99	509.74	100.05	10.19	5.10	6.00
7	Chan	1200.00	1300.00	6356.84	790.89	100.05	21.19	7.91	8.04
8	Chan	1300.00	1400.00	6555.63	805.63	100.04	21.85	8.06	8.14
9	Chan	1400.00	1500.00	3232.55	527.13	100.06	10.78	5.27	6.13
10	ROB	1500.00	1720.00	2124.46	666.27	220.00	7.08	3.03	3.19
11	ROB	1720.00	1940.00	1374.23	513.02	220.00	4.58	2.33	2.68
12	ROB	1940.00	2160.00	1000.83	424.15	220.00	3.34	1.93	2.36
13	ROB	2160.00	2380.00	798.80	370.48	220.00	2.66	1.68	2.16
14	ROB	2380.00	2600.00	99.03	62.71	59.42	0.33	1.06	1.58

Appendix C HEC-RAS Examples (continued)

Existing - no bridge:

```
CROSS SECTION
RIVER: Test 1
REACH: 1                  RS: 13.5
INPUT
Description: Downstream Section
Station Elevation Data    num = 18
  Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
    0    20.2     100    16.2     200    12.2     500   11.95     850   11.29
  900    11.2    1100    10.2    1215     6.7    1250     6.1    1300    4.25
 1350    6.05    1385     6.3    1500    10.2    1700    11.2    2100   11.95
 2400    12.2    2500    16.2    2600    20.2

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042

Bank Sta:          Lengths:                      Coeff
Left   Right    Left   Channel   Right     Contr.   Expan.
1100    1500     600     600     600         .1       .3

Warning: The energy loss was greater than 1.0 ft (0.3 m). between the current
and previous cross section.
         This may indicate the need for additional cross sections.
```

```
CROSS SECTION
RIVER: Test 1
REACH: 1                  RS: 7.5
INPUT
Description: Downstream Section
Station Elevation Data    num = 18
  Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
    0      19     100      15     200      11     500   10.75     850   10.09
  900      10    1100       9    1215     5.5    1250     4.9    1300    3.05
 1350    4.85    1385     5.1    1500       9    1700      10    2100   10.75
 2400      11    2500      15    2600      19

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042

Bank Sta:          Lengths:                      Coeff
Left   Right    Left   Channel   Right     Contr.   Expan.
1100    1500       0       0       0         .1       .3
```

Appendix C HEC-RAS Examples (continued)

Existing - no bridge:

SUMMARY OF MANNING'S N VALUES
River: Test 1

Reach	River Sta.	n1	n2	n3
1	21	.042	.032	.042
1	14.5	.042	.032	.042
1	13.5	.042	.032	.042
1	7.5	.042	.032	.042

SUMMARY OF REACH LENGTHS
River: Test 1

Reach	River Sta.	Left	Channel	Right
1	21	650	650	650
1	14.5	100	100	100
1	13.5	600	600	600
1	7.5	0	0	0

SUMMARY OF CONTRACTION AND EXPANSION COEFFICIENTS
River: Test 1

Reach	River Sta.	Contr.	Expan.
1	21	.1	.3
1	14.5	.1	.3
1	13.5	.1	.3
1	7.5	.1	.3

Profile Output Table - Q+Flow Dist.

Reach	River Sta	Profile	E.G. Elev (ft)	E.G. Slope (ft/ft)	W.S. Elev (ft)	Crit W.S. (ft)	Min Ch El (ft)
1	21	PF 1	15.84	0.002000	15.28		5.75
1	14.5	PF 1	14.54	0.002000	13.98		4.45
1	13.5	PF 1	14.34	0.002000	13.78		4.25
1	7.5	PF 1	13.14	0.002004	12.57	11.85	3.05

River Sta	Q Left (cfs)	Vel Left (ft/s)	Q Channel (cfs)	Vel Chnl (ft/s)	Q Right (cfs)	Vel Right (ft/s)	Top Width (ft)	Area Left (sq ft)	Area Channel (sq ft)	Area Right (sq ft)
21	5400.38	2.65	19202.56	7.29	5397.06	2.65	2278.78	2037.23	2633.33	2036.48
14.5	5400.65	2.65	19202.01	7.29	5397.34	2.65	2278.78	2037.38	2633.39	2036.63
13.5	5400.55	2.65	19202.21	7.29	5397.24	2.65	2278.78	2037.33	2633.37	2036.58
7.5	5396.70	2.65	19209.92	7.30	5393.38	2.65	2278.67	2035.22	2632.47	2034.47

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
          X     X  XXXXXX    XXXX        XXXX      XX      XXXX
          X     X  X        X    X       X   X    X  X    X
          X     X  X        X            X   X   X    X   X
          XXXXXXX  XXXX     X      XXX   XXXX    XXXXXX    XXXX
          X     X  X        X            X  X    X    X        X
          X     X  X        X    X       X   X   X    X        X
          X     X  XXXXXX    XXXX        X    X  X    X   XXXXX
```

```
PROJECT DATA
Project Title: HEC18_existing_test
Project File : HEC18EXS_T.prj
Run Date and Time: 2/26/2004 8:58:57 AM

Project in English units

Project Description:
Hydraulics Manual

PLAN DATA

Plan Title: Plan 11
Plan File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.p11

           Geometry Title: HEC18_Br1_addsect
           Geometry File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.g08

           Flow Title    : Flow 01
           Flow File     : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.f01

Plan Summary Information:
Number of:  Cross Sections =    9    Multiple Openings  =    0
            Culverts       =    0    Inline Structures  =    0
            Bridges        =    1    Lateral Structures =    0

Computational Information
    Water surface calculation tolerance  =  0.01
    Critical depth calculation tolerance =  0.01
    Maximum number of iterations         =  20
    Maximum difference tolerance         =  0.3
    Flow tolerance factor                =  0.001

Computation Options
    Critical depth computed only where necessary
    Conveyance Calculation Method: At breaks in n values only
    Friction Slope Method:         Average Conveyance
    Computational Flow Regime:     Subcritical Flow
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
FLOW DATA

Flow Title: Flow 01
Flow File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.f01

Flow Data (cfs)
  River           Reach           RS                    PF 1
  Test 1          1               21                   30000


Boundary Conditions
  River           Reach           Profile          Upstream         Downstream
  Test 1          1               PF 1
        Known WS = 12.57


GEOMETRY DATA
Geometry Title: HEC18_Br1_addsect
Geometry File : C:\PROJECTS\PROJECTS\projects\ADOT\ADT064_Drainage
Manual\HEC_RAS\HEC18EXS_T.g08

CROSS SECTION
RIVER: Test 1
REACH: 1                   RS: 21

INPUT
Description:
Station Elevation Data    num = 17
  Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
    0    21.7     100    17.7     200    13.7     500   13.45     900    12.7
 1100    11.7    1215     8.2    1250     7.6    1300    5.75    1350    7.55
 1385     7.8    1500    11.7    1700    12.7    2100   13.45    2400    13.7
 2500    17.7    2600    21.7

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042

 Bank Sta:
 Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
 1100    1500              200     200     200              .1       .3
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
  Profile #PF 1
            Pos         Left Sta    Right Sta       Flow        Area
W.P.    Percent Conv.   Hydr Depth Velocity
                          (ft)        (ft)         (cfs)     (sq ft)
(ft)                   (ft)       (ft/s)
  1         LOB          0.00       220.00        184.17      128.39
82.58      0.61        1.56       1.43

  2         LOB          220.00     440.00       1163.05      574.10
220.00      3.88        2.61       2.03

  3         LOB          440.00     660.00       1349.84      627.77
220.00      4.50        2.85       2.15

  4         LOB          660.00     880.00       1683.15      716.64
220.00      5.61        3.26       2.35

  5         LOB          880.00     1100.00      2324.83      869.89
220.00      7.75        3.95       2.67

  6         Chan         1100.00    1200.00      2817.26      602.29
100.05      9.39        6.02       4.68

  7         Chan         1200.00    1300.00      5334.74      883.45
100.05     17.78        8.83       6.04

  8         Chan         1300.00    1400.00      5484.01      898.18
100.04     18.28        8.98       6.11

  9         Chan         1400.00    1500.00      2953.93      619.69
100.06      9.85        6.20       4.77

  10        ROB          1500.00    1720.00      2324.83      869.89
220.00      7.75        3.95       2.67

  11        ROB          1720.00    1940.00      1683.15      716.64
220.00      5.61        3.26       2.35

  12        ROB          1940.00    2160.00      1349.84      627.77
220.00      4.50        2.85       2.15

  13        ROB          2160.00    2380.00      1163.05      574.10
220.00      3.88        2.61       2.03

  14        ROB          2380.00    2600.00       184.17      128.39
82.58      0.61       1.56       1.43

CROSS SECTION
RIVER: Test 1
REACH: 1                 RS: 19

INPUT
Description: Downstream Section
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    21.3     100    17.3     200    13.3     500   13.05     900    12.3
1100    11.3    1215     7.8    1250     7.2    1300    5.35    1350    7.15
1385     7.4    1500    11.3    1700    12.3    2100   13.05    2400    13.3
```

```
2500    17.3    2600    21.3
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
Bank Sta:
Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
1100    1500              200     200     200             .1       .3

Ineffective Flow     num=       2
   Sta L   Sta R    Elev  Permanent
       0     406    22.9       T
    2196    2600    22.9       T

CROSS SECTION
RIVER: Test 1
REACH: 1                  RS: 17
INPUT
Description: Downstream Section
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    20.9     100    16.9     200    12.9     500   12.65     900    11.9
1100    10.9    1215     7.4    1250     6.8    1300    4.95    1350    6.75
1385       7    1500    10.9    1700    11.9    2100   12.65    2400    12.9
2500    16.9    2600    20.9

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
Bank Sta:
Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
1100    1500              200     200     200             .1       .3

Ineffective Flow     num=       2
   Sta L   Sta R    Elev  Permanent
       0     606    22.5       T
    1796    2600    22.5       T

CROSS SECTION
RIVER: Test 1
REACH: 1                  RS: 15.0
INPUT
Description: Downstream Section
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    20.5     100    16.5     200    12.5     500   12.25     900    11.5
1100    10.5    1215       7    1250     6.4    1300    4.55    1350    6.35
1385     6.6    1500    10.5    1700    11.5    2100   12.25    2400    12.5
2500    16.5    2600    20.5

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
Bank Sta:
Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
1100    1500               50      50      50             .1       .3

Ineffective Flow     num = 2
   Sta L   Sta R    Elev  Permanent
       0     806    22.1       T
```

```
    1596    2600    22.1       T
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
CROSS SECTION
RIVER: Test 1
REACH: 1                   RS: 14.5

INPUT
Description: Downstream Section
Station Elevation Data    num = 17
 Sta     Elev     Sta     Elev     Sta     Elev     Sta    Elev     Sta    Elev
   0     20.4     100     16.4     200     12.4     500    12.15    900    11.4
1100     10.4    1215      6.9    1250      6.3    1300     4.45   1350    6.25
1385      6.5    1500     10.4    1700     11.4    2100    12.15   2400    12.4
2500     16.4    2600     20.4

REACH: 1                   RS: 14.5

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
Bank Sta:
Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
1100    1500             100     100     100               .1       .3

Ineffective Flow     num=       2
   Sta L   Sta R    Elev  Permanent
       0     856      22       T
    1546    2600      22       T

  Profile #PF 1
           Pos          Left Sta    Right Sta       Flow       Area
W.P.    Percent Conv.   Hydr Depth Velocity
                          (ft)        (ft)          (cfs)    (sq ft)
(ft)                      (ft)       (ft/s)
  1          LOB          0.00        214.00         0.00      71.08
 61.24      0.00        1.16       0.00

  2          LOB          214.00      428.00         0.00      425.65
214.00      0.00        1.99       0.00

  3          LOB          428.00      642.00         0.00      474.31
214.00      0.00        2.22       0.00

  4          LOB          642.00      856.00         0.00      557.48
214.00      0.00        2.61       0.00

  5          LOB          856.00      1100.00     3480.75      802.90
244.00     11.60        3.29       4.34

  6          Chan         1100.00     1200.00     4190.29      540.99
100.05     13.97        5.41       7.75

  7          Chan         1200.00     1300.00     8417.30      822.14
100.05     28.06        8.22      10.24

  8          Chan         1300.00     1400.00     8670.47      836.88
100.04     28.90        8.37      10.36

  9          Chan         1400.00     1500.00     4416.87      558.38
100.06     14.72        5.58       7.91
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
  Profile #PF 1
             Pos            Left Sta    Right Sta       Flow       Area
W.P.    Percent Conv.    Hydr Depth Velocity
                           (ft)        (ft)           (cfs)    (sq ft)
(ft)                       (ft)       (ft/s)
  10          ROB          1500.00     1546.00       824.33     173.57
 46.00       2.75        3.77       4.75

  11          ROB          1546.00     1756.80         0.00     665.09
210.80       0.00        3.16       0.00

  12          ROB          1756.80     1967.60         0.00     544.72
210.80       0.00        2.58       0.00

  13          ROB          1967.60     2178.40         0.00     464.60
210.80       0.00        2.20       0.00

  14          ROB          2178.40     2389.20         0.00     418.44
210.80       0.00        1.99       0.00

  15          ROB          2389.20     2600.00         0.00      65.01
58.04       0.00       1.12       0.00


BRIDGE

RIVER: Test 1
REACH: 1                   RS: 14.0

INPUT
Description: Bridge#1
B
Distance from Upstream XS = 30
Deck/Roadway Width        = 40
Weir Coefficient          = 2.6
Upstream Deck/Roadway Coordinates
    Num = 8
     Sta Hi Cord Lo Cord     Sta Hi Cord Lo Cord     Sta Hi Cord Lo Cord
       0      22      10     100      22      10     500      22      10
     850      22      10   850.1      22      18    1500      22      18
  1500.1      22      10    2600      22      10

Upstream Bridge Cross-Section Data
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    20.4     100    16.4     200    12.4     500   12.15     900    11.4
1100    10.4    1215     6.9    1250     6.3    1300    4.45    1350    6.25
1385     6.5    1500    10.4    1700    11.4    2100   12.15    2400    12.4
2500    16.4    2600    20.4

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
Bank Sta: Left   Right    Coeff Contr.   Expan.
          1100    1500             .1       .3

Ineffective Flow     num = 2
   Sta L   Sta R    Elev  Permanent
       0     856      22        T
    1546    2600      22        T

Downstream Deck/Roadway Coordinates
    Num = 8
     Sta Hi Cord Lo Cord     Sta Hi Cord Lo Cord     Sta Hi Cord Lo Cord
       0      22      10     100      22      10     500      22      10
     850      22      10   850.1      22      18    1500      22      18
  1500.1      22      10    2600      22      10

Downstream Bridge Cross-Section Data
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    20.2     100    16.2     200    12.2     500   11.95     900    11.2
1100    10.2    1215     6.7    1250     6.1    1300    4.25    1350    6.05
1385     6.3    1500    10.2    1700    11.2    2100   11.95    2400    12.2
2500    16.2    2600    20.2

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042

Bank Sta: Left   Right    Coeff Contr.   Expan.
          1100    1500             .1       .3

Ineffective Flow     num = 2
   Sta L   Sta R    Elev  Permanent
       0     876      22        T
    1526    2600      22        T

Upstream Embankment side slope             = 2 horiz. to 1.0 vertical
Downstream Embankment side slope           = 2 horiz. to 1.0 vertical
Maximum allowable submergence for weir flow =     .95
Elevation at which weir flow begins        =
Energy head used in spillway design        =
Spillway height used in design             =
Weir crest shape                           = Broad Crested

Number of Abutments = 1

Abutment Data
Upstream     num = 4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
     816      18     817       4    1515       4    1516      18


Downstream     num = 4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
     816      18     817       4    1515       4    1516      18

Number of Bridge Coefficient Sets = 1
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
Low Flow Methods and Data
       Energy
Selected Low Flow Methods = Highest Energy Answer

High Flow Method
       Energy Only

Additional Bridge Parameters
       Add Friction component to Momentum
       Do not add Weight component to Momentum
       Class B flow critical depth computations use critical depth
       inside the bridge at the upstream end
       Criteria to check for pressure flow = Upstream energy grade line

CROSS SECTION
RIVER: Test 1
REACH: 1                 RS: 13.5
INPUT
Description: Downstream Section
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    20.2     100    16.2     200    12.2     500   11.95     900    11.2
1100    10.2    1215     6.7    1250     6.1    1300    4.25    1350    6.05
1385     6.3    1500    10.2    1700    11.2    2100   11.95    2400    12.2
2500    16.2    2600    20.2

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
Bank Sta:
Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
1100    1500              200     200     200             .1       .3

Ineffective Flow     num = 2
   Sta L   Sta R    Elev  Permanent
       0     876      22       T
    1526    2600      22       T


Warning: The velocity head has changed by more than 0.5 ft (0.15 m).  This may
indicate the need for additional cross sections.

Warning: The conveyance ratio (upstream conveyance divided by downstream
conveyance) is less than 0.7 or greater than 1.4.  This may indicate the need
for additional cross sections.

Warning: The energy loss was greater than 1.0 ft (0.3 m). between the current
and previous cross section.  This may indicate the need for additional cross
sections.
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
CROSS SECTION

RIVER: Test 1
REACH: 1                   RS: 11.5

INPUT
Description:
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    19.8     100    15.8     200    11.8     500   11.55     900    10.8
1100     9.8    1215     6.3    1250     5.7    1300    3.85    1350    5.65
1385     5.9    1500     9.8    1700    10.8    2100   11.55    2400    11.8
2500    15.8    2600    19.8

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
Bank Sta:
Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
1100    1500             200     200     200               .1       .3

CROSS SECTION
RIVER: Test 1
REACH: 1                   RS: 9.5

INPUT
Description:
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0    19.4     100    15.4     200    11.4     500   11.15     900    10.4
1100     9.4    1215     5.9    1250     5.3    1300    3.45    1350    5.25
1385     5.5    1500     9.4    1700    10.4    2100   11.15    2400    11.4
2500    15.4    2600    19.4

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
Bank Sta:
Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
1100    1500             200     200     200               .1       .3


CROSS SECTION
RIVER: Test 1
REACH: 1                   RS: 7.5

INPUT
Description: Downstream Section
Station Elevation Data    num = 17
 Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
   0      19     100      15     200      11     500   10.75     900      10
1100       9    1215     5.5    1250     4.9    1300    3.05    1350    4.85
1385     5.1    1500       9    1700      10    2100   10.75    2400      11
2500      15    2600      19

Manning's n Values        num = 3
     Sta   n Val     Sta   n Val     Sta   n Val
       0    .042    1100    .032    1500    .042
```

Appendix C HEC-RAS Examples (continued)

With Bridge:

```
Bank Sta:
Left   Right    Lengths: Left Channel   Right      Coeff Contr.   Expan.
1100    1500               0       0       0              .1       .3


Warning: The parabolic search method failed to converge on critical depth.
The program will try the cross section slice/secant method to find critical
depth.
```

SUMMARY OF MANNING'S N VALUES

River: Test 1

Reach	River Sta.	n1	n2	n3
1	21	.042	.032	.042
1	19	.042	.032	.042
1	17	.042	.032	.042
1	15.0	.042	.032	.042
1	14.5	.042	.032	.042
1	14.0	Bridge		
1	13.5	.042	.032	.042
1	11.5	.042	.032	.042
1	9.5	.042	.032	.042
1	7.5	.042	.032	.042

SUMMARY OF REACH LENGTHS
River: Test 1

Reach	River Sta.	Left	Channel	Right
1	21	200	200	200
1	19	200	200	200
1	17	200	200	200
1	15.0	50	50	50
1	14.5	100	100	100
1	14.0	Bridge		
1	13.5	200	200	200
1	11.5	200	200	200
1	9.5	200	200	200
1	7.5	0	0	0

SUMMARY OF CONTRACTION AND EXPANSION COEFFICIENTS
River: Test 1

Reach	River Sta.	Contr.	Expan.
1	21	.1	.3
1	19	.1	.3
1	17	.1	.3
1	15.0	.1	.3
1	14.5	.1	.3
1	14.0	Bridge	
1	13.5	.1	.3
1	11.5	.1	.3
1	9.5	.1	.3
1	7.5	.1	.3

Appendix C HEC-RAS Examples (continued)

With Bridge:

Profile Output Table - Q+Flow Dist.

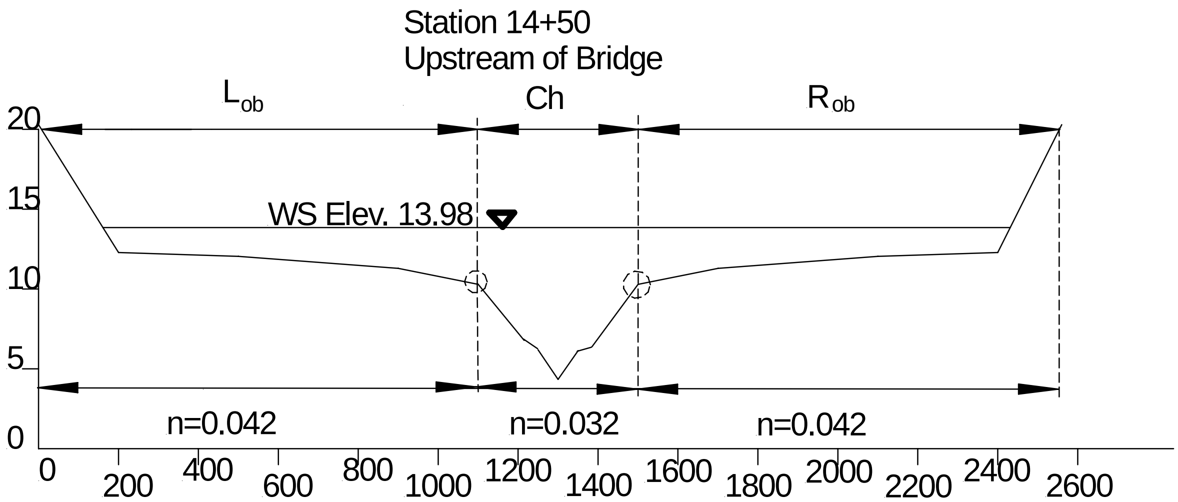
Reach	River Sta	Profile	E.G. Elev (ft)	E.G. Slope (ft/ft)	W.S. Elev (ft)	Crit W.S. (ft)	Min Ch El (ft)	Q Left (cfs)	Vel Left (ft/s)	Q Channel (cfs)	Vel Chnl (ft/s)	Q Right (cfs)	Vel Right (ft/s)	Top Width (ft)	Area Left (sq ft)	Area Channel (sq ft)	Area Right (sq ft)
1	21	PF 1	16.50	0.000963	16.20		5.75	6705.03	2.30	16589.93	5.52	6705.04	2.30	2325.06	2916.79	3003.61	2916.79
1	19	PF 1	16.30	0.001005	15.96		5.35	6220.25	2.58	17547.16	5.72	6232.59	2.57	2332.94	3068.77	3066.64	3068.77
1	17	PF 1	16.04	0.001401	15.51		4.95	5664.31	3.14	20512.76	6.73	3822.93	3.29	2330.65	3024.45	3048.28	3024.45
1	15.0	PF 1	15.65	0.002344	14.71		4.55	4111.40	3.97	24249.83	8.40	1638.77	4.30	2310.66	2640.51	2888.36	2640.51
1	14.5	PF 1	15.49	0.003068	14.29	13.00	4.45	3480.75	4.34	25694.93	9.32	824.33	4.75	2294.41	2331.42	2758.40	2331.42
1	14.0		Bridge														
1	13.5	PF 1	14.98	0.005813	13.08		4.25	2457.32	4.73	27149.93	11.54	392.75	5.37	2243.82	1385.89	2353.70	1385.89
1	11.5	PF 1	13.94	0.002003	13.37		3.85	5394.79	2.65	19210.43	7.30	5394.78	2.65	2278.69	2034.91	2632.66	2034.91
1	9.5	PF 1	13.54	0.002007	12.97		3.45	5391.64	2.65	19216.73	7.30	5391.64	2.65	2278.60	2033.19	2631.93	2033.19
1	7.5	PF 1	13.14	0.002010	12.57	11.85	3.05	5388.18	2.65	19223.64	7.31	5388.18	2.65	2278.50	2031.31	2631.13	2031.31

Appendix D Abutment Scour

HEC-18 example:

Determine the abutment scour for the given conditions:

The approach channel cross-section, station 14+50, without the bridge is shown below:



Stream Slope: S = 0.002'/'

Table 1
Cross-section Description

Point	Station	Elevation	Manning's n
1	0	20.40	0.042
2	200	12.40	0.042
3	500	12.15	0.042
4	900	11.4	0.042
5	1100	10.40	0.032
6	1215	6.90	0.032
7	1250	6.30	0.032
8	1300	4.45	0.032
9	1350	6.25	0.032
10	1385	6.50	0.032
11	1500	10.40	0.042
12	1700	11.40	0.042
13	2100	12.15	0.042
14	2400	12.40	0.042
15	2600	20.40	0.042

Appendix D Abutment Scour (continued)

Discharge: QT = 30,000 cfs.
Unencroached condition:
Flow distribution:

From HEC-RAS for existing conditions at Section 14+50, the water surface elevation is 13.98.

Table 2
HEC-RAS
Without bridge

Slice ID.	Left Station	Right Station	Flow	Area	Hyd. Depth	Velocity
1	0	220	99.1	62.74	1.06	1.58
2	220	440	799.1	370.6	1.68	2.16
3	440	660	1001.2	424.3	1.93	2.36
4	660	880	1374.6	513.1	2.33	2.68
5	880	1100	2121.8	666.4	3.03	3.19
6	1100	1200	3157.2	507.8	5.10	6.00
7	1200	1300	3232.7	527.2	7.91	8.08
8	1300	1400	6555.6	805.7	8.06	8.14
9	1400	1500	3232.7	527.2	5.27	6.13
10	1500	1720	2124.8	666.4	3.03	3.19
11	1720	1940	1374.6	513.1	2.33	2.68
12	1940	2160	1001.2	424.3	1.93	2.36
13	2160	2380	799.1	370.6	1.68	2.16
14	2380	2600	99.1	62.7	1.06	1.58

Q_l= 5395.8 cfs; Q_{mc}= 19,205.4 cfs; Q_r = 5398.8 cfs

650' BRIDGE
The bridge is 650 long between the faces of abutments. The left abutment is set 200 feet left of bank, station 850. Right abutment is at the bankline, station 1500. The bridge has 6 stem wall piers that are 5 feet thick and 40 feet long. Set the piers at the following stations: 942.5, 1035.5, 1128.5, 1221.5, 1314.5, and 1407.5.

For abutment scour prediction for foundation design, assume embankment in front of abutment is scoured; therefore the abutment scour is calculated for a vertical face. Elevation at 820 = 11.55'

Encroached Section:

At the left abutment: the obstructed flow, Station 0 to Station 850.
Q_{obl} = 5395.8-2121.8-(30/220)*(1374.6) = 3087 cfs.
The obstructed area = 62.7 + 370.6 + 424.3 + 513. 1+ (190/220)*666.4 = 1946.2 sq.ft.

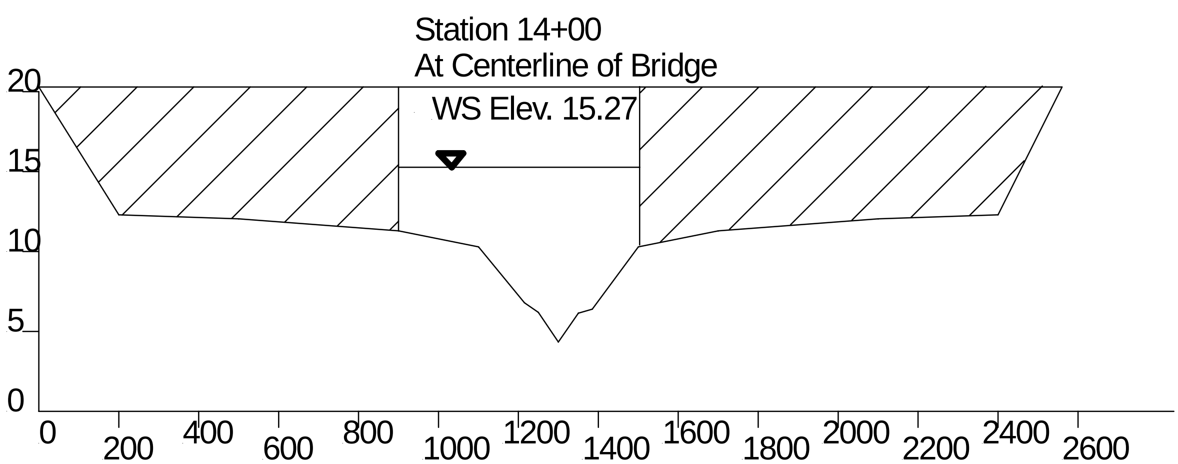
At the right abutment the obstructed flow, station 1500 to station 2600 is Q_{obr} = 5399 cfs.
The obstructed area = 666.4 + 513.1 + 424.3 + 370.6 + 62.7 = 2037.1 sq.ft.

Appendix D Abutment Scour (continued)

The distance from the bridge to the upstream section is 30 feet. Use a 1:1 contraction ratio, therefore at River Station 14.50, set the ineffective flow at 820 and 1530. At section 21.0 the encroachment stations are 170 and 2180. For the downstream section, use an expansion coefficient of 2:1, this is from the expansion coefficient table for b/B=0.25, slope = 10 feet/mile and N_{ob}/N_{mc}=1.3. Range of expansion coefficients is 1.3 to 2.0.

Table 3
Encroachment Stations
650' Bridge

River Station	Left Encroachment Station	Right Encroachment Station
7+50	535	1815
9+50	635	1715
11+50	735	1615
13+50	835	1515
13+80, BD	850	1500
14+20, BU	850	1500
14+50	820	1530
15+00	770	1580
17+00	570	1780
19+00	370	1980
21+00	170	2180



(water surface at 14.83')

Appendix D Abutment Scour (continued)

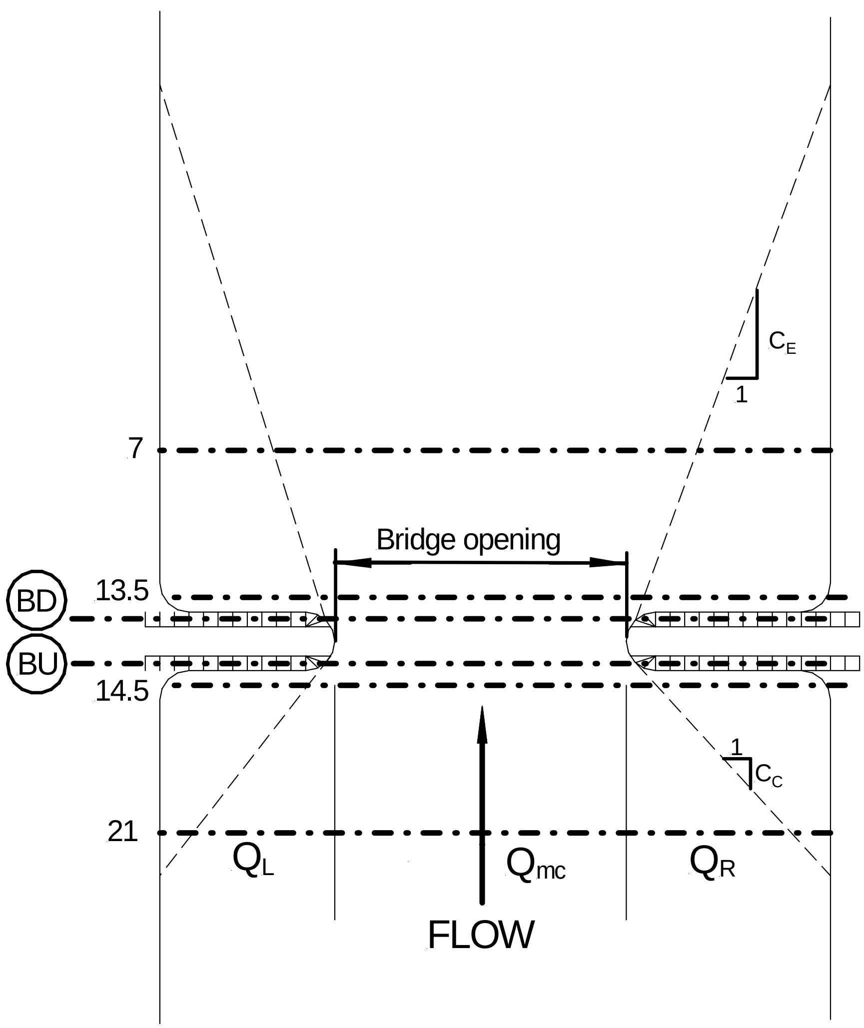
Running HEC-RAS for the encroachments defined above results in the following conditions through the bridge opening: Station 14.50

Table 4
HEC-RAS
650' Bridge

Water Surface elevation 14.83

Left Station	Right Station	Discharge	Hydraulic Depth	Velocity
820	1100	4305	3.76	4.09
1100	1200	4245	5.95	7.14
1200	1300	8091	8.76	9.24
1300	1400	8319	8.91	9.34
1400	1500	4453	6.12	7.27
1500	1530	588	4.35	4.50

The unit discharge at the right abutment: q = (4453+588)/130 = 38.6 cfs/ft.



Appendix D Abutment Scour (continued)

Section 21. WSE = 16.37

Pos - Flow	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P (ft)	Convey. Per cent	Hydr Depth (ft)	Vel. (ft/s)
1 LOB	0	170.0	0	26.8	536.7	0	0.73	0
2 LOB	170.0	402.5	1260.2	618.8	232.5	4.20	2.66	2.04
3 LOB	402.5	635.0	1514.6	691.0	232.5	5.05	2.97	2.19
4 LOB	635.0	867.5	1883.0	787.4	232.5	6.28	3.39	2.39
5 LOB	867.5	1100.0	2580.4	951.2	232.5	8.60	4.09	2.71
6 CHAN	1100.0	1200.0	2890.5	618.7	100.0	9.63	6.19	4.67
7 CHAN	1200.0	1300.0	5396.5	899.9	100.0	17.99	9.00	6.00
8 CHAN	1300.0	1400.0	5544.7	914.6	100.0	18.48	9.15	6.06
9 CHAN	1400.0	1500.0	3026.9	636.1	100.1	10.09	6.36	4.76
10 ROB	1500.0	1726.6	2541.4	930.2	226.7	8.47	4.10	2.73
11 ROB	1726.6	1953.3	1860.2	771.4	226.7	6.20	3.40	2.41
12 ROB	1953.3	2180.0	1501.6	678.4	226.7	5.01	2.99	2.21
13 ROB	2180.0	2390.0	0	579.9	210.0	0	2.76	0
14 ROB	2390.0	2600.0	0	115.5	76.7	0	1.51	0

Profile Output Table - Q+Flow Dist.

```
Reach      River Sta    Profile      E.G. Elev  E.G. Slope  W.S.E Crit  W.S.
                                       (ft)       (ft/ft)      (ft)    (ft)
Min Ch El    Q Left      Vel Left     Q Channel   Vel Chnl    Q Right    Vel Right
 (ft)       (cfs)       (ft/s)      (cfs)      (ft/s)      (cfs)        (ft/s)
Top Width   Area Left   Area Channel  Area Right
 (ft)        (sq ft)     (sq ft)       (sq ft)
--------------------------------------------------------------------------------
1             21           PF 1         16.67      0.000925   16.37     14.52
5.75         7238.21      2.37        16858.61      5.49     5903.18     2.48
2333.28     3075.32       3069.35      3075.32

1             19           PF 1         16.47      0.000988    16.12
5.35         6889.77      2.62        18007.81      5.75     5102.42     2.74
2341.00     3224.96       3131.15      3224.96

1             17           PF 1         16.24      0.001195    15.78
4.95         6189.55      3.02        20048.55      6.36     3761.89     3.19
2343.79     3279.10       3153.45      3279.10

1             15.0         PF 1         15.92      0.001858    15.13
4.55         4832.31      3.77        23709.76      7.76     1457.94     4.11
2331.52     3041.27       3055.25      3041.27

1             14.5         PF 1         15.80      0.002280    14.83     13.05
4.45         4305.01      4.09        25107.16      8.44      587.83     4.50
2321.34     2845.32       2973.88      2845.32

1             14.0                                  Bridge

1             13.5          PF 1          15.31      0.003157             14.08
4.25         3733.31       4.35       25986.15       9.44      280.54      4.87
2293.78     2319.55       2753.38      2319.55

1             11.5          PF 1          14.66      0.002665    13.68      12.45
3.85         4348.98       3.87       23883.56       8.67     1767.46      4.28
2293.85     2320.90       2753.95      2320.90
```

Appendix D Abutment Scour (continued)

Reach	River Sta	Profile	E.G. Elev (ft)	E.G. Slope (ft/ft)	W.S.E Crit (ft)	W.S. (ft)
Min Ch El (ft)	Q Left (cfs)	Vel Left (ft/s)	Q Channel (cfs)	Vel Chnl (ft/s)	Q Right (cfs)	Vel Right (ft/s)
Top Width (ft)	Area Left (sq ft)	Area Channel (sq ft)	Area Right (sq ft)			
1	9.5	PF 1	14.11	0.002314	13.30	12.07
3.45	4863.20	3.52	22374.43	8.10	2762.37	3.82
2294.95	2341.72	2762.74	2341.72			
1	7.5	PF 1	13.64	0.002003	12.96	11.70
3.05	5329.25	3.23	21132.85	7.58	3537.90	3.47
2298.07	2400.92	2787.72	2400.92			

TYPE OF SCOUR
Evaluate the type of scour, Live-bed or clear water:

<u>Clear-water Scour</u>
Clear-water scour occurs when the approach flow is not transporting sediment. This occurs when the velocity of the flow under consideration is less than the tractive shear velocity. This will usually occur in overbank areas.

<u>Live-Bed Scour</u>
Live-bed scour occurs when the approach flow is transporting sediment. This occurs when the velocity of the flow under consideration is greater than the tractive shear velocity of the sediment.

Critical Velocity:
$V_c = 11.17\ (d^{0.167})\ D_{50}{}^{0.33}$ Bed material is sand with a d_{50} of 0.0066 ft.

For main channel flow, station 1300 to station 1400, from Table 2, the hydraulic depth is 8.06 ft

$V_c = 11.17\ (8.06)^{\wedge 0.167}\ (0.0066)^{\wedge 0.33}$
V_c = 2.97 ft/sec. For sections 1 through 4, the left overbank, the existing velocity is less than the critical velocity. For the main channel the velocity is greater than the critical velocity, therefore the abutment scour is clear water scour.

Appendix D Abutment Scour (continued)

By Froehlich's Method:

$y_s/y_a = 2.27*K_1*K_2*(L'/y_a)^{0.43}*Fr^{0.61}+1$

Where: K_1= coefficient for abutment shape

K_2 = coefficient for angle of embankment to flow

L' = length of active flow obstructed by the embankment

y_a = average depth of flow on the floodplain

Fr = Froude number of approach flow upstream of the abutment

For our conditions: From section 21 with bridge in place:

Pos Flow	Left Sta(ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P(ft)	Convey. %	Hydr Depth (ft)	Vel. (ft/s)
1 LOB	0	170.0	0	26.8	536.7	0	0.73	0
2 LOB	170.0	402.5	1260.2	618.8	232.5	4.20	2.66	2.04
3 LOB	402.5	635.0	1514.6	691.0	232.5	5.05	2.97	2.19
4 LOB	635.0	867.5	1883.0	787.4	232.5	6.28	3.39	2.39
			4657.8	2124.0				

Unit flow adjacent to left abutment = 1883.0/232.5 = 8.10 cfs/ft.

Pos Flow	Left Sta(ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P(ft)	Convey. %	Hydr Depth (ft)	Vel. (ft/s)
10 ROB	1500.0	1726.6	2541.4	930.2	226.7	8.47	4.10	2.73
11 ROB	1726.6	1953.3	1860.2	771.4	226.7	6.20	3.40	2.41
12 ROB	1953.3	2180.0	1501.6	678.4	226.7	5.01	2.99	2.21
13 ROB	2180.0	2390.0	0	579.9	210.0	0	2.76	0
14 ROB	2390.0	2600.0	0	115.5	76.7	0	1.51	0
			5903.2	3075.4				

Unit flow adjacent to right abutment = 2541.4/226.5 = 11.22 cfs/ft.

For Left abutment:

The obstructed flow, Q_{obl} = 4657.8 cfs. The obstructed area = 2124.0 sq.ft.

K_1 = 1, vertical face abutment

K_2 = 1, embankment perpendicular to flow

L' = length of active flow obstructed by the embankment, = 4657.8/8.10 = 575.0'

y_a = average depth of flow on the floodplain, A/L = 2124.0/575.0 = 3.69'

L'/ y_a = 575.0/3.69 = 156. **Greater than 25, use L'/ y_a = 25**

Fr = Froude number of approach flow upstream of the abutment:

V=Q/A = 4657.8/2124.0 = 2.19 ft/sec.

$Fr = V/(g*y_a)^{0.5}$ = 2.19/(32.2*3.69) $^{\wedge 0.5}$ = 0.201

Appendix D Abutment Scour (continued)

By Froehlich's Method: (continued)

$y_s/y_a = 2.27*K_1*K_2*(L'/y_a)^{0.43}*Fr^{0.61}+1 = 2.27*1*1*(25)^{0.43}*(0.201)^{0.61} + 1$
$y_s/y_a = 2.27*3.99*0.376 +1 = 3.41+1 = 4.41$
$y_s = 4.41*y_a$
$= 4.41*3.69 = 16.3'$
At Left abutment: y_s = 16.3'

For Left abutment:
If L'/y_a is not limited to 25, but the actual is used, $L'/y_a = 156$,

then $y_s/y_a = 2.27*156^{\wedge 0.43}*0.376 +1 = 7.49+1 = 8.49$
$= 8.49*3.67 = 31.2'$

For the right abutment:
The obstructed flow, Q_{obr} = 5903.2 cfs. The obstructed area = 3075.4 sq.ft.
K_1= 1, vertical face abutment
K_2= 1, embankment perpendicular to flow
L' = length of active flow obstructed by the embankment, = 5903.2/11.22 = 526.1'
y_a = average depth of flow on the floodplain, A/L = 3075.4/526.1 = 5.85'
L'/y_a = 526.1/5.85 = 90. **Greater than 25, use L'/y_a = 25**
Fr = Froude number of approach flow upstream of the abutment:
V=Q/A = 5903.2/3075.4 = 1.92 ft/sec.
$Fr = V/(g*y_a)^{0.5} = 1.92/(32.2*5.85)^{\wedge 0.5} = 0.140$
$y_s/y_a = 2.27*K_1*K_2*(L'/y_a)^{0.43}*Fr^{0.61}+1 = 2.27*1*1*(25)^{0.43}*(0.140)^{0.61} + 1$
$y_s/y_a = 2.27*3.99*0.301 +1 =2.73+1 = 3.73$
$y_s =3.73*y_a$
$= 3.73*5.85 = 21.8'$
At right abutment: y_s = 21.8'

If L'/y_a is not limited to 25, but the actual is used, $L'/y_a = 90$,

then $y_s/y_a = 2.27*90^{\wedge 0.43}*0.301 +1 = 4.73+1 = 5.73$
$= 5.73*5.85 = 33.5'$

Appendix D Abutment Scour (continued)

HIRE: For HIRE, scour is a function of the depth and Froude number for the flow adjacent to the abutment.

$y_s = 4.0*K_1*K_2 *Fr^{0.33}\ y_a$

At left abutment: y_a = 3.76, V = 4.09. Fr = 0.372

$y_s = 4.0*(1.0/0.55)*1.0\ (0.372)^{\wedge 0.33}\ (3.76) = 7.273*0.722*3.76 = 19.7'$

At right abutment: y_a = 4.35, V = 4.50. Fr = 0.380

$y_s = 4.0*(1.0/0.55)*1.0\ (0.380)^{\wedge 0.33}\ (4.35) = 7.273*0.726*4.35 = 23.0'$

"Pier scour"

For these analyses,
As the bank erodes, the abutment may be exposed to flow as a pier, scour is also computed for this condition.

Assume the abutment is supported on two shafts that act independently. Say 5' or 7' diameter shafts at 25 foot spacing. Use the hydraulic parameters for flow through the bridge (Station 1300 to Station 1400). Hydraulic depth = 8.91 ft. and velocity =9.34 ft/sec. Channel low point elevation 4.35.

Scour at a simple round pier.

a.) Since the depth of flow is less than 12 feet the scour is predicted for only the case with debris. The scour elevation is measured from bottom of debris, the stream bed.

Pier Geometry: Diameter=5.0 ft or 7.0 ft, single round shaft, K_1=1
Debris=4.0', K_1=1.1

Flow variables: Y_1=8.91 ft. V=9.34 ft./sec.

Angle of attack: ADOT minimum=15 degrees.

Effective width: a= 5.0'+4.0'=8.0 ft.

Froude Number: Fr $=v/((G*Y_1)^{0.5})$
$=(9.34)/((32.2*8.91)^{0.5})=0.55$

For 5' shaft: consider width with debris.
Effective width: a= 5.0'+4.0'=9.0 ft.

$Y_s/Y_1=2.0*K_1*K_2*((a/Y_1)^{0.65})*((Fr^{0.43})$

Appendix D Abutment Scour (continued)

“Pier scour” (continued)

For 5’ shaft: consider width with debris.

$Y_S/8.9=2.0*(1.1)*(1.0)*((9.0/8.91)^{0.65})*(0.55^{0.43})$
$Y_S=8.9*(2.2)*(1.0)*(0.77)$
$Y_S=15.1$

For 7’ shaft:
Effective width: a= 7.0’+4.0’=11.0 ft.

$Y_S/Y_1=2.0*K_1*K_2*((a/Y_1)^{0.65})*((Fr^{0.43})$
$Y_S/8.9=2.0*(1.1)*(1.0)*((11.0/8.91)^{0.65})*(0.55^{0.43})$
$Y_S=8.9*(2.2)*(1.147)*(0.77)$
$Y_S=17.3’$

Diameter	Scour	Elevation
5	15.1	-10.65
7	17.3	-12.85

Summary – Q = 30,000 cfs.

	Left Abutment		Right Abutment		Thalweg	
Ground Elev.	11.5		10.4		4.3	
SCOUR						
Method	Depth	Elev.[3]	Depth	Elev.[3]	Depth	Elev.[3]
Froehlich[1]	16.3	-12.0	21.8	-17.5		
HIRE:	19.7	-15.4	23.0	-18.7*		
5’ “Pier Scour”					15.1	-10.8*
7’ “Pier Scour”					17.3	-13.0*
Froehlich[2]	31.2	-26.9	33.5	-29.2		

* Value for design, depending on “pier diameter”, use lower elevation from either HIRE or “Pier Scour”.

NOTES: 1. Froehlich’s with L'/y_a limited to 25.
2. Froehlich’s with L'/y_a equal to actual value.
3. Elevations are calculated by subtracting scour depth from thalweg elevation.

Appendix D Abutment Scour (continued)

For Superflood of 51,000 cfs.
Unencroached condition:
Flow distribution:

From HEC-RAS for existing conditions at Section 14+50, the water surface elevation is 15.38, depth of flow = 15.38-4.45 = 10.93.

Table 5
HEC-RAS
Without bridge

Slice ID.	Left Station	Right Station	Flow	Area	Velocity
1	0	220	309.5	170.6	2.29
2	220	440	2222.4	679.1	3.27
3	440	660	2523.5	732.9	3.44
4	660	880	3056.7	822.5	3.72
5	880	1100	4060.0	974.9	4.16
6	1100	1200	4621.2	650.0	7.11
7	1200	1300	8412.5	931.2	9.03
8	1300	1400	8635.8	945.9	9.13
9	1400	1500	4828.8	667.4	7.23
10	1500	1720	4059.9	974.9	4.16
11	1720	1940	3053.0	821.7	3.72
12	1940	2160	2522.7	732.8	3.44
13	2160	2380	2222.4	679.1	3.27
14	2380	2600	390.5	170.6	2.29

Q_l=12,253 cfs; Q_{mc}= 24,498 cfs; Q_r =12,249 cfs
A_l= 3379.8 sq. ft. A_{mc}= 3194.5 sq. ft. Q_r = 3379.1 sq. ft.

Since the average velocity in the overbanks is greater than 2.97 ft/sec, the critical velocity, there is sediment transport. Look at "live-bed" scour.

Encroached Section:

At the left abutment: the obstructed flow, Station 0 to Station 850.
Q_{obl} = 12,253-4060-(30/220)*(3056.7) = 7776.2 cfs.
The obstructed area = 3379.8-974.92-(30/220)*(822.25) = 2292.7 sq.ft.

At the right abutment the obstructed flow, station 1500 to station 2600 is Q_{obr} = 12,249 cfs.
The obstructed area = 3379.1 sq.ft.

The distance from the bridge to the upstream section is 30 feet. Use a 1:1 contraction ratio, therefore at River Station 14.50, set the ineffective flow at 820 and 1530. At section 21.0 the encroachment stations are 170 and 2180.   For the downstream section use an expansion coefficient of 2:1, this is from the expansion coefficient table for b/B=0.25, slope = 10 feet/mile and Nob/Nmc=1.3. Range of expansion coefficients is 1.3 to 2.0.

Appendix D Abutment Scour (continued)

Running HEC-RAS for the encroachments defined above results in the following conditions through the bridge opening:

Table 6
HEC-RAS
650' Bridge

Water Surface elevation 14.83,

Left Station	Right Station	Discharge	Hydraulic Depth	Velocity
820	1100	9642	5.98	5.76
1100	1200	7508	8.16	9.20
1200	1300	12,295	10.98	11.20
1300	1400	12,571	11.13	11.30
1400	1500	7776	8.34	9.33
1500	1530	1208	6.57	6.13

By Froehlich's Method:

$y_{s/}\ y_a = 2.27 * K_1 * K_2 * (L'/y_a)^{0.43} * Fr^{0.61} + 1$

Where: K_1= coefficient for abutment shape

K_2 = coefficient for angle of embankment to flow

L' = length of active flow obstructed by the embankment

y_a = average depth of flow on the floodplain

Fr = Froude number of approach flow upstream of the abutment

For our conditions: From section 21 with bridge in place:

Pos Flow	Left Sta(ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P (ft)	Convey. %	Hydr Depth (ft)	Vel. (ft/s)
1 LOB	0	170.0	0	209.5	102.44	0	2.05	0
2 LOB	170.0	402.5	3296.8	1230.0	232.5	6.46	5.29	2.68
3 LOB	402.5	635.0	3625.7	1302.2	232.5	7.11	5.60	2.78
4 LOB	635.0	867.5	4084.0	1398.6	232.5	8.01	6.02	2.92
			11006.5	4140.3				

Unit flow adjacent to left abutment = 4084.0/232.5 =17.57 cfs/ft.

Pos Flow	Left Sta (ft)	Right Sta (ft)	Flow (cfs)	Area (sq ft)	W.P (ft)	Convey. %	Hydr Depth (ft)	Vel. (ft/s)
10 ROB	1500.0	1726.6	4810.6	1526.0	226.7	9.43	6.73	3.15
11 ROB	1726.6	1953.3	4005.4	1367.2	226.7	7.85	6.03	2.93
12 ROB	1953.3	2180.0	3561.7	1270.2	226.7	6.98	5.62	2.80
13 ROB	2180.0	2390.0	0	1131.9	210.0	0	5.39	0
14 ROB	2390.0	2600.0	0	403.4	142.5	0	2.83	0
			12377.7	5698.7				

Unit flow adjacent to right abutment = 3561.7/226.5 = 15.72 cfs/ft.

Appendix D Abutment Scour (continued)

Froehlich's Method: (continued)

For Left abutment:
The obstructed flow, Q_{obl} = 11006.5 cfs. The obstructed area = 4140.3 sq.ft.

K_1= 1, vertical face abutment
K_2 = 1, embankment perpendicular to flow
L' = length of active flow obstructed by the embankment, = 11006.5/17.57 = 626.4'
y_a = average depth of flow on the floodplain, A/L = 4140.3/626.4 = 6.61'
L'/ y_a = 626.4/6.61 = 94.8. Greater than 25, use L'/ y_a = 25
Fr = Froude number of approach flow upstream of the abutment:
V=Q/A = 11006.5/4140.3= 2.66
Fr = $V/(g^* y_a)^{0.5}$ = 2.66/(32.2*6.61) ^0.5 = 0.182
$y_s/ y_a = 2.27^*K_1^*K_2 ^*(L'/y_a)^{0.43}{}^*Fr^{0.61}+1 = 2.27^*1^*1^*(25)^{0.43}{}^*(0.182)^{0.61} + 1$
y_s/ y_a = 2.27*3.99*0.354 +1 = 3.21+1 = 4.21
$y_s = 4.21^*y_a$
= 4.41*6.61 = 29.2'
At Left abutment: y_s = 29.2'

If L'/ y_a is not limited to 25, but the actual is used, L'/ y_a = 94.8
then y_s/ y_a = 2.27*94.8^0.43*0.354 +1 = 5.70+1 = 6.70
= 6.70*6.61 = 44.3'

For the right abutment:
The obstructed flow, Q_{obr} = 12377.7 cfs. The obstructed area = 5698.7 sq.ft.

K_1= 1, vertical face abutment
K_2= 1, embankment perpendicular to flow
L' = length of active flow obstructed by the embankment, = 12377.7/15.72 = 787.4'
y_a = average depth of flow on the floodplain, A/L = 5698.7/787.4 = 7.24'
L'/ y_a = 787.4/7.24 = 108.7. Greater than 25, use L'/ y_a = 25
Fr = Froude number of approach flow upstream of the abutment:
V=Q/A = 12377/5698.7 = 2.17
Fr = $V/(g^* y_a)^{0.5}$ = 2.17/(32.2*7.24)^0.5 = 0.142

$y_s/ y_a = 2.27^*K_1^*K_2 ^*(L'/y_a)^{0.43}{}^*Fr^{0.61}+1 = 2.27^*1^*1^*(25)^{0.43}{}^*(0.142)^{0.61} + 1$
y_s/ y_a = 2.27*3.99*0.304 +1 =2.75+1 = 3.75
$y_s = 3.75^*y_a$
= 3.75*7.24 = 27.2'
At right abutment: y_s = 27.2'

Appendix D Abutment Scour (continued)

Froehlich's Method: (continued)

If L'/ y_a is not limited to 25, but the actual is used, L'/ y_a = 109,

then $y_{s}/ y_a = 2.27*109^{\wedge 0.43}*0.304 +1 = 5.19+1 = 6.19$

$= 6.19*7.24 = 44.8'$

HIRE: For HIRE, scour is a function of the depth and Froude number for the flow adjacent to the abutment.

$y_s = 4.0*K_1*K_2 *Fr^{0.33}\ y_a$

At left abutment: y_a = 5.98, V = 5.76. Fr = 0.415

$y_s = 4.0*(1.0/0.55)*1.0*(0.415)^{\wedge 0.33}\ (5.98) = 7.273*0.748*5.98 = 32.5'$

At right abutment: y_a = 6.57, V = 6.13. Fr = 0.421

$y_s = 4.0*(1.0/0.55)*1.0\ (0.421)^{\wedge 0.33}\ (6.57) = 7.273*0.752*6.57 = 35.9'$

"Pier scour"

For these analyses,
As the bank erodes, the abutment may be exposed to flow as a pier, scour is also computed for this condition.

Assume the abutment is supported on two shafts that act independently. Say 5' or 7' diameter shafts at 25 foot spacing. Use the hydraulic parameters for flow through the bridge. Hydraulic depth = 11.13 ft. and velocity =11.3 ft/sec. Channel low point elevation 4.35.

Scour at a simple round pier.

b.) Since the depth of flow is less than 12 feet the scour is predicted for only the case with debris. The scour elevation is measured from bottom of debris, the stream bed.

Pier Geometry: Diameter=5.0 ft or 7.0 ft, single round shaft, K_1=1
Debris=4.0', K_1=1.1

Flow variables: Y_1=11.13 ft. V=11.3 ft./sec.

Angle of attack: ADOT minimum=15 degrees.

Froude Number: Fr $=v/((G*Y_1)^{0.5})$
$=(11.3)/((32.2*11.13)^{0.5})=0.60$

Appendix D Abutment Scour (continued)

"Pier scour" (continued)

$Y_S/Y_1=2.0*K_1*K_2*((a/Y_1)^{0.65})*((Fr^{0.43})$

For 5' shaft: consider width with debris.
Effective width: a= 5.0'+4.0'=9.0 ft.

$Y_S/11.13=2.0*(1.1)*(1.0)*((9.0/11.13)^{0.65})*(0.60^{0.43})$
$Y_S=11.13*(2.2)*(1.0)*(0.87)*(0.80)$
$Y_S=17.0$

For 7' shaft:
Effective width: a= 7.0'+4.0'=11.0 ft.

$Y_S/Y_1=2.0*K_1*K_2*((a/Y_1)^{0.65})*((Fr^{0.43})$
$Y_S/11.13=2.0*(1.1)*(1.0)*((11.0/11.13)^{0.65})*(0.60^{0.43})$
$Y_S=11.13*(2.2)*(0.99)*(0.80)$
$Y_S=19.4'$

Summary – Q = 51,000 cfs

	Left Abutment		Right Abutment		Thalweg	
Ground Elev.	11.5		10.4		4.3	
SCOUR						
Method	Depth	Elev.[3]	Depth	Elev.[3]	Depth	Elev.[3]
Froehlich[1]	29.2'	-24.9	27.2'	-22.9		
HIRE:	32.5'	-28.2*	35.9'	-31.6*		
5' "Pier Scour"					17.0	-12.7
7' "Pier Scour"					19.4	-15.1
Froehlich[2]	44.3'	-40.0	44.8'	-40.5		

*** Value for design, use lower elevation from HIRE**

NOTES: 1. Froehlich's with L'/y_a limited to 25.
2. Froehlich's with L'/y_a equal to actual value.
3. Elevations are calculated by subtracting scour depth from thalweg elevation.

Appendix E Pier Scour

- Simple Round Pier w/ debris
- Stem wall Pier w/ debris
- Simple Round Pier w/ flow depth >debris

Example 1. Scour at a simple round pier w/ debris.

Pier Geometry: Diameter=8.0 ft, single round shaft.
Debris=4.0 ft.
K_1=1.1, K_2 = 1.0, K_3 = 1.1, K_4 = 1.0

Flow variables: Y_1=10.2 ft. V=11.02 ft./sec.
Debris is for full depth, Y_1<12 ft.
Streambed elevation, 100.0

Angle of attack: ADOT minimum=15 degrees.

Effective width: a= 8.0'+4.0'=12.0 ft.

Froude Number: Fr $=v/((G*Y_1)^{0.5})$
$=(11.02)/((32.2*10.2)^{0.5})=0.63$

$Y_s/Y_1=2.0*K_1*K_2*K_3*K_4*((a/Y_1)^{0.65})*((Fr^{\wedge 0.43})$
$Y_s/10.2=2.0*(1.1)*(1.0)*(1.1)*(1.0)*((12.0/10.2)^{0.65})*(0.61^{0.43})$
$Y_s=10.2*(2.42)*(1.13)*(0.81)$
$Y_s=22.6$
Scour elev. = 100.0-22.6=77.4

Example 2. Scour at a stem wall pier.

Pier Geometry: Stem wall thickness=1.0 ft, length=48 ft.
Debris=4.0 ft., a= 1'+4' = 5'
K_1=1.1, K_3 = 1.1, K_4 = 1.0
L/a=48/5=9.6

Angle of attack: ADOT minimum=15 degrees. Due to site conditions, use 20 degrees
$K_2=(\cos \theta+(L/a)*\sin \theta)^{0.65}$
$K_2=(\cos(20)+(9.6)*\sin(20))^{0.65}$
$K_2=(0.940+9.6*0.342)^{0.65} = (4.22)^{0.65}$
$K_2=2.55$

Flow variables: Y_1=10.2 ft. V=11.02 ft./sec.
Debris is for full depth, Y_1<12 ft.
Stream bed at elev, 100.0

Appendix E Pier Scour (Continued)

Example 2. Scour at a stem wall pier. (continued)

Froude Number: Fr $=v/((G*Y_1)^{0.5})$
$=(11.02)/((32.2*10.2)^{0.5})=0.63$

$Y_s/Y_1=2.0*K_1*K_2* K_3* K_4* ((a/Y_1)^{0.65})*((Fr^{\wedge 0.43})$

$Y_s/10.2=2.0*(1.1)*(2.55)*(1.1)*(1.0)*((5/10.2)^{0.65})*(0.61^{0.43})$
$Y_s=10.2*(6.17)*(0.63)*(0.81)$
$Y_s=10.2*3.15 = 32.1$ ft.
Scour elev. = 100.0-32.1= 67.9 ft

Example 3. Scour at a simple round pier. Depth of flow greater than debris depth.

Scour is predicted for two cases: a.) based on debris width measured from bottom of debris and b.) without debris from stream bed.

Pier Geometry: Diameter=4.0 ft, single round shaft without debris, $K_1=1$
Debris=4.0', single round shaft with debris, $K_1=1.1$
$K_3 = 1.1$, $K_4 = 1.0$

Flow variables: $Y_1=15.0$ ft. V=11.02 ft./sec.
Debris is less than full depth, $Y_1>12$ ft.

Stream bed at elev, 100.0

Angle of attack: ADOT minimum=15 degrees.

Effective width: a= 4.0'+4.0'=8.0 ft.

Froude Number: Fr $=v/((G*Y_1)^{0.5})$
$=(11.02)/((32.2*10.2)^{0.5})=0.63$

Calculate two scour depths.

A.) consider width with debris.
Measure scour depth from bottom of debris.
$Y_s/Y_1=2.0*K_1*K_2* K_3*K_4* ((a/Y_1)^{0.65})*((Fr^{0.43})$
$Y_s/15.0=2.0*(1.1)*(1.0)*(1.1)*(1.0)* ((8.0/15.0)^{0.65})*(0.61^{0.43})$
$Y_s=15.0*(2.42)*(0.66)*(0.81)$
$Y_s=19.4$
Scour elev. = 100.0+(15-12)-19.4= 83.6

Appendix F SCOUR DESIGN INFORMATION

The ADOT Bridge Group has prepared the **LRFD Bridge Design Guidelines**. Included in the manual is a requirement for structural analysis of the effects of scour. To properly implement this requirement, estimates of scour must be provided to bridge design personnel by the drainage engineer.

Two Stages of Flow shall have scour calculated. They are:

1. Design Frequency Level (Case 3)
   This shall include the long term degradation, and the general and local scour for flows up to and including the design event. Bank protection shall be designed to withstand this scour occurrence. Debris effects shall be included in the scour calculations.

2. Maximum Expected Flood Level (SUPERFLOOD) (Case 4)

   This shall include the long term degradation, and the general and local scour for flows up to and including the maximum expected flood event. The goal of this level of design is that the bridge structure is still standing and usable, with repair or replacement of approach embankments, if necessary. Bank protection may have failed. Debris effects shall be included in the scour calculations.

The drainage engineer shall anticipate the data needs of the bridge designer and provide the necessary information in a clear and concise format. This should include the following items:

1. The existing channel thalweg (elevation) (Case 1).
2. Water surface elevation for Design frequency flood (Case 3)
3. Long term degradation (including headcut movement from existing gravel mining or allowance for future gravel mining) Case 3).
4. General and local scour at the design flow event (Case 3).
5. Water surface elevation for the maximum expected flood (Case 4)
6. General and local scour at the maximum expected flood level (Case 4).

CHAPTER 11

BANK PROTECTION

Chapter 11 Bank Protection
Table Of Contents

11.1 Introduction

11.1.1 Purpose

One of the hazards of placing a highway near a river or stream channel or other water body is the potential for erosion of the highway embankment by moving water. If erosion of the highway embankment is to be prevented, bank protection must be anticipated and the proper type and amount of protection must be provided in the right locations.

This chapter provides procedures for the design of revetments to be used as channel bank protection and channel linings on larger streams and rivers. For small discharges, procedures presented in the Channel Chapter should be used. Emphasis in this chapter has been placed on wire-tied rock riprap and soil cement revetments due to their costs and past performance. Other channel stabilization methods such as spurs, guide banks, retard structures, longitudinal dikes and bulkheads are discussed in "Stream Stability at Highway Structures," Hydraulic Engineering Circular No. 20. Bank protection shall not be used to reduce the foundation depths of bridges.

11.1.2 Erosion Potential

Channel and bank stabilization is essential to the design of any structure affected by the water environment. The identification of the potential for bank erosion, and the subsequent need for stabilization, is best accomplished through observation. A three level analysis procedure is provided in FHWA Hydraulic Engineering Circular No. 20. The three level analyses provides a rigorous procedure for determining the geomorphological characteristics, evaluating the existing conditions through field observations and determining the hydraulic and sediment transport properties of the stream.

Observations provide the most positive indication of erosion potential. Observation comparison can be based on historic information, or current site conditions. Aerial photographs, old maps and surveying notes and bridge design files and river survey data as well as gaging station records and interviews of long-time residents can provide documentation of any recent and potentially current channel movement or bank instabilities.

In addition, current site conditions can be used to evaluate stability. Even when historic information indicates that a bank has been relatively stable in the past, local conditions may indicate more recent instabilities. Local site conditions which are indicative of instabilities may include tipping and falling of vegetation along the bank, cracks along the bank surface, the presence of slump blocks, fresh vegetation laying in the channel near the channel banks, deflection of channel flows in the direction of the bank due to some recently deposited obstruction or channel course change, fresh vertical face cuts along the bank, locally high velocities along the bank, new bar formation downstream from an eroding bank, local headcuts, pending or recent cutoffs, etc. It is also important to recognize that the presence of any one of these conditions does not in itself indicate an erosion problem; some bank erosion is common in all channels even when the channel is stable.

11.1 Introduction (continued)

11.1.3 Symbols And Definitions

Table 11-1 Symbols And Definitions

Symbol	Definition	Units
AOS	Apparent opening size in filter cloth	mm
A	Coefficient used to determine the apparent opening size	
C	Coefficient, relates free vortex motion to velocity streamlines for unequal radius of curvature	-
C_u	Uniformity coefficient	
D_{50}	The median bed material size	ft.
D_{15}	The 15% finer particle size	ft.
D_{85}	The 85% finer particle size	ft.
d_{avg}	Average flow depth in the main flow channel	ft.
d_s	Estimated probable maximum depth of scour	ft.
g	Gravitational acceleration (32.2 ft/s^2)	ft/sec^2
H	Wave height	ft.
k	Permeability	cm/sec or mm/sec
K_1	Correction term reflecting bank angle	-
n	Manning's roughness coefficient	-
O_{95}	Opening size in the geotextile material for which 95% of the openings are smaller	mm
Q_{mc}	Discharge in the zone of main channel flow	ft^3/sec
R	Hydraulic radius	ft.
R_o	Mean radius of the channel centerline at the bend	ft.
S_f, S	Friction slope or energy grade line slope	ft/ft
SF	Stability factor	-
S_s, s	Specific gravity of the riprap material	-
T	Top width of the channel between its banks	ft.
V	Velocity	ft/sec
V_a	Mean channel velocity	ft/sec
W_{50}	Weight of the median particle	lbs.
Z	Superelevation of the water surface	ft.
γ	Unit weight of the riprap material	lbs/ft^3
θ	Bank angle with the horizontal	degrees
Φ	Riprap material's angle of repose	degrees

11.2 Policy

Highway alignments and improvements often cross, encroach upon or otherwise require construction of a new channel or modification of the existing channel. It is necessary to protect the public, the highway investment and the environment from the natural reaction to the highway changes. ADOT policy requires that the facility, including bank protection, will perform without significant damage for flood and flow conditions experienced on the operational frequency recurrence interval. ADOT policy also requires that the facility, including bank protection, be evaluated regarding the hazard to adjacent people and property for flood and flow conditions experienced on a 100-year recurrence interval. The facility, to the maximum extent possible, shall perpetuate natural drainage conditions thus protecting and maintaining the environment.

11.3 Design Criteria

To provide an acceptable standard level of service, ADOT traditionally employs the pre-established operational design frequencies that are based on the importance of the transportation facility to the system and the allowable risk for that facility. This is true for revetment protection. However, although the operational design flow frequency standards represent the consensus on reasonable values, actual design must consider consequences of any other event that may produce a more severe hydraulic condition. Under certain conditions, it may be appropriate to establish the level of risk allowable for a site and to design to that level. In addition, design standards of other agencies that have control or jurisdiction over the waterway or facility concerned should be addressed in the design.

Minimum Criteria:
Design Discharge -- Operational Frequency Flow
Freeboard at Operational Design Flow--1 foot plus velocity head

However, the designer should be aware that in some instances, a lower discharge may produce hydraulically worse conditions with respect to riprap stability. It is suggested that several discharge levels be evaluated to ensure that the design is adequate for all discharge conditions up to that selected as the design discharge for structures associated with bank protection.

The selection of the particular bank protection method used will depend on several factors. Among the factors to be considered are:

- constructibility
- risk of failure
- consequence of failure
- maintenance
- environmental impacts
- cost

11.4 Bank And Lining Failure Modes

11.4.1 Potential Failures

Prior to designing the bank stabilization plan, the designer must be aware of common erosion mechanisms and revetment failure modes, and the causes or driving forces behind bank erosion processes. Inadequate recognition of potential erosion processes at a particular site may lead to failure of the revetment system.

Bank failure modes include:
- particle erosion,
- translational slide,
- modified slump, and
- slump.

11.4.2 Particle Erosion

Particle erosion is the most commonly considered erosion mechanism. Particle erosion results when the tractive force exerted by the flowing water exceeds the bank material's ability to resist movement. In addition, if displaced stones are not transported from the eroded area, a mound of displaced rock will develop on the channel bed. This mound has been observed to cause flow concentration along the bank, resulting in further bank erosion.

A frequent particle erosion failure is the loss of the underlying material resulting in undermining and eventual collapse of the revetment protection. Usually the underlying material is lost through the revetment or piped under the toe of the revetment protection. This failure is very common in and extremely damaging to rigid types of protective linings. Providing a suitable filter, either natural or fabrics in conjunction with hydrostatic relief features will prevent this failure.

Another frequent type of particle erosion failure occurs at the edges of the protective feature. The interface creates turbulence that in turn increases the tractive stresses placed on the protective layer, underlying layers, and the natural bank material beyond the revetment. This failure area needs to receive special attention since extension of the protective feature usually only moves, not eliminates, the failure.

11.4.3 Translation Slide

A translational slide is a failure of riprap caused by the downslope movement of a mass of stones, with the fault line on a horizontal plane. The initial phases of a translational slide are indicated by cracks in the upper part of the bank that extend parallel to the channel. As the slide progresses, the lower part of bank separates from the upper part, and moves downslope as a homogeneous body. A resulting bulge may appear at the base of the bank if the channel bed is not scoured.

11.4 Bank And Lining Failure Modes (continued)

11.4.4 Modified Slump

The failure of riprap referred to as modified slump is the mass movement of material along an internal slip surface within the riprap blanket; the underlying material supporting the riprap does not fail. This type of failure is similar in many respects to the translational slide, but the geometry of the damaged riprap is similar in shape to initial stages of failure caused by particle erosion.

11.4.5 Slump

Slump is a rotational-gravitational movement of material along a surface of rupture that has a concave upward curve. The cause of slump failures is related to shear failure of the underlying base material. The primary feature of a slump failure is the localized displacement of base material along a slip surface, which is usually caused by excess pore pressure that reduces friction along a fault line in the base material.

11.5 Revetment Types

11.5.1 Common Types

The types of revetment used for bank protection and stabilization by ADOT include:

- wire-tied rock (gabions),
- soil cement,
- paved lining (concrete slope pavement),
- grouted rock,
- rock riprap,
- grouted fabric.

A type of revetment that has been used in the past, but is not currently recommended are sand/cement bags. Descriptions of each of these revetment types are included below.

11.5.2 Wire-Tied Rock

Wire-tied, or gabion, revetment consist of wire mesh elements filled with rock. Wire-tied rock revetments are generally of two types distinguished by shape: wire mattresses, or gabions. In wire-tied mattress designs, the individual wire mesh units are laid end-to-end and side-to-side to form a mattress layer on the channel bed or bank. The baskets comprising the mattress are subdivided and generally have a depth dimension that is much smaller than their width or length. Gabions, on the other hand, are typically rectangular or trapezoidal in shape. They are more equal-dimensional, having depths that are approximately the same as their widths, and of the same order of magnitude as their lengths. Gabions may be stacked vertically to increase the protected height.

11.5 Revetment Types (continued)

11.5.3 Soil Cement

Soil-cement generally consists of a dry mix of sand and cement and admixtures batched either in a central mixing plant or on-site. It is usually transported, placed by equipment capable of producing the width and thickness required and compacted to the required density. Control of the moisture and time after introduction of the mixing water is critical. Curing is required. This results in a rigid protection. Soil-cement can be placed either as a lining or in stepped horizontal layers. The stepped horizontal layers are extremely stable provided toe scour protection or embedment has been incorporated in the design.

11.5.4 Concrete Slope Pavement

Concrete pavement revetments (concrete slope pavement) are cast-in-place on a prepared slope to provide the necessary bank protection. Concrete pavement is a rigid revetment which does not conform to changes in bank geometry due to a removal of foundation support by subsidence, undermining, outward displacement by hydrostatic pressure, slide action, or erosion of the supporting embankment at its ends. The loss of even small sections of the supporting embankment can lead to complete failure of the revetment system.

11.5.5 Grouted Rock

Grouted rock revetment consists of rock slope-protection having voids filled with concrete grout to form a monolithic armor. Grouted rock is a rigid revetment; it will not conform to changes in the bank geometry due to settlement. As with other monolithic revetments, grouted rock is particularly susceptible to failure from undermining and the subsequent loss of the supporting bank material. Although it is rigid, grouted rock is not extremely strong: therefore, the loss of even a small area of bank support can cause failure of large portions of the revetment.

11.5.6 Riprap

Rock riprap is described as a layer or facing of loose rock, dumped or hand-placed. Materials other than rock are also referred to as riprap; for example, rubble, broken concrete slabs and preformed concrete shapes (slabs, blocks, rectangular prisms, etc.). These materials are similar to rock in that they can be hand-placed or dumped onto an embankment to form a flexible revetment.

11.5.7 Grouted Fabric Slope Pavement

Grouted fabric slope pavement revetments are constructed by injecting sand-cement mortar between two layers of double-woven fabric that has been positioned on the slope to be protected. Mortar is injected into this fabric envelope either underwater or in-the-dry. The fabric enclosure prevents dilution of the mortar during placement underwater. The two layers of fabric act first as the top and bottom form to hold the mortar in place while it hardens. This fabric, to which the mortar remains tightly bonded, then acts as tensile reinforcing to hold the mortar in place on the slope. These revetments are analogous to slope paving with reinforced concrete. The bottom layer of fabric acts as a filter cloth underlayment to prevent

11.5 Revetment Types (continued)

11.5.7 Grouted Fabric Slope Pavement (continued)

loss of soil particles through any cracks which may develop in the revetment as a result of soil subsidence. Often greater relief of hydrostatic uplift is provided by weep holes or filter points that are normally woven into the fabric and remain unobstructed by mortar during the filling operation.

11.5.8 Sand-Cement Bags

A method that has been used in the past, but is not recommended is sand-cement bag revetment. Sand-cement bag revetment generally consist of a dry mix of sand and cement placed in a burlap or other suitable bag. They are hand-placed in contact with adjacent bags. They require firm support from the protected bank. Usually a filter fabric is placed underneath this type of riprap. Adequate protection of the terminals and toe is essential. The riprap has little flexibility, low tensile strength and is susceptible to damage particularly on flatter slopes where the area of contact between the bags is less.

11.6 Design Concepts

11.6.1 Introduction

Design concepts related to the design of bank protection are discussed in this section. Subjects covered in this section include flow types, section geometry, flow in channel bends, flow resistance and extent of protection.

11.6.2 Flow Types

Open channel flow can be classified from three points of reference. These are:
- uniform, gradually varying, or rapidly varying flow;
- steady or unsteady flow; and
- subcritical or supercritical flow.

Design relationships presented in this chapter are based on the assumption of uniform, steady, subcritical flow. These relationships are also valid for gradually varying flow conditions. While the individual hydraulic relationships presented are not in themselves applicable to rapidly varying, unsteady, or supercritical flow conditions, procedures are presented for extending their use to these flow conditions (see the Channel chapter for more details related to channel design). Rapidly varying, unsteady flow conditions are common in areas of flow expansion, flow contraction and reverse flow. These conditions are common at and immediately downstream of bridges. Supercritical or near supercritical flow conditions are common at bridge constrictions and on steep sloped channels.

11.6 Design Concepts (continued)

11.6.2 Flow Types (continued)

Non-uniform, unsteady and near supercritical flow conditions create stresses on the channel boundary which are significantly different from those induced by uniform, steady, subcritical flow. These stresses are difficult to assess quantitatively. The stability factor method of riprap design presented in Section 11.7.1 provides a means of adjusting the final riprap design (which is based on relationships derived for steady, uniform, subcritical flow) for the uncertainties associated with these other flow conditions. The adjustment is made through the assignment of a stability factor. The magnitude of the stability factor is based on the level of uncertainty inherent in the design flow conditions.

11.6.3 Section Geometry

Design procedures presented in this chapter require as input channel cross-section geometry. The cross section geometry is necessary to establish the hydraulic design parameters (such as flow depth, top width, velocity, hydraulic radius, etc.) required by the bank protection design procedures, as well as to establish a construction cross section for placement of the revetment material. When the entire channel perimeter is to be stabilized, the selection of the appropriate channel geometry is only a function of the desired channel conveyance properties and any limiting geometric constraints. However, when the channel bank alone is to be protected, the design must consider the existing channel bottom geometry.

The development of an appropriate channel section for analysis is very subjective. The intent is to develop a section that reasonably simulates a **worst-case condition** with respect to bank protection stability. Information that can be used to evaluate channel geometry includes current channel surveys, past channel surveys (if available) and current and past aerial photos. In addition, the effect channel stabilization will have on the local channel section must be considered.

The first challenge arises in the selection of determining the channel profile to be used: Often it is intended to use the existing channel bottom profile. A single channel profile is usually not enough to establish the design cross section. In addition to current channel surveys, historic surveys can provide valuable information. A comparison of current and past channel surveys at the location provides information on the general stability of the site, as well as a history of past channel geometry changes. Often, past surveys at a particular site will not be available. If this is the case, past surveys at other sites in the vicinity of the design location may be used to evaluate past changes in channel geometry.

11.6.4 Flow In Channel Bends

Flow conditions in channel bends are complicated by the distortion of flow patterns in the vicinity of the bend. In long, relatively straight channels, the flow conditions are uniform and symmetrical about the centerline of the channel. However, in channel bends, the centrifugal forces and secondary currents produced lead to non-uniform and non-symmetrical flow conditions. Special consideration must be given to the increased velocities and shear stresses that are generated as a result of non-uniform flow in bends.

The following equations for estimating bend scour in sand-bed channels is presented in the “Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona” (1989):

11.6 Design Concepts (continued)

11.6.4 Flow In Channel Bends (continued)

$$Z_{bs} = \frac{0.0687 Y_{max} V_m^{0.8}}{Y_h^{0.4}\, S_e^{0.3}} \left[2.1\{\sin^2(\mathbf{a}/2)/\cos(\mathbf{a})\}^{0.2} - 1\right] \qquad \textbf{(11-1)}$$

Where:

Z_{bs} = Bend scour component of total scour depth, in feet;

= 0 when $r_c/T >= 10.0$, or $\mathbf{a} < 17.8^\circ$ deg.

= computed value when $0.5 < r_c/T < 10.0$, or $17.8^\circ < \mathbf{a} < 60^\circ$ deg

= computed value at $\mathbf{a} = 60^\circ$ when $r_c/T <= 0.5$ or $\mathbf{a} > 60^\circ$

Y_{max}= Maximum depth of flow immediately upstream of bend, in feet.

Y_h = Hydraulic depth of flow immediately upstream of bend, in feet.

V_m= Average velocity of flow immediately upstream of bend, in Ft/sec.

S_e = Energy slope immediately upstream of bend, in ft/ft.

$\mathbf{a}$ = angle formed by projection of channel centerline from point of curvature to a point which meets tangent line to the outer bank of the channel, in degrees (see below)

For a simple circular curve, the following relationship exists between the bend angle, $\mathbf{a}$, the ratio of centerline radius, r_c and the channel top width, T.

$$r_c/T = \frac{\cos \mathbf{a}}{4 \sin^2(\mathbf{a}/2)}$$

Where:

r_c = Radius of curvature along centerline of channel, in feet.

T = Channel top width, in feet.

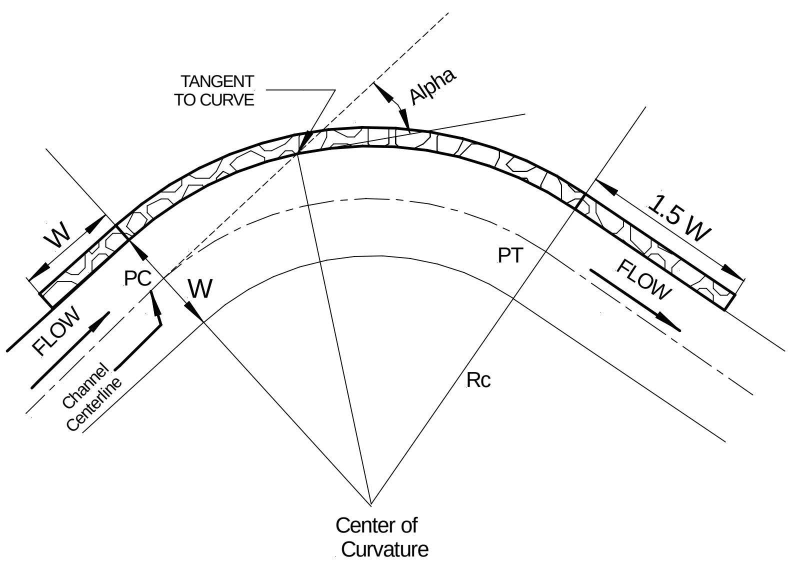


Figure 11-1 Flow at a bend

11.6 Design Concepts (continued)

11.6.4 Flow In Channel Bends (continued)

If the bend deviates significantly from a simple circular curve, the curve should be subdivided into a series of circular curves. The bend scour should be calculated for each segment of the curve based on the angle applicable to that segment. As the interaction of bend scour with other occurring channel forming activities is not known, the bend scour shall be considered to act independent of the other activities and shall be added to any other considered changes in the channel shape. Bend scour shall be considered to have a lateral extent downstream of the channel PT a distance x estimated as:

$$x = \frac{0.6Y^{1.17}}{n} \qquad \textbf{(11-2)}$$

where:

x = Distance from downstream end of channel curvature(point of tangency, PT) to the downstream point at which secondary currents have dissipated, in feet;
n = Manning’s roughness coefficient;
Y = depth of flow, in feet; (use the maximum depth of flow, exclusive of scour)

Superelevation of flow in channel bends is another important consideration in the design of revetments. Although the magnitude of superelevation is generally small when compared with the overall flow depth in the bend (usually less than 1 foot) it should be considered when establishing free board limits for bank protection schemes on sharp bends. The magnitude of superelevation at a channel bend may be estimated for subcritical flow by the following equation:

$$\mathbf{Z = C\,[(V_a^2 T)/(gR_o)]} \qquad \textbf{(11.3)}$$

where: Z = superelevation of the water surface, ft
C = coefficient that relates free vortex motion to velocity streamlines for unequal radius of curvature,
V_a = mean channel velocity, ft/sec
T = water-surface width at section, ft
g = gravitational acceleration – 32.2 ft/sec^2,
R_o = the mean radius of the channel centerline at the bend, ft

The coefficient C has been evaluated by the U.S. Geological Survey (USGS) and ranges between 0.5 and 3.0, with an average value of 1.5.

11.6.5 Flow Resistance

The hydraulic analysis performed as a part of the design process requires the estimation of Manning's roughness coefficient. Physical characteristics upon which the resistance equations are based include the channel base material, surface irregularities, variations in section geometry, bed form, obstructions, vegetation, channel meandering, flow depth and channel slope. In addition, seasonal changes in these factors must also be considered. See Channels Chapter, for a discussion of the selection of Manning's n values.

11.6 Design Concepts (continued)

11.6.6 Extent Of Protection

Extent of protection refers to the longitudinal and vertical extent of protection required to adequately protect the channel bank.

11.6.6.1 Longitudinal Extent

The longitudinal extent of protection required for a particular bank protection plan is highly dependent on local site conditions. In general, the revetment should be continuous for a distance greater than the length that is impacted by channel-flow forces severe enough to cause dislodging and/or transport of bank material. Although this is a vague criterion, it demands serious consideration. Review of existing bank protection sites has revealed that a common misconception in streambank protection is to provide protection too far upstream and not far enough downstream.

One criterion for establishing the longitudinal limits of protection required is illustrated in Figure 11-1. As illustrated, the minimum distances recommended for bank protection are an upstream distance of 1.0 channel width and a downstream distance of 1.5 channel widths from corresponding reference lines. All reference lines pass through tangents to the bend at the bend entrance or exit. This criterion is based on analysis of flow conditions in symmetric channel bends under ideal laboratory conditions.

Real-world conditions are rarely as simplistic. In actuality, many site-specific factors have a bearing on the actual length of bank that should be protected. A designer will find the above criteria difficult to apply on mildly curving bends or on channels having irregular, non-symmetric bends. Also, other channel controls (such as bridge abutments) might already be producing a stabilizing effect on the bend so that only a part of the channel bend needs to be stabilized. In addition, the magnitude or nature of the flow event might only cause erosion problems in a very localized portion of the bend, requiring that only a short channel length be stabilized. Therefore, the above criteria should only be used as a starting point. Additional analysis of site-specific factors is necessary to define the actual extent of protection required.

Field reconnaissance is a useful tool for the evaluation of the longitudinal extent of protection required, particularly if the channel is actively eroding. In straight channel reaches, scars on the channel bank may be useful to help identify the limits required for channel bank protection. In this case, it is recommended that upstream and downstream limits of the protection scheme be extended a minimum of one channel width beyond the observed erosion limits.

In curved channel reaches, the scars on the channel bank can be used to establish the upstream limit of erosion. Here a minimum of one channel width should be added to the observed upstream limit to define the limit of protection. The downstream limit of protection required in curved channel reaches is not as easy to define. Since the natural progression of bank erosion is in the downstream direction, the present visual limit of erosion might not define the ultimate downstream limit. Additional analysis based on consideration of flow patterns in the channel bend may be required.

11.6 Design Concepts (continued)

11.6.6 Extent Of Protection (continued)

11.6.6.2 Vertical Extent

The vertical extent of protection required of a revetment includes design height and foundation or toe depth.

Design Height

The design height of a riprap installation should be equal to the design highwater elevation plus some allowance for freeboard. Freeboard is provided to ensure that the desired degree of protection will not be reduced by unaccounted factors. Some such factors include:

- superelevation in channel bends,
- hydraulic jumps, and
- flow irregularities due to piers, transitions and flow junctions.

In addition, erratic phenomena such as unforeseen embankment settlement, the accumulation of silt, trash and debris in the channel, and aquatic or other growth in the channels should be considered when setting freeboard heights.

As indicated, there are many factors that must be considered in the selection of an appropriate freeboard height. As a minimum, it is recommended that a freeboard elevation of 1 foot plus velocity head be used. The Federal Emergency Management Agency requires 3 ft. for levee protection and 4 ft. at bridges for the 100-year flood. When computational procedures indicate that additional freeboard may be required, the greater height should be used. In addition, it is recommended that the designer consult existing records, and interview persons who have knowledge of past conditions when establishing the necessary vertical extent of protection required for a particular revetment installation.

Toe Depth

The undermining of revetment toe protection has been identified as one of the primary mechanisms of revetment failure. In the design of bank protection, estimates of the depth of scour are needed so that the protective layer is placed sufficiently low in the streambed to prevent undermining. The ultimate depth of scour must consider channel degradation as well as natural scour and fill processes. In application, the depth of scour, d_s, should be measured from the lowest elevation in the cross section. It should be assumed that the low point in the cross section may eventually move adjacent to the protection (even if this is not the case in the current survey). A minimum embedment depth of 5 feet should be used in natural channels.

11.7 Design Guidelines

11.7.1 Rock Riprap

This section contains design guidelines for the design of rock riprap. Guidelines are provided for bank slope, rock size, rock gradation, riprap layer thickness, filter design, edge treatment and construction considerations. In addition, typical construction details are illustrated. In most cases, the guidelines presented apply equally to rock and rubble riprap.

11.7.1.1 Bank Slope

A primary consideration in the design of stable riprap bank protection schemes is the slope of the channel bank. For riprap installations, normally the maximum recommended face slope is 2H:1V. To be stable under an identical flow conditions, a revetment with a steep slope will need larger sizes and greater thickness than one with a flatter slope.

11.7.1.2 Rock Size

The stability of a particular riprap particle is a function of its size, expressed either in terms of its weight or equivalent diameter. In the following sections, relationships are presented for evaluating the riprap size required to resist particle and wave erosion forces.

Particle Erosion

Two methods or approaches have been used historically to evaluate a material's resistance to particle erosion. These methods are the permissible velocity approach and the permissible tractive force (shear stress) approach. Under the permissible velocity approach the channel is assumed stable if the computed mean velocity is lower than the maximum permissible velocity. The tractive force (boundary shear stress) approach focuses on stresses developed at the interface between flowing water and materials forming the channel boundary.

Design Relationship

A riprap design relationship that is based on tractive force theory yet has velocity as its primary design parameter is presented in Equation 11.6. The design relationship is based on the assumption of uniform, gradually varying flow. Figure 11-7 presents a graphical solution to Equation 11-6.

$$\mathbf{D_{50} = 0.001C\, V_a^{3} / (d_{avg}^{0.5}\, K_1^{1.5})} \qquad \textbf{(11.6)}$$

Where: D_{50} = the median riprap particle size, inches
V_a = the average velocity in the main channel, ft/sec
C = Correction factor (described below)
d_{avg} = the average flow depth in the main flow channel, ft
K_1 = Factor for bank slope, see below

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

K_1 is defined as:

$$K_1 = [1-(\sin^2\Theta/\sin^2\Phi)]^{0.5} \tag{11.7}$$

Where: Θ = the bank angle with the horizontal
Φ = the riprap material's angle of repose

Equation 11.7 can be solved using Figures 11-8 and 11-9. The average flow depth and velocity used in Equation 11.6 are main channel values. The main channel is defined as the area between the channel banks (see Figure 11-2 below).

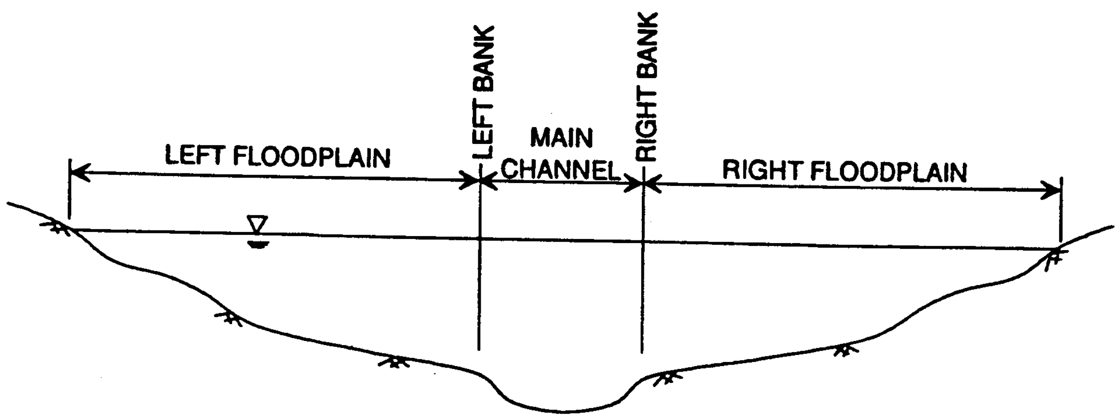


Figure 11-2 Definition Sketch; Channel Flow Distribution

Equation 11.6 is based on a rock riprap specific gravity of 2.65, and a stability factor of 1.2. Equations 11.8 and 11.9 present correction factors for other specific gravities and stability factors.

$$C_{sg} = 2.12/(S_s - 1)^{1.5} \tag{11.8}$$

Where: S_s = the specific gravity of the rock riprap

$$C_{sf} = (SF/1.2)^{1.5} \tag{11.9}$$

Where: SF = the stability factor to be applied.

The correction factors computed using Equations 11.8 and 11.9 are multiplied together to form a single correction factor C. This correction factor, C, is then multiplied by the riprap size computed from Equation 11.6 to arrive at a stable riprap size. Figure 11-10 provides a solution to Equations 11.8 and 11.9 using correction factor C.

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

The stability factor, SF, used in Equations 11.6 and 11.9 requires additional explanation. The stability factor is defined as the ratio of the average tractive force exerted by the flow field and the riprap material's critical shear stress. As long as the stability factor is greater than 1, the critical shear stress of the material is greater than the flow induced tractive stress, the riprap is considered to be stable. As mentioned above, a stability factor of 1.2 was used in the development of Equation 11.6.

The Stability Factor is used to reflect the level of uncertainty in the hydraulic conditions at a particular site. The basic relationship in Equation 11.6 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 impacts, or the cumulative effect of high shear stresses and forces from wind generated waves. The Stability Factor is used to increase the design rock size when these conditions must be considered. Table 11-3 presents guidelines for the selection of an appropriate value for the stability factor.

Table 11-3 Guidelines For The Selection Of Stability Factors

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	1.2
Gradually varying flow; Moderate bend curvature (30 >curve radius/channel width > 10); Impact from waves or floating debris moderate	1.21 - 1.6
Approaching rapidly varying flow; Sharp bend curvature (10 > curve radius/channel width); Significant impact potential from floating debris; Significant wind generated waves (1-2 ft.); High flow turbulence; Turbulently mixing flow at bridge abutments; Significant uncertainty in design parameters	1.61 - 2.0

Riprap shall have a Stability Factor (SF) equal to or greater than 1.25 for culver outlets and ditches; 1.40 for roadway embankments and channels; and 1.6 for bank protection at bridges.

Application

Application of the relationship in Equation 11.6 is limited to uniform or gradually varying flow conditions that are in **straight or mildly curving channel reaches of relatively uniform cross section**. However, design needs dictate that the relationship also be applicable in nonuniform, rapidly varying flow conditions often exhibited in natural channels with sharp bends and steep slopes, and in the vicinity of bridge piers and abutments. To fill the need for a design relationship that can be applied at sharp bends and on steep slopes in natural channels, and at bridge abutments, it is recommended that Equation 11.6 be used with appropriate adjustments in velocity and/or stability factor as outlined below.

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

Bend Factor

For channels with sharp bends, r/w <= 7, the following correction factor shall be applied to the Stability Factor.

$D = 2.65/(r/w)^{0.5}$ with $D > 1.0$

Where: r = radius of channel at center line, ft
w = channel top width, ft

NOTE: D = 1 for r/w > 7.

Steep Slopes

Flow conditions in steep sloped channels are rarely uniform, and are characterized by high flow velocities and significant flow turbulence. In applying Equation 11.6 to steep slope channels, care must be exercised in the determination of an appropriate velocity. When determining the flow velocity in steep sloped channels, it is recommended that Equation 11.10 be used to determine the channel roughness coefficient. It is also important to thoughtfully consider the guidelines for selection of stability factors as presented in Table 11-3.

On high gradient streams it is extremely difficult to obtain a good estimate of the median bed material size. For high gradient streams with slopes greater then 0.002 and bed material larger than 0.2 ft. (gravel, cobble, or boulder size material), it is recommended that the relationship given in the following equation be used to evaluate the base Manning's n.

$$\mathbf{n = 0.39\, S_f^{0.38}\, R^{-0.16}} \qquad \mathbf{(11.10)}$$

Where: S_f = friction slope, ft/ft
R = hydraulic radius, ft

11.7.1.3 Rock Gradation

The gradation of stones in riprap revetment affects the riprap's resistance to erosion. The stone should be reasonably well graded throughout the riprap layer thickness. The gradation limits should not be so restrictive that production costs would be excessive. Table 11-4 presents suggested guidelines for establishing gradation limits.

11.7 Design Guidelines (continued)

Rock Riprap (continued)

Table 11-4 Rock Riprap Gradation Limits

Percent of Gradation Smaller Than	Stone Size Range, Ft.	Stone Weight Range (lb)
100	1.5 D_{50} to 1.7 D_{50}	3.0 W_{50} to 5.0 W_{50}
85	3.0 W_{50} to 5.0 W_{50}	2.0 W_{50} to 2.75 W_{50}
50	1.0 D_{50} to 1.15 D_{50}	1.0 W_{50} to 1.5 W_{50}
15	0.4 D_{50} to 0.6 D_{50}	0.1 W_{50} to 0.2 W_{50}

It is recognized that the use of a four point gradation as specified in Table 11-4 might in some cases be too harsh a specification for some sources. If this is the case, the 85% specification can be dropped. In most instances, a uniform gradation between D_{50} and D_{100} will result in an appropriate D_{85}.

11.7.1.4 Layer Thickness

All stones should be contained reasonably well within the riprap layer thickness to provide maximum resistance against erosion. Oversize stones, even in isolated spots, may cause riprap failure by precluding mutual support between individual stones, providing large voids that expose filter and bedding materials, and creating excessive local turbulence that removes smaller stones. Small amounts of oversize stone should be removed individually and replaced with proper size stones. The following criteria apply to the riprap layer thickness.

- It should not be less than the spherical diameter of the D_{100} (W_{100}) stone, or less than 2.0 times the spherical diameter of the D_{50} (W_{50}) stone, whichever results in the greater thickness.

- It should not be less than 12 inches for practical placement.

- The thickness determined by either 1 or 2 should be increased by 50% when the riprap is placed underwater to provide for uncertainties associated with this type of placement.

- An increase in thickness of 6 to 12 inches, accompanied by an appropriate increase in stone sizes, should be provided where riprap revetment will be subject to attack by floating debris or ice, or by waves from boat wakes, wind, or bedforms.

11.7 Design Guidelines (continued)

Rock Riprap (continued)

Table 11-5 Riprap Gradation Classes

Riprap Class	Rock Size[1] (ft)	Rock Size[2] (pounds)	Percent of Riprap Smaller Than
Facing	1.30	200	100
	1.00	75	50
	0.40	5	10
Light	1.80	500	100
	1.30	200	50
	0.40	5	10
0.25 ton	2.25	1000	100
	1.80	500	50
	1.00	75	10
0.50 ton	2.85	2000	100
	2.25	1000	50
	1.80	500	5
1 ton	3.60	4000	100
	2.85	2000	50
	2.25	1000	5
2 ton	4.50	8000	100
	3.60	4000	50
	2.85	2000	5

1 Assuming a specific gravity of 2.65.
2 Based on AASHTO gradations.

11.7 Design Guidelines (continued)

11.7.1.5 Filter Design

A filter is a transitional layer of gravel, small stone, or fabric placed between the underlying soil and the structure. The filter prevents the migration of the fine soil particles through voids in the structure, distributes the weight of the armor units to provide more uniform settlement and permits relief of hydrostatic pressures within the soils. Geotextile filters have replaced granular filters in most applications.

For rock riprap, a filter ratio of 5 or less between layers will usually result in a stable condition. The filter ratio is defined as the ratio of the 15% particle size, (D_{15}) of the coarser layer to the 85% particle (D_{85}) of the finer layer. An additional requirement for stability is that the ratio of the 15% of the coarser material to the 15% particle size of the finer material should exceed 5 but be less than 40. These requirements are stated as

$$\frac{D_{15}\text{, (coarser layer)}}{D_{85}\text{, (finer layer)}} < 5 < \frac{D_{15}\text{, (coarser layer)}}{D_{15}\text{, (finer layer)}} < 40$$

The first test is intended to prevent piping, the right portion of the second test provides for adequate permeability and the right portion provides for uniformity criteria. In addition to the particle size ratios, the grain size curves should approximately parallel to minimize the infiltration of the fine material for the finer layer to the coarser layer. Figure 11-14 can be used to plot the gradation curves.

Geotextile Filters

Considerations for use of geotextile filters include the following:

- Geotextiles have widely variable hydraulic properties and must be designed based on project specific conditions and performance requirements.
- Geotextile filter performance is sensitive to construction procedures.
- Special installation and inspection procedures may be necessary when using geotextile filters.

Geotextile Filter Design

The design of geotextile filters should consider the following performance areas.
- Soil retention (piping resistance)
- Permeability
- Clogging
- Survivability

It is extremely desirable that individual site conditions and performance requirements be established in conjunction with the geotextile design. Generalized geotextile requirements should be used only on very small or non-critical/non-severe installations where a detailed analysis is not warranted. The American Association of State Highway and Transportation Officials (AASHTO) has developed materials and construction specifications (AASHTO Specification M-288) for routine, non-critical/non-severe geotextile applications. Details of geotextile filter design, for all levels of project severity and criticality are presented in the Federal Highway Administration publication, "Geosynthetic Design and Construction Guidelines,"

11.7 Design Guidelines (continued)

11.7.1.5 Filter Design (continued)

(FHWA-HI-95-038). This reference provides detailed guidance on specifying and installing geotextiles for a variety of transportation applications. The American Society for Testing Material Committee D-35 has developed standard testing procedures for approximately 35 general, index and performance properties of geosynthetics. These standard test procedures are recommended for use in design and specifications when using geosynthetics. See Appendix A for Geotextile Design information.

The following design steps are necessary for geotextile design in riprap and other permanent erosion control applications:

Step 1 **-** Evaluate the application site (determine if the application is critical or severe).

Step 2 **-** Obtain and test soil samples (perform grain size analysis and permeability tests).

Step 3 **-** Evaluate possible armor material and placement procedures.

Step 4 **-** Calculate anticipated reverse flow through the erosion control system.

Step 5 - Determine geotextile requirements:

a. Soil Retention
b. Permeability/Permittivity
c. Clogging
d. Survivability

Step 6 **-** Estimate cost and prepare specifications.

Geotextile Installation Procedures

To provide good performance, a properly selected cloth should be installed with due regard for the following precautions:

- Grade area and remove debris to provide a smooth, fairly even surface.

- Place geotextile loosely, laid with the machine (generally roll) direction in the direction of anticipated water flow or movement.

- Seam or overlap the geotextile as required.

11.7 Design Guidelines (continued)

11.7.1.5 Rock Riprap (continued)

- The maximum allowable slope on which a riprap-geotextile system can be placed is equal to the lowest soil-geotextile friction angle for the natural ground or stone-geotextile friction angle for cover (armor) materials. Additional reductions in slope may be necessary due to hydraulic considerations and possible long-term stability. For slopes greater than 2.5H:1V, special construction procedures will be required.

- For streambank and wave action applications, the geotextile must be keyed in at the bottom of the slope. If the system cannot be extended a few feet above anticipated high water level, the geotextile should also be keyed in at the crest of the slope.

- Place the revetment (cushion layer and/or riprap) over the geotextile width, while avoiding puncturing it.

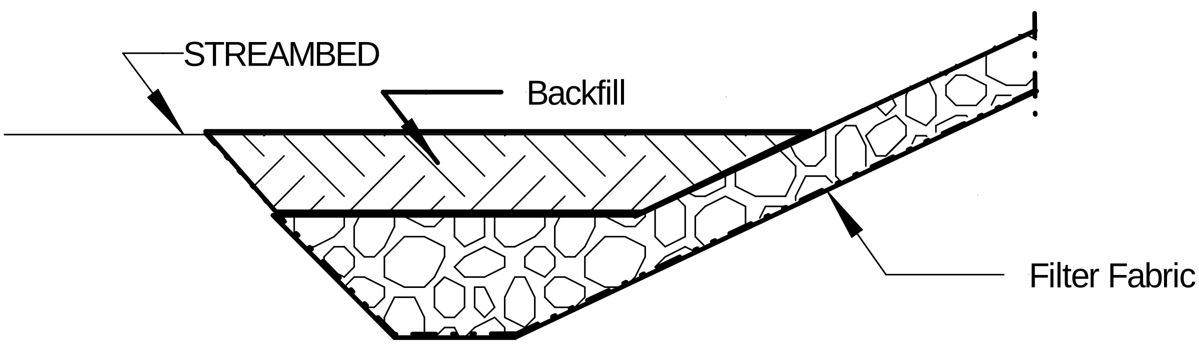


Figure 11-3 Geotextile Filters

11.7.1.6 Edge Treatment

The edges of riprap revetments (flanks, toe and head) require special treatment to prevent undermining. The flanks of the revetment should be designed as illustrated in Figure 11-4. The upstream flank and the downstream flank are illustrated in section A-A of this figure. Undermining of the revetment toe is one of the primary mechanisms of riprap failure. The toe of the riprap should be designed to be below the anticipated scour as illustrated in Figure 11-5.

11.7 Design Guidelines (continued)

11.7.1.6 Edge Treatment (continued)

The toe material should be placed in a toe trench along the entire length of the riprap blanket. Where the toe material cannot be placed at the desired depth, the riprap blanket should terminate in a thick, stone toe at the lowest level possible (see alternate design in Figure 11-6). **It shall be located below the channel thalweg.**

The size of the toe trench or the alternate stone toe is controlled by the anticipated depth of scour along the revetment. As scour occurs (and in most cases it will) the stone in the toe will launch into the eroded area as illustrated in Figure 11-8. Observation of the performance of these types of rock toe designs indicates that the riprap will launch to a final slope of approximately 2H:1V.

The volume of rock required for the toe must be equal to or exceed one and one-half times the volume of rock required to extend the riprap blanket (at its design thickness and on a slope of 2H:1V) to the anticipated depth of scour. Dimensions should be based on the required volume using the thickness and depth determined by the scour evaluation. The alternate location can be used when the amount of rock required would not constrain the channel. Establishing a design scour depth is covered in Section 11.6.7

11.7.1.7 Construction Considerations

The construction considerations related to the construction of riprap revetments include bank slope or angle, bank preparation and riprap placement.

Bank Preparation

The bank should be prepared by first clearing all trees and debris from the bank, and grading the bank surface to the desired slope. In general, the graded surface should not deviate from the specified slope line by more than 6 in. However, local depressions larger than this can be accommodated since initial placement of filter material and/or rock for the revetment will fill these depressions. In addition, any large boulders or debris found buried near the edges of the revetment should be removed.

Riprap Placement

The common methods of riprap placement are machine placing, such as from a skip, dragline, or some form of bucket; and dumping from trucks and spreading by bulldozer. In the machine placement method, sufficiently small increments of stone should be released as close to their final positions as practical. Re-handling or dragging operations to smooth the revetment surface tend to result in segregation and breakage of stone, and can result in a rough revetment surface. Stone should not be dropped from an excessive height as this may result in the same undesirable conditions.

11.7 Design Guidelines (continued)

11.7.1.7 Construction Considerations (continued)

Riprap placement by dumping with spreading is satisfactory provided the required layer thickness is achieved. Riprap placement by dumping and spreading is the least desirable method as a large amount of segregation and breakage can occur.

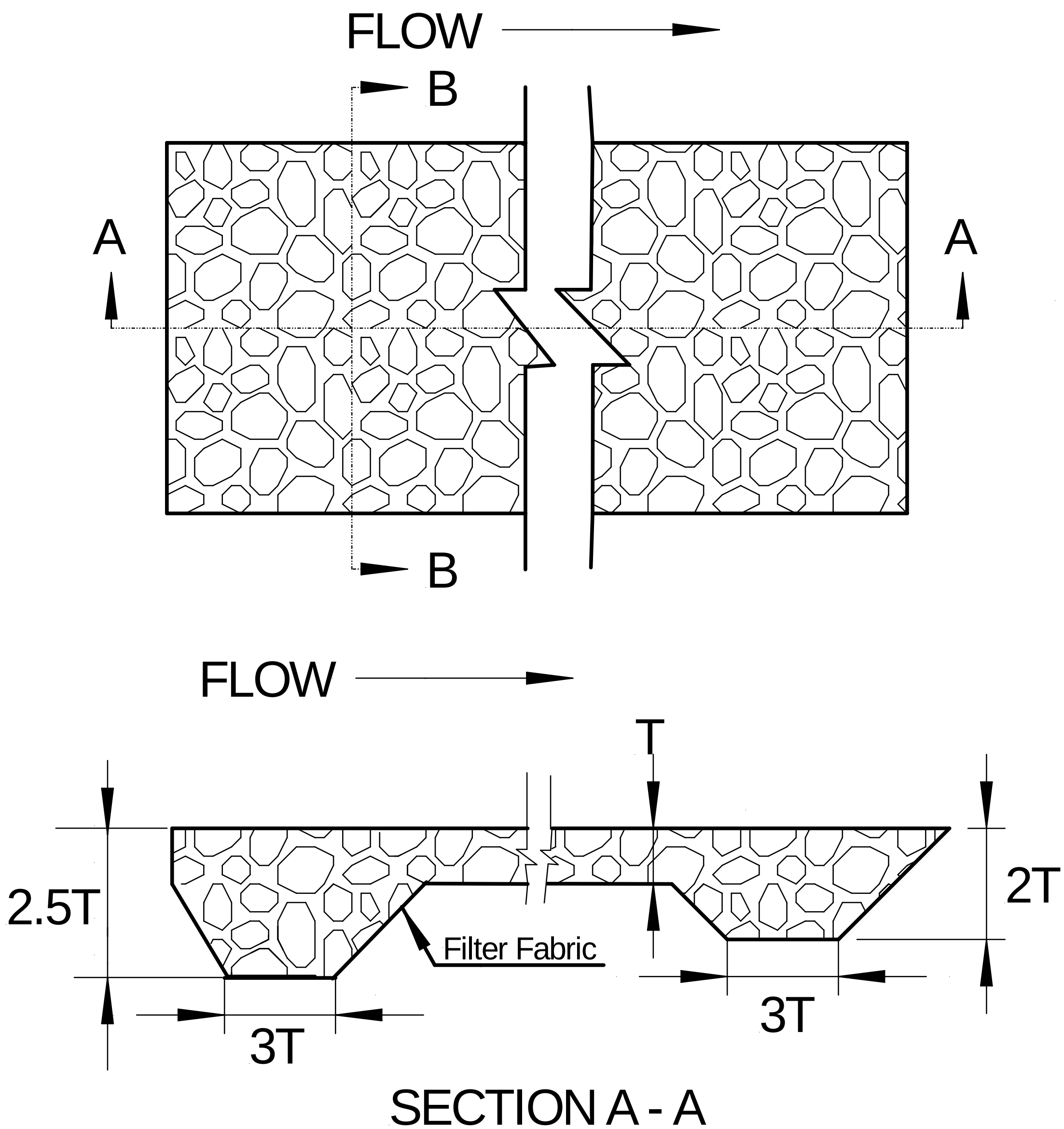


Figure 11-4 Typical Riprap Installation: Plan and Flank Details

11.7 Design Guidelines (continued)

11.7.1.7 Construction Considerations (continued)

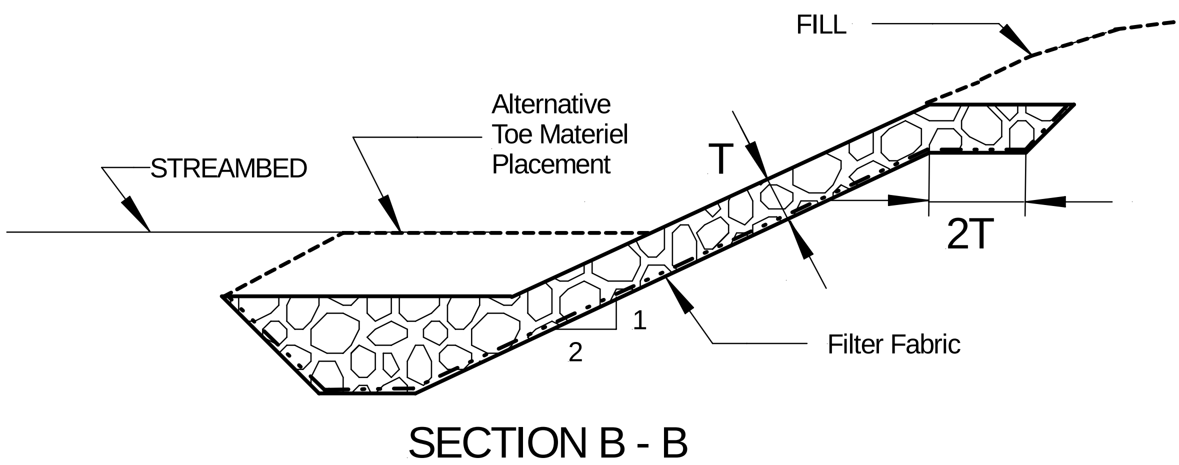


Figure 11-5 Typical Riprap Installation: End View (Bank Protection Only)

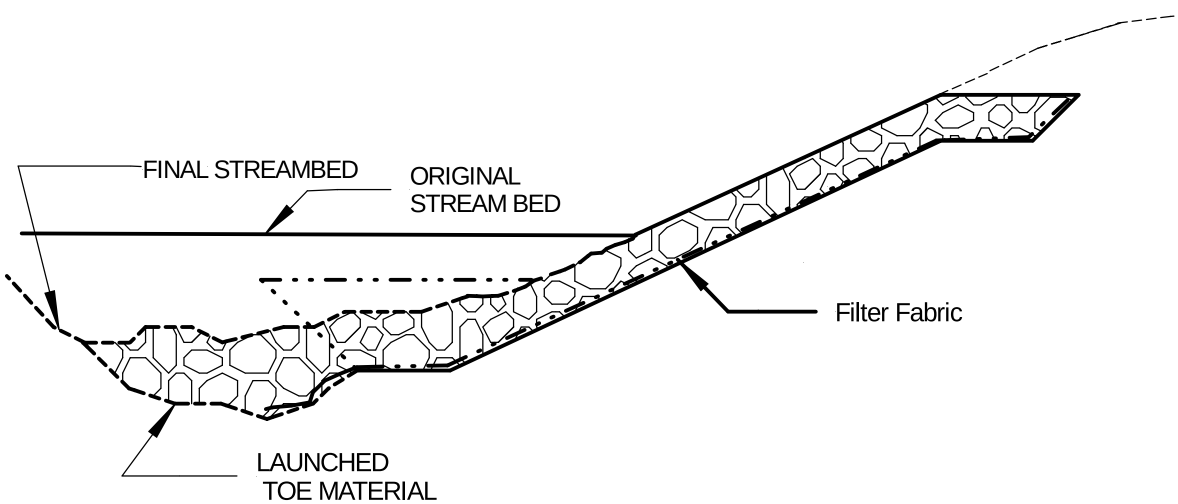


Figure 11-6 Launching Of Riprap Toe Material

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

11.7.1.8 Design Procedure

Rock riprap design procedure outlined in the following sections is comprised of three primary sections: preliminary data analysis, rock sizing and revetment detail design. The individual steps in the procedure are numbered consecutively throughout each of the sections. Figure 11-13 provides a useful format for recording data at each step of the analysis.

Preliminary Data

Step 1 Compile all necessary field data including (channel cross section surveys, soils data, aerial photographs, history of problems at site, etc.).

Step 2 Determine design discharge.

Step 3 Develop design cross section(s). Note: The rock sizing procedures described in the following steps are designed to prevent riprap failure from particle erosion.

Step 4 Compute design water surface.

(a) When evaluating the design water surface, Manning's n should be estimated. If a riprap lining is being designed for the entire channel perimeter, an estimate of the rock size may be required to determine the roughness coefficient n (Section 11.6.6).

(b) If the design section is a regular trapezoidal shape, and flow can be assumed to be uniform, use design procedures from the Open Channels Chapter.

(c) If the design section is irregular or flow is not uniform, backwater procedures must be used to determine the design water surface.

(d) Any backwater analysis conducted must be based on conveyance weighing of flows in the main channel, right bank and left bank.

Step 5 Determine design average velocity and depth.

(a) Average velocity and depth should be determined for the design section in conjunction with the computations of step 4. In general, the average depth and velocity in the main flow channel should be used.

(b) If riprap is being designed to protect channel banks, abutments, or piers located in the floodplain, average floodplain depths and velocities should be used.

11.7 Design Guidelines (continued)

11.7.1.8 Design Procedure (continued)

Step 6 Compute the bank angle correction factor K_1 (Equation 11.7, Figures 11-8 and 11-9).

Step 7 Determine riprap size required to resist particle erosion (Equation 11.6, Figure 11-7).

(a) Initially assume no corrections.

(b) Evaluate correction factor for rock riprap specific gravity and stability factor ($C = C_{sg}C_{sf}$).

(c) If designing riprap for piers or abutments see Bridge Chapter.

Step 8 Review Design. If entire channel perimeter is being stabilized, and an assumed D_{50} was used in determination of Manning's n for backwater computations, return to step 4 and repeat steps 4 through 7.

Step 9 Select final D_{50} riprap size, set material gradation (see section 11.7.1.3 and Figure 11-10), and determine riprap layer thickness (see section 11.7.1.4).

Step 10 Define limits of protection.

(a) Determine longitudinal extent of protection required (section 11.6.7).

(b) Determine appropriate vertical extent of revetment (section 11.6.7).

Step 11 Design filter layer (section 11.7.1.5, Figure 11-12).

(a) Determine appropriate filter material size, and gradation.
(b) Determine layer thickness.

Step 12 Design edge details (flanks and toe) (section 11.7.1.6).

11.7 Design Guidelines (continued)

11.7.1.7 Rock Riprap (continued)

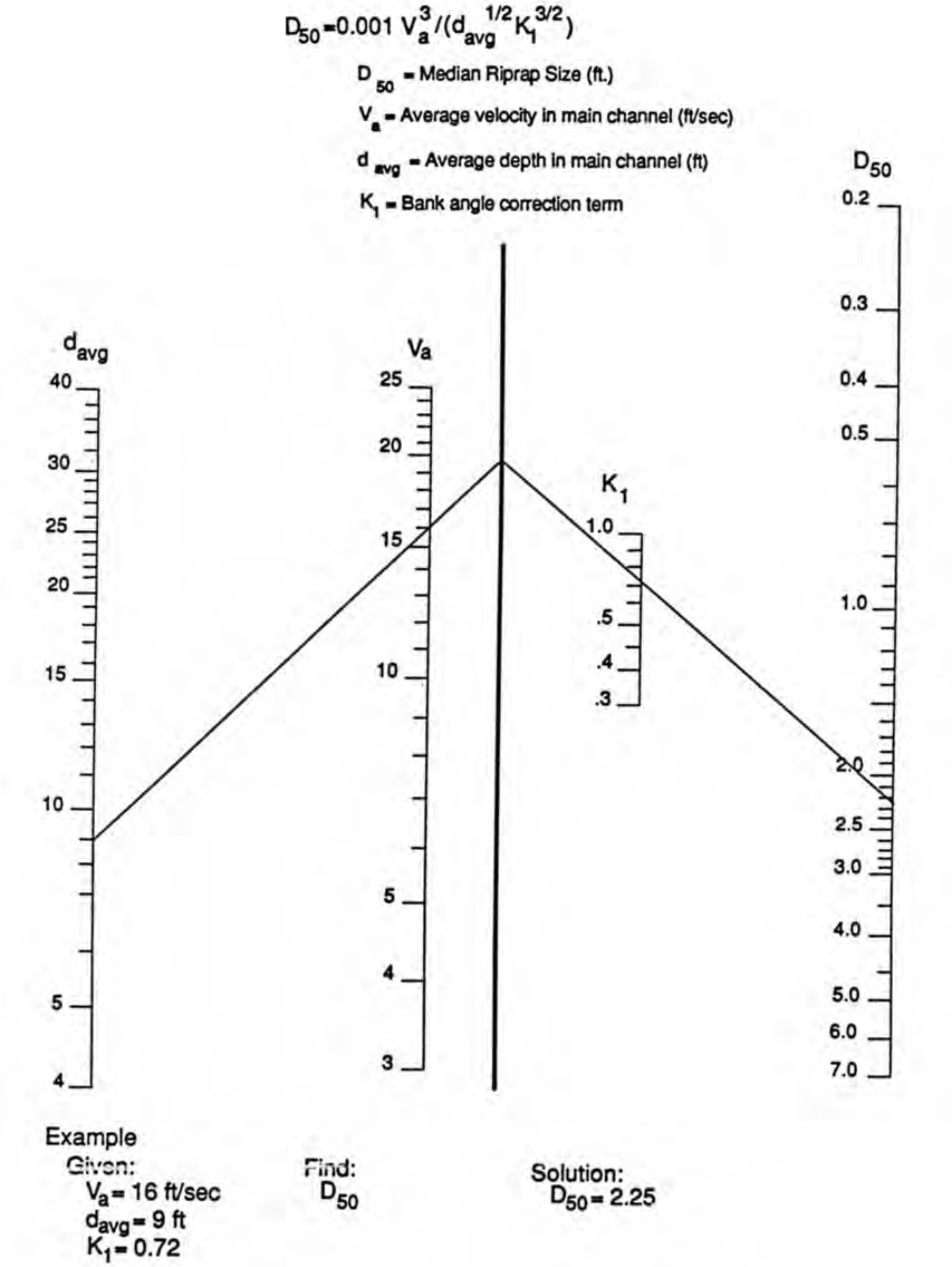


Figure 11-7 Riprap Size Relationship

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

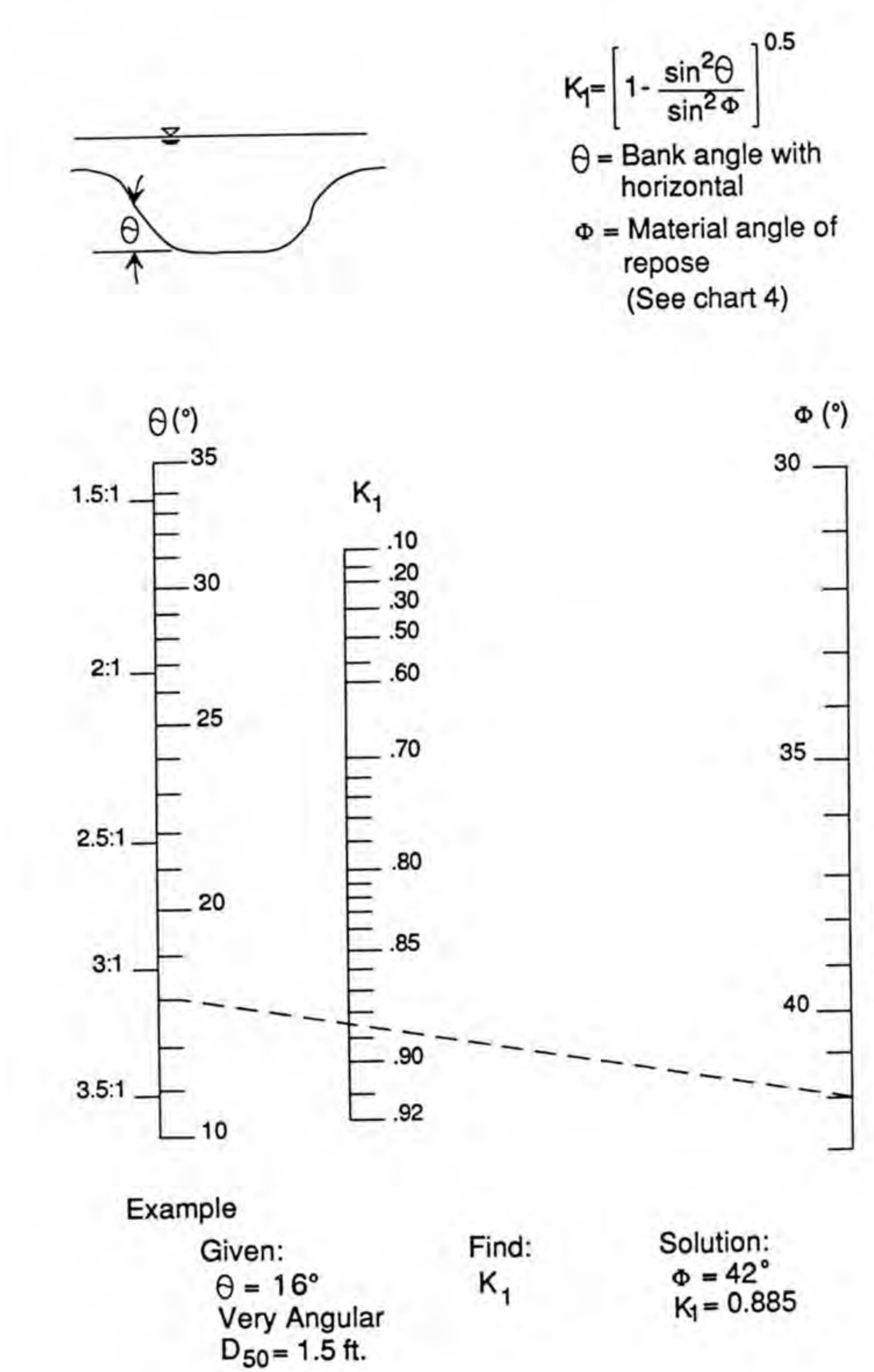


Figure 11-8 Bank Angle Correction Factor (K_1) Nomograph

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

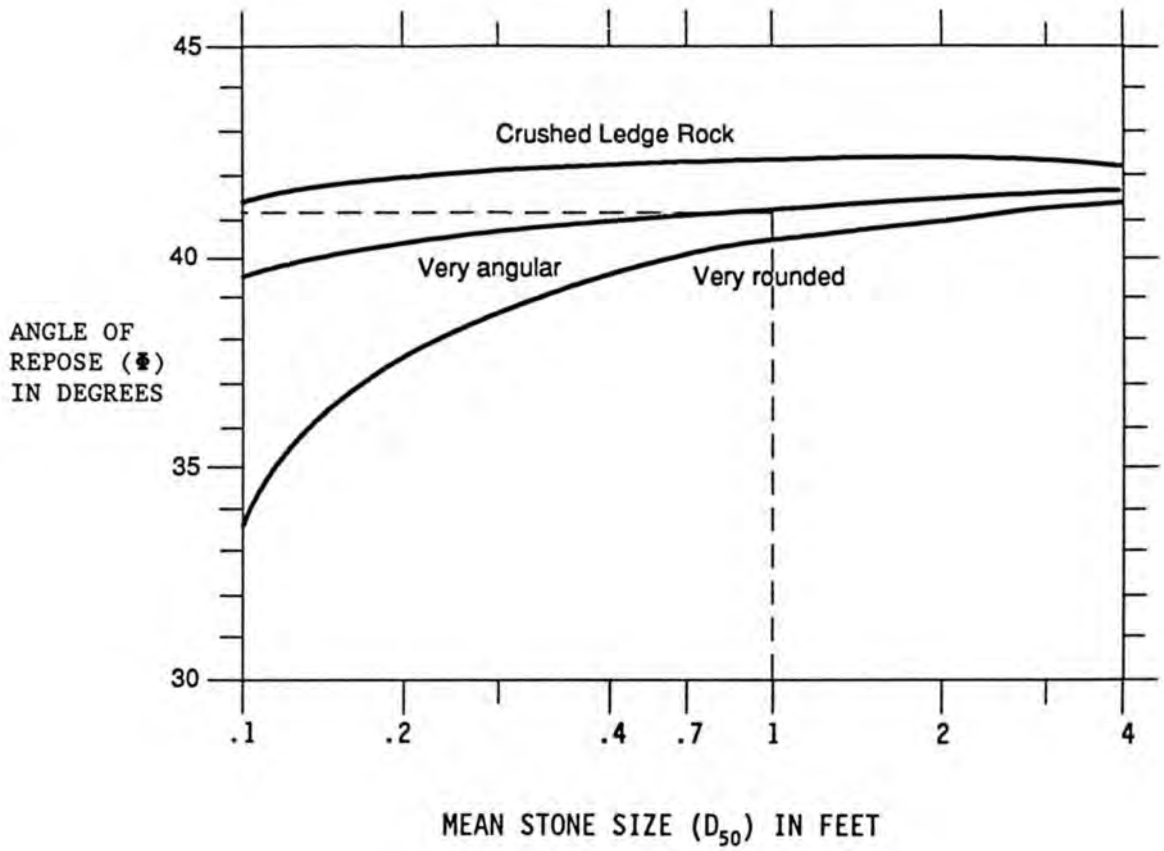


Figure 11-9 Angle Of Repose Of Riprap In Terms Of Mean Size And Shape Of Stone

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

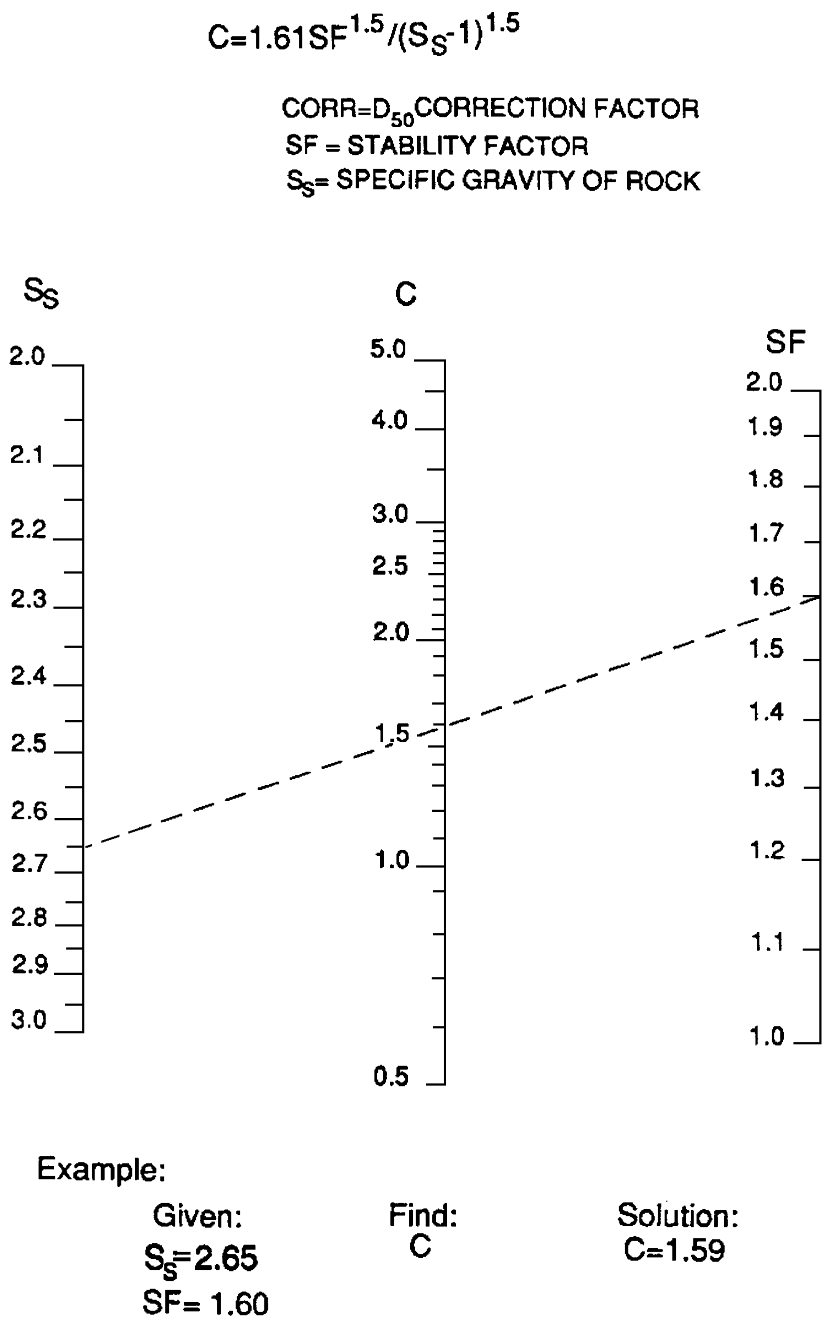


Figure 11-10 Correction Factor For Riprap Size

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

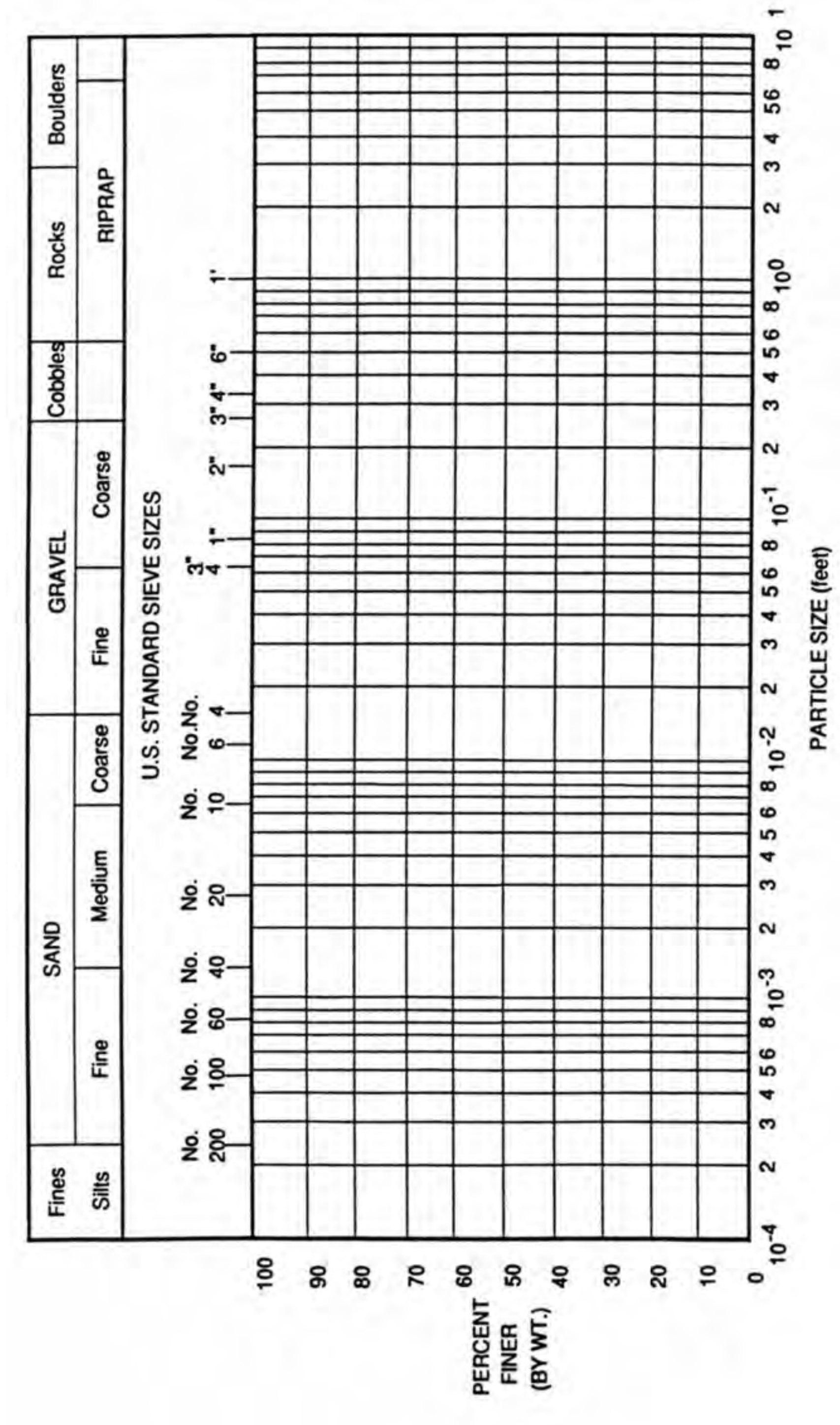


Figure 11-11 Material Gradation Chart

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

FILTER DESIGN
Project Name: ____ Project No.: __________
Subject ____ Page ___ of ______
By _________________________ Date ________ Checked By ____________________ Date ______

GRANULAR FILTER

LAYER	DESCRIPTION	D_{15}	D_{85}	RATIO:			
				D_{15}RIPRAP / D_{85} SOIL	<5	D_{15}RIPRAP / $D1_5$ SOIL	.<40
	l						

DATA SUMMARY			
LAYER DESCRIPTION	D_{15}	D_{85}	THICKNESS

FABRIC FILTER
PHYSICAL PROPERTIES CLASS ___________
HYDRAULIC PROPERTIES:
PIPING RESISTANCE , 50% PASSING #200 AOS<0.6mm
PERMEABILITY SOIL PERMEABILITY <FABRIC PERMEABILITY

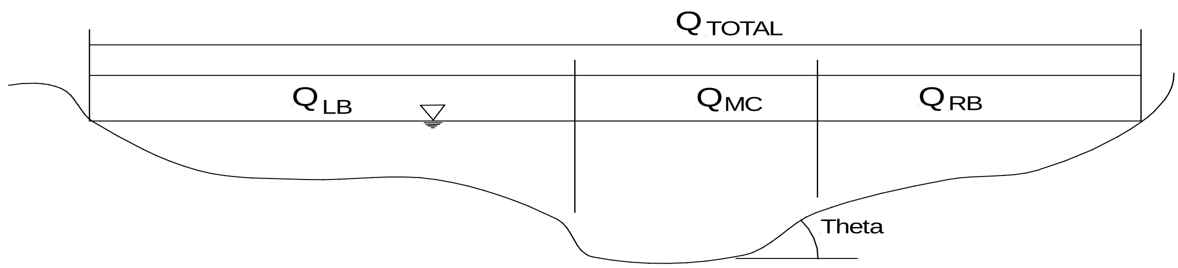
SELECTED FABRIC FILTER SPECIFICATIONS: ___________________________

Figure 11-12 Filter Design

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

Rock Riprap
Project Name: ______________________________ Project No.: __________ Subject ______________________________ Page ___ of _______ By ____________________ Date _________ Checked By ____________ Date _______



Discharge DATA			
Q_{total}	Q_{LB}	Q_{MC}	Q_{RB}

DATA SUMMARY	Bank	Bed
Area, Ft^2		
Velocity, (ft./sec.)		
Depth, d_a, ft		
Theta, **Q**		
Phi, **F**		
K_1		
D_{50}, ft		
SF		
S_g, **g**		
C		
$C_{P/A}$		
D_{50}		

RIPRAP Characteristics		
SIZE		
D_{50}		
Class		
THICKNESS		
$2D_{50}$		
D_{100}		
USE		
Gradation	Percent Finer	Size
	100	
	50	
	5-10	

FABRIC CHARACTERISTICS	
AOS <	
PERMEABILITY>	

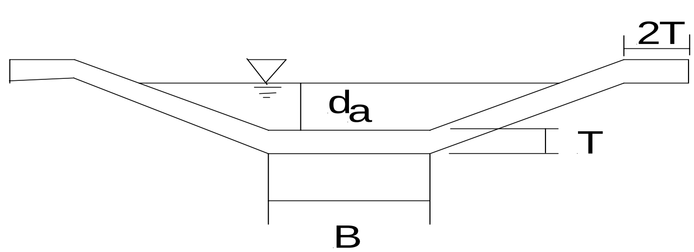


Figure 11-13 Riprap Size Form (Riprap Size Particle Erosion)

11.7 Design Guidelines (continued)

11.7.1 Rock Riprap (continued)

11.7.1.9 Design Examples

The following design examples illustrate the use of the design methods and procedures outlined above. Two examples are given; Example 1 illustrates the design of a riprap lined channel section. Example 2 illustrates the design of riprap as bank protection. In the examples, the steps correlate with the design procedure outline presented above. Computations are also shown on appropriate figures.

Example 1

A 1250 ft channel reach is to be realigned to make room for the widening of an existing highway. Realignment of the channel reach will necessitate straightening the channel and reducing its length from 1250 ft to 1000 ft. The channel is to be sized to carry 5,000 cfs within its banks. Additional site conditions are as follows:

- flow conditions can be assumed to be uniform or gradually varying;
- the existing channel profile dictates that the straightened reach be designed at a uniform slope of 0.0049;
- the natural soils are gap graded from medium sands to coarse gravels giving the following distribution.
  D_{85} = 0.105 ft, D_{50} = 0.064 ft., D_{15} = 0.0045 f t.
  K (permeability) = 3.5×10^{-2} cm/s
- Available rock riprap has a specific gravity of 2.65 and D_{50} = 12 inches.

Design a stable trapezoidal riprap lined channel for this site. Design figures used to summarize data in this example are reproduced in Figures 11-14 and 11-17.

Step 1 Compile Field Data
- See given information for this example.
- Other field data would typically include site history, geometric constraints, roadway crossing profiles, site topography, etc.

Step 2 Design Discharge
- Given as 5,000 ft^3/sec.
- Discharge in main channel equals the design discharge since entire design discharge is to be contained in channel as specified.

Step 3 Design Cross Section
- As specified, a trapezoidal section is to be designed.
- Initially assume a trapezoidal section with 20 ft bottom width and 2H: 1V side slopes (see Figure 11-14).

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

Step 4 Compute Design Water Surface
(a) Determine roughness coefficient (n = 0.04)
(b) Compute flow depth

Assume R = 7 feet

- Solve Manning's equation for normal depth.
  $Q = (1/n)\ A\ R^{2/3}\ S^{1/2}$
  d = 11.8 ft.
- Compute hydraulic radius to compare with the assumed value used in Step 4(a) (use computer programs, available charts and tables, or manually compute).
  $R = A/P$
  $R = 514.5/72.8$
  R = 7.1 ft. which is approximately equal to R (assumed) therefore,
  d = 11.8 ft OK

Step 5 Determine Design Parameters

$A = 11.8(11.8(4)+20+20)/2= 514.5\ ft^2$
$V_a = Q/A = 5000/514.5 = 9.7$ ft/sec
$d_a = d = 11.8$ (uniform channel bottom)
$K_1 = 1$ for bed

Step 6 Bank Angle Correction Factor

Θ = 2H: 1V
$\Phi = 41^o$ (from Figure 11-11)
$K_1 = 0.73$ (from Figure 11-10)

Step 7 Determine riprap size (see section 11.7.1.8)

(a) Using Figure 11-15
for channel bed $D_{50} = 0.28$ ft
for channel bank $D_{50} = 0.43$ ft.

(b) Riprap specific gravity = 2.65 (given)
(uniform flow, little or no uncertainty in design)
Stability factor = 1.2 (Table 11-3)
C = 1 (Figure 11-10)

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

(c) no piers or abutments to evaluate for this example, therefore
$C_{p/a} = 1$

(d) Corrected riprap size:
For channel bed
$D'_{50} = D_{50} = 0.28$ ft.
For channel banks
$D'_{50} = D_{50} = 0.43$ ft.

Step 8 Not applicable

Step 9 Select Design Riprap Size, Gradation and Layer Thickness

D_{50} size: Recommend AASHTO Face Class riprap, table 11-5
$D_{50} = 0.95$ ft. (for entire perimeter)

Layer thickness (T):
$T = 2\ D_{50} = 2(0.95) = 1.9$ ft., or $T = D_{100} = 1.3$ ft.
Use T = 2.0 ft

Step 10 Define limits of protection.
Longitudinal Extent of Protection

Riprap lining to extend along entire length of straightened reach plus some additional upstream and downstream distance.

Vertical Extent of Protection

Riprap entire channel perimeter to top-of-bank.

Step 11 Filter Layer Design

(a) Filter material size:

$$<5 \quad \frac{D_{15}\text{ [coarser layer]}}{D_{85}\text{ [finer layer]}} \quad \text{and} \quad \frac{D_{15}\text{ [coarser layer]}}{D_{15}\text{ [finer layer]}} < 40$$

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

For the riprap to soil interface:

$$\frac{D_{15}\,[\text{riprap}]}{D_{85}\,[\text{soil}]} = \frac{0.6}{0.105} = 6 > 5$$

and

$$\frac{D_{15}\,[\text{riprap}]}{D_{15}\,[\text{soil}]} = \frac{0.6}{0.0045} = 133 > 40$$

Therefore, a filter fabric is needed.

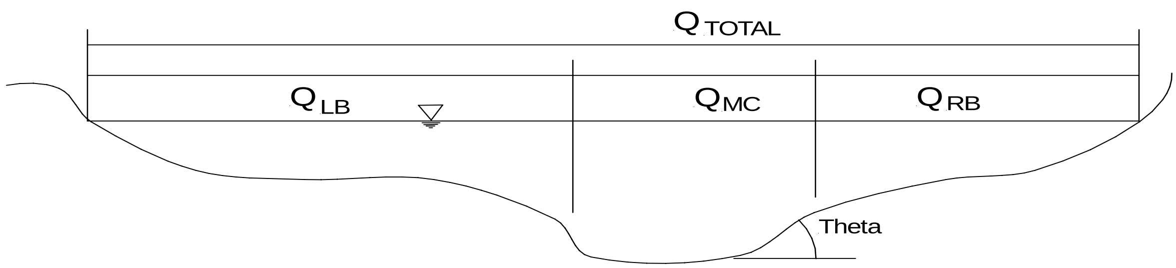
Step 12 Edge Details

Line entire perimeter; edge details as per Figure 11-6 (also see sketch on Figure 11-18).

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

Rock Riprap
Project Name: __Design Example____ Project No.: _ADT064____
Subject _____Channel Lining____ _ Page _1_ of ________
By ______________________________________ Date __________ Checked By _______________ Date ________



Discharge DATA			
Q_{total}	Q_{LB}	Q_{MC}	Q_{RB}
5,000 cfs			

DATA SUMMARY	Bank	Bed
Area, Ft^2	--	514.5
Velocity, (ft./sec.)	--	9.7
Depth, d_a, ft	--	11.8
Theta, **Q**	26.5 (2:1)	--
Phi, F	41	41
K_1	.73	1.0
D_{50}, ft	0.43	0.28
SF	1.2	1.2
S_g, g	2.65	2.65
C	1	1
$C_{P/A}$	n/a	n/a
D_{50}	0.43	0.28

RIPRAP Characteristics		
SIZE		
D_{50}	0.95	
Class	Facing	
THICKNESS		
$2D_{50}$	1.90	
D_{100}	1.30	
USE:	2.0	
Gradation	Percent Finer	Size
	100	1.30
	50	0.95
	5-10	0.40

FABRIC CHARACTERISTICS	
AOS <	
PERMEABILITY >	

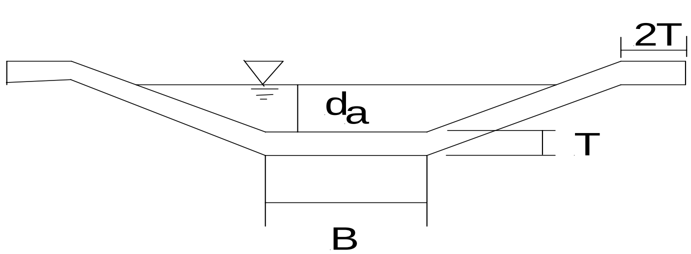


Figure 11-14 Riprap Size Form (Example 1)

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

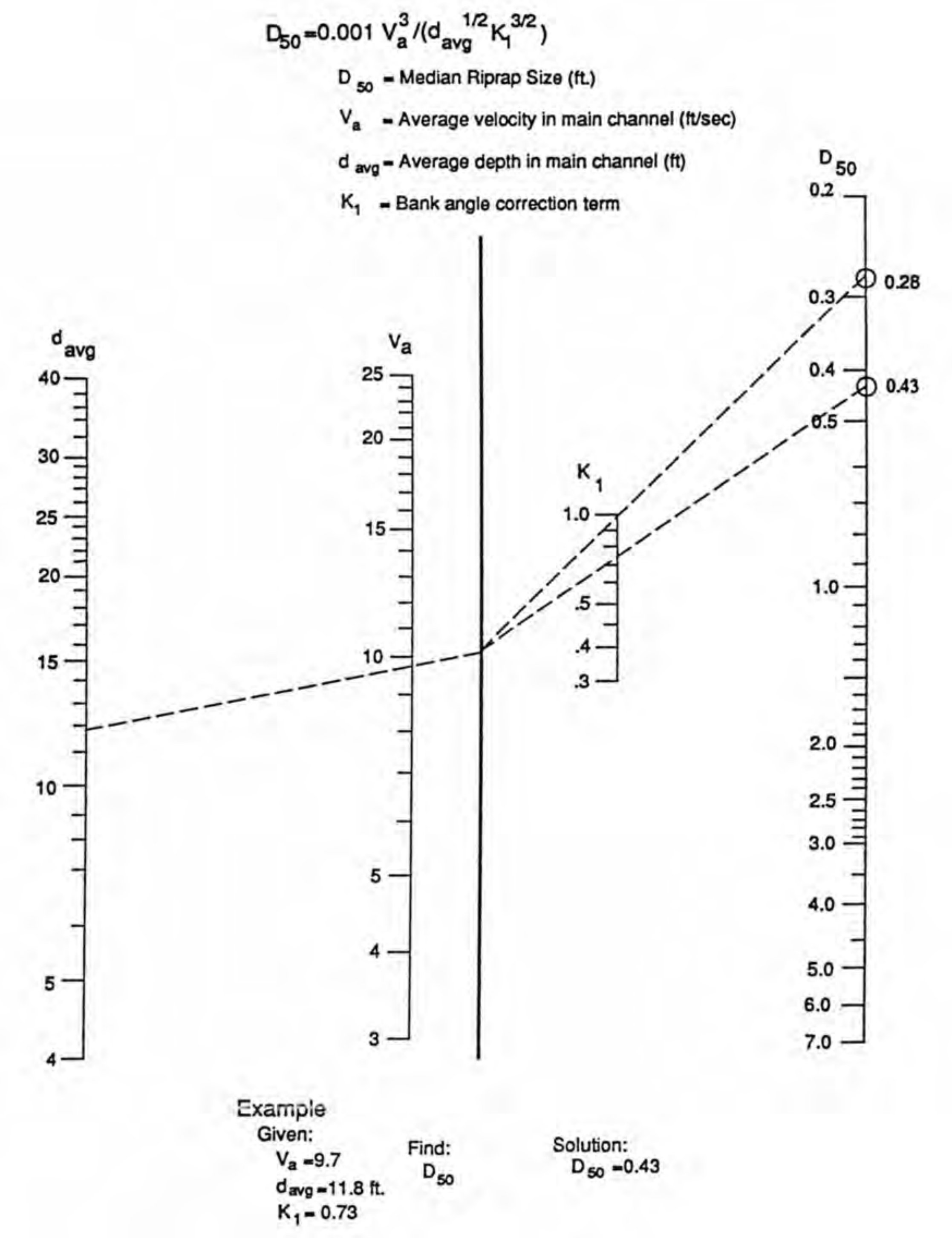


Figure 11-15 Riprap Size Relationship (Example 1, Step 7)

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

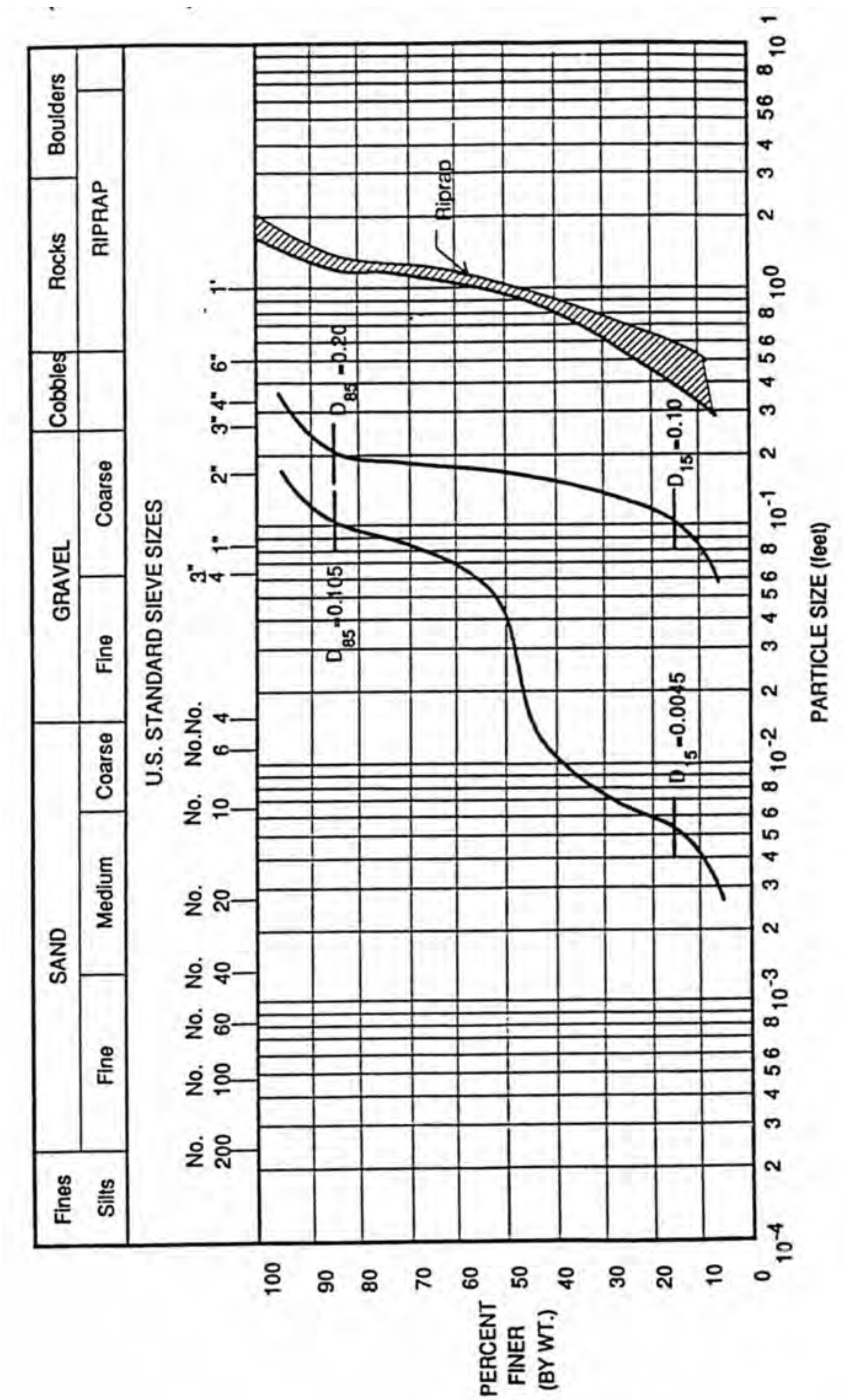


Figure 11-16 Material Gradation

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

FILTER DESIGN
Project Name: ____________________ Project No.: ____________
Subject ____________________ Page ___ of _______
By ____________ Date _________ Checked By ____________ Date ________

GRANULAR FILTER

LAYER	DESCRIPTION	D_{15}	D_{85}	RATIO:			
				$\frac{D_{15} RIPRAP}{D_{85} SOIL}$	<5	$\frac{D_{15} RIPRAP}{D1_5 SOIL}$	<40
	Riprap	0.60					
	Soil	0.0045	0.105				
				6	No	133	No

DATA SUMMARY			
LAYER DESCRIPTION	D_{15}	D_{85}	THICKNESS
Riprap	0.60		
Soil	0.0045	0.105	

FABRIC FILTER
PHYSICAL PROPERTIES CLASS ____________
HYDRAULIC PROPERTIES:
PIPING RESISTANCE , 50% PASSING #200 AOS<0.6mm
PERMEABILITY SOIL PERMEABILITY <FABRIC PERMEABILITY

SELECTED FABRIC FILTER SPECIFICATIONS: ______________________________

Figure 11-17 Filter Design

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

Example 2

The site illustrated in Figure 11-18 and discussed below is migrating laterally towards Route 1. Design a riprap revetment to stabilize the active bank erosion at this site.

The process of developing an appropriate channel geometry is illustrated in Figure 11-18. Figure 11-18 illustrates the location of the design site at position "2" along Route 1. The section illustrated in Figure 11-18c was surveyed at this location, and represents the current condition. No previous channel surveys were available at this site. However, data from several old surveys were available in the vicinity of a railroad crossing upstream (location 1). Figure 11-18b illustrates these survey data. The surveys indicate that there is a trend for the thalweg of the channel to migrate within the right half of the channel. Since location 1 and 2 are along bends of similar radii, it can be reasonably assumed that a similar phenomenon occurs at location 2. A thalweg located immediately adjacent to the channel bank reasonably represents the worst case hydraulically for the section at location 2. Therefore, the surveyed section at location 2 is modified to reflect this. In addition, the maximum section depth (located in the thalweg) is increased to reflect the effect of stabilizing the bank. The maximum depth in the thalweg is set to 1.7 times the average depth of the original section (note that it is assumed that the average depth before modification of the section is the same as the average depth after modification). The final modified section geometry is illustrated in Figure 11-18c.

Additional site conditions are as follows:

- flow conditions are gradually varying;
- channel characteristics are as described above;
- topographic survey indicates:
  - channel slope = 0.0024 ft/ft
  - channel width = 300 ft
  - bend radius = 1200 ft
- channel bottom is armored with cobble size material having a D_{50} of approximately 0.5 ft;
- bank soils are silty sand with the following soil characteristics:
  D_{85} = 0.0042 ft.
  D_{50} = 0.0015 ft.
  D_{15} = 0.00045 ft
  K (permeability) = 1.0×10^{-4} cm/s
- available rock riprap has a specific gravity of 2.60, and is described as angular;
- field observations indicate that the banks are severely cut just downstream of the bend apex; erosion was also observed downstream to the bend exit and upstream to the bend quarter points;
- bank height along cut banks is approximately 9 ft.

Design Figures used to summarize data in this example are reproduced in Figures 11-20 and 11-22.

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

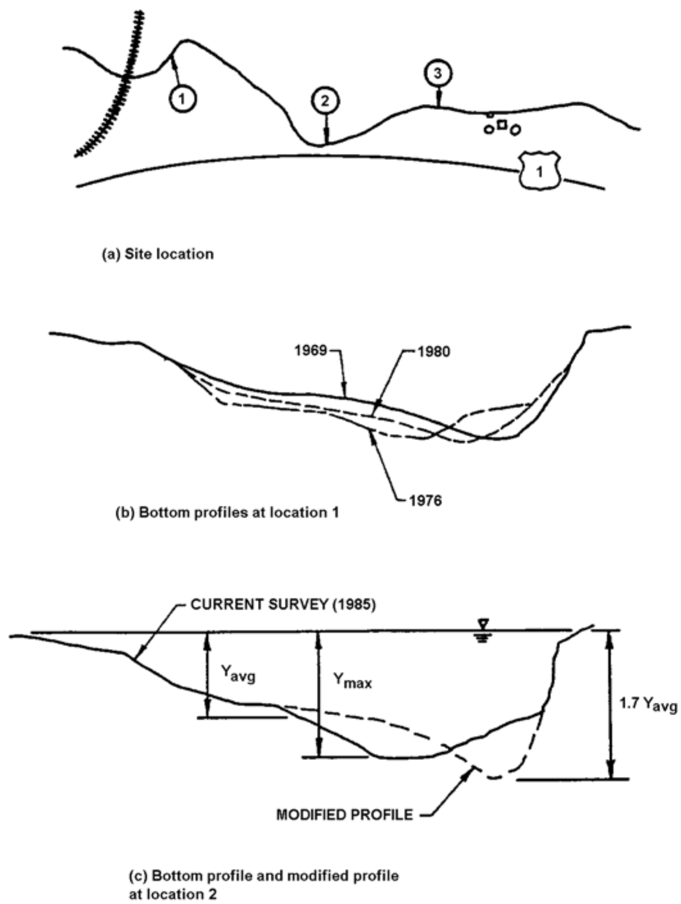


Figure 11-18 Channel Geometry Development (Example 2)

Step 1 Compile Field Data

- See given information for this example.
- See site history given above.

Step 2 Design Discharge

- Given as 46,700 ft^3/s.
- From backwater analysis of this reach, it is determined that the discharge confined to the main channel (Q_{mc}) is 34,700 ft^3/s.

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

Step 3 Design Cross Section

•Only the channel bank is to be stabilized; therefore, the channel section will consist of the existing channel with the bank graded to an appropriate angle to support the riprap revetment. Figure 11-18 illustrates the existing channel section.

- To minimize loss of bank vegetation, and limit the encroachment of the channel on adjacent lands, a 2H:1V bank slope is to be used.

- As given, the current bank height along the cut banks is 9 ft.

Step 4 Compute Design water Surface

(a) Determine roughness coefficient (n = 0.042).
This represents the average reach "n" used in the backwater analysis.

(b) Compute flow depth
•Flow depth determined from backwater analysis. The maximum main channel depth was determined to be: d_{max} = 15 ft.

Hydraulic radius for main channel
R = 10.4 ft (from backwater analysis)
R assumed (10 ft) is approximately equal to R actual, therefore, "n" as computed is OK.

Step 5 Determine Other Design Parameters

From backwater analysis: (all main channel values)
A = 2750 ft^2
V_a = 12.6 ft/s
d_a = d = 12.0 ft

Step 6 Bank Angle Correction Factor

Q = 2:1
$\Phi = 41^o$
K_1 = 0.73 (from Figure 11-6)

Step 7 Determine riprap size

(a) Using Figure 11-7
D_{50} = 0.9 ft.

(b) Riprap specific gravity = 2.60 (given)

Stability factor = 1.6 (gradually varying flow, sharp bend — bend radius to width = 4)
C = 1.6

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

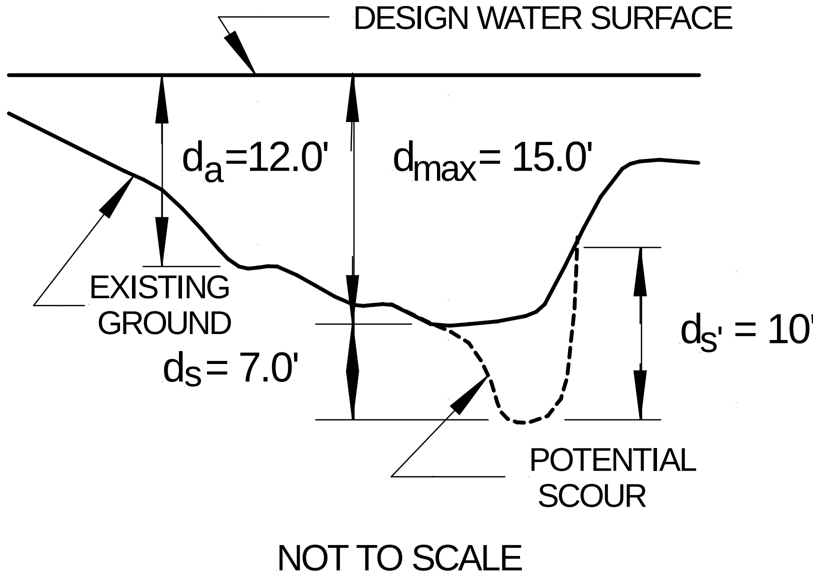


Figure 11-19 Channel Cross Section For Example 2 Illustrating Flow And Potential Scour Depths

(c) no piers or abutments to evaluate for this example, therefore
$C_{p/a} = 1$

(d) Corrected riprap size

$D'_{50} = D_{50}(1.6)(1.0) = 1.44$ ft

Step 8 Not applicable

Step 9 Select Design Riprap Size, Gradation and Layer Thickness (Preliminary Design of Waterway Area)

D_{50} size: Recommend AASHTO ¼ ton class riprap
$D_{50} = 1.8$ ft
Gradation: See Figure 11-21
Layer thickness (T):
$T = 2\ D_{50} = 2(1.8) = 3.6$ ft, or $T = D_{100} = 2.25$ ft, Use T = 3.6 ft

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

Step 10 Limits of Protection

(a) Longitudinal Extent of Protection

Field observations indicate that the banks are severely cut just downstream of the bend apex; erosion was also observed downstream to the bend exit and upstream to the bend quarter points. Therefore, establish longitudinal limits of protection to extend to a point 300 ft (W) upstream of the bank entrance, and to a point 450 ft (1.5 W) downstream of the bend exit.

(b) Vertical Extent of Protection

Riprap entire channel bank from top-of-bank to below depth of anticipated scour. Scour depth evaluated as illustrated in section 11.6.7.2:

$d_s = 6.5\ D_{50}^{-0.11}$ (Equation 11.3)
$d_s = 6.5\ (0.5)^{-0.11} = 7.0$ ft.

Adding this to the observed maximum depth yields a potential maximum scour depth of:
15.0+7.0 = 22.0 ft

The bank material should be run to this depth, or a sufficient volume of stone should be placed at the bank toe to protect against the necessary depth of scour.

Step 11 Filter Layer Design

(a) Filter material size: (Figure 11-21)

$$5 < \frac{D_{15}\ [\text{coarser layer}]}{D_{15}\ [\text{finer layer}]} < 40$$

For the riprap to soil interface:

$$\frac{D_{15}\ [\text{riprap}]}{D_{85}\ [\text{soil}]} = \frac{0.5}{0.0042} = 119 > 5 \text{ and}$$

$$\frac{D_{15}\ [\text{riprap}]}{D_{15}\ [\text{soil}]} = \frac{0.5}{0.00045} = 1111 > 40$$

Therefore, a filter fabric is needed.

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

Step 12 Edge Details

(a) Flank details: See Figure 11-23
(b) Toe detail: See Figure 11-23

Anticipated scour depth below existing channel bottom at the bank (d'_s) is the depth of scour (computed in step 10) minus the current bed elevation at the bank (see Figure 11-21): 22 ft– 12 ft= 10 ft

Rock quantity required below the existing bed:

$R_q = d'_s(\sin^{-1}\Theta)(T)(1.5)$ (1.5)

Where: R_q = required riprap quantity per foot of bank, ft^2
Θ = the bank angle with the horizontal, degrees
T = the riprap layer thickness, ft

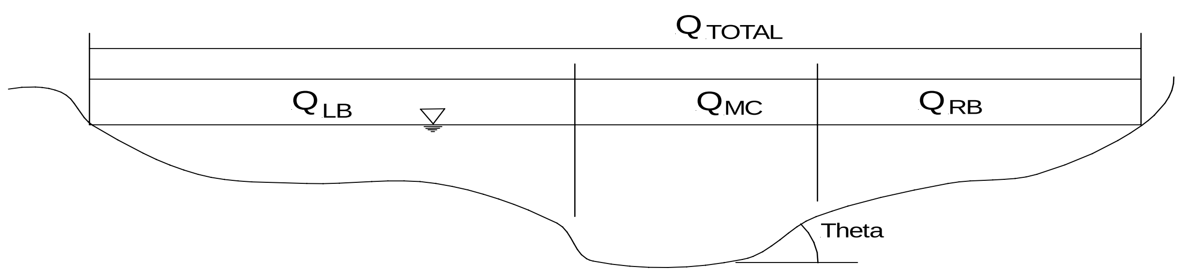
$R_q = (10)(2.24)(3)(1.5) = 101\ ft^2$

A 6 ft deep trapezoidal toe trench with side slopes of 2H: 1V and 1H: 1V, and a bottom width of 6 ft contains the necessary volume. Figure 11-23 illustrates the resulting toe trench detail.

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

Rock Riprap
Project Name: ____ Project No.: ___________
Subject ____ Page ___ of _______
By _________________________________ Date _________ Checked By ______________ Date _______



Discharge DATA			
Q_{total}	Q_{LB}	Q_{MC}	Q_{RB}

DATA SUMMARY	Bank	Bed
Area, Ft^2		
Velocity, (ft./sec.)		
Depth, d_a, ft		
Theta, **Q**		
Phi, **F**		
K_1		
D_{50}, ft		
SF		
S_g, **g**		
C		
$C_{P/A}$		
D_{50}		

RIPRAP Characteristics		
SIZE		
D_{50}		
Class		
THICKNESS		
$2D_{50}$		
D_{100}		
USE		
Gradation	Percent Finer	Size
	100	
	50	
	5-10	

FABRIC CHARACTERISTICS	
AOS <	
PERMEABILITY>	

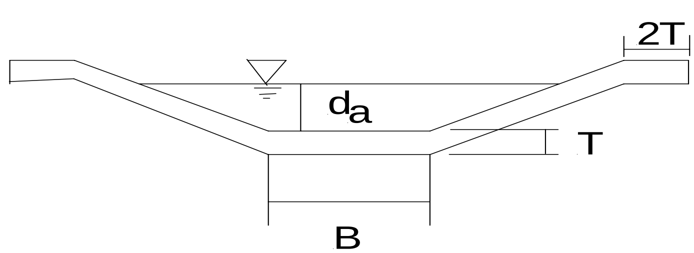


Figure 11-20 Riprap Size Form Example 2

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

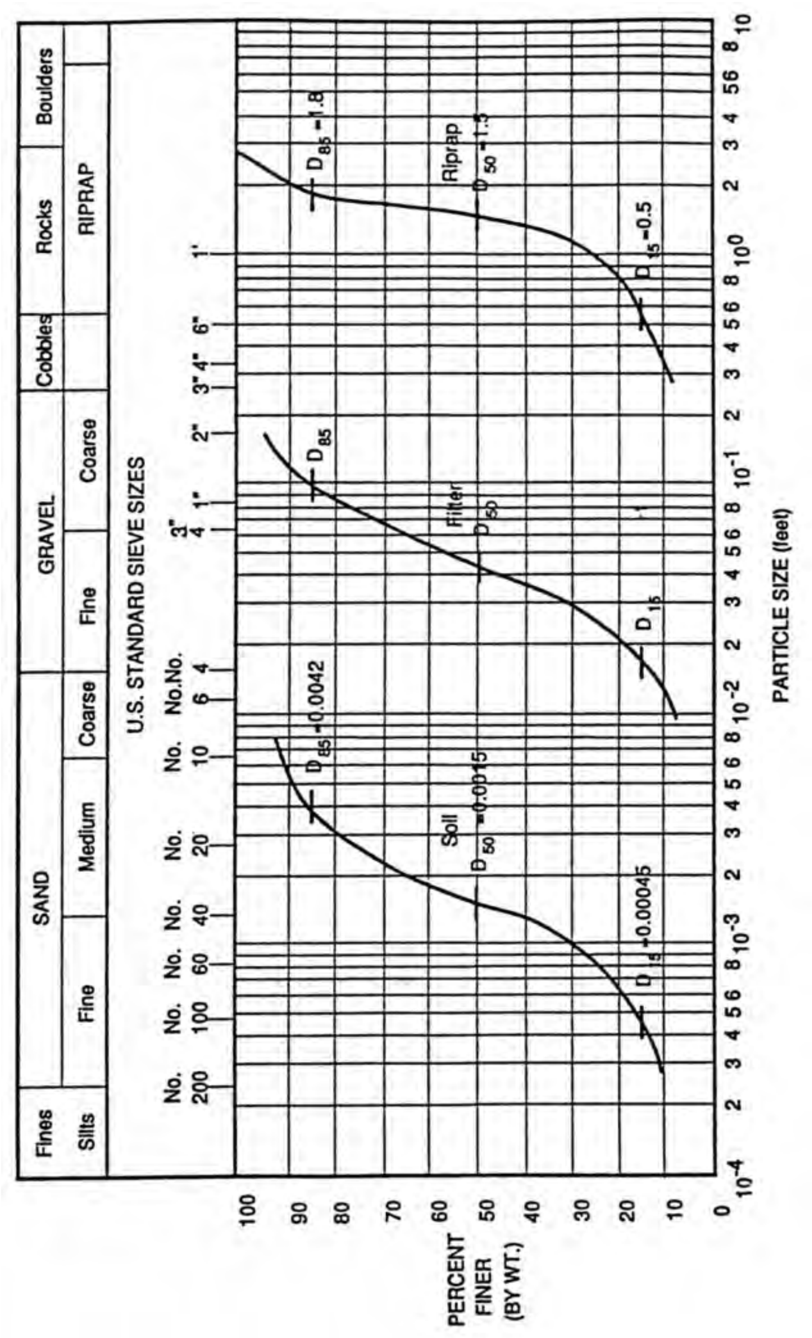


Figure 11-21 Material Gradation, Example 2 (Natural Soils)

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

FILTER DESIGN
Project Name: ____ Project No.: __________
Subject ____ Page ___ of _______
By _________________________ Date _________ Checked By _____________________ Date ________

GRANULAR FILTER

LAYER	DESCRIPTION	D_{15}	D_{85}	RATIO:			
				$\frac{D_{15} RIPRAP}{D_{85} SOIL}$	<5	$\frac{D_{15} RIPRAP}{D_{15} SOIL}$	<40
	Riprap	0.60					
	Soil	0.0045	0.105				
				6	No		

DATA SUMMARY			
LAYER DESCRIPTION	D_{15}	D_{85}	THICKNESS
Riprap	0.60		
Soil	0.0045	0.105	

FABRIC FILTER
PHYSICAL PROPERTIES CLASS ____________
HYDRAULIC PROPERTIES:

PIPING RESISTANCE, 50% PASSING #200 AOS<0.6mm
PERMEABILITY SOIL PERMEABILITY <FABRIC PERMEABILITY

SELECTED FABRIC FILTER SPECIFICATIONS: ____________________________

Figure 11-22 Filter Design, Example 2

11.7 Design Guidelines (continued)

11.7.1 Design Examples (continued)

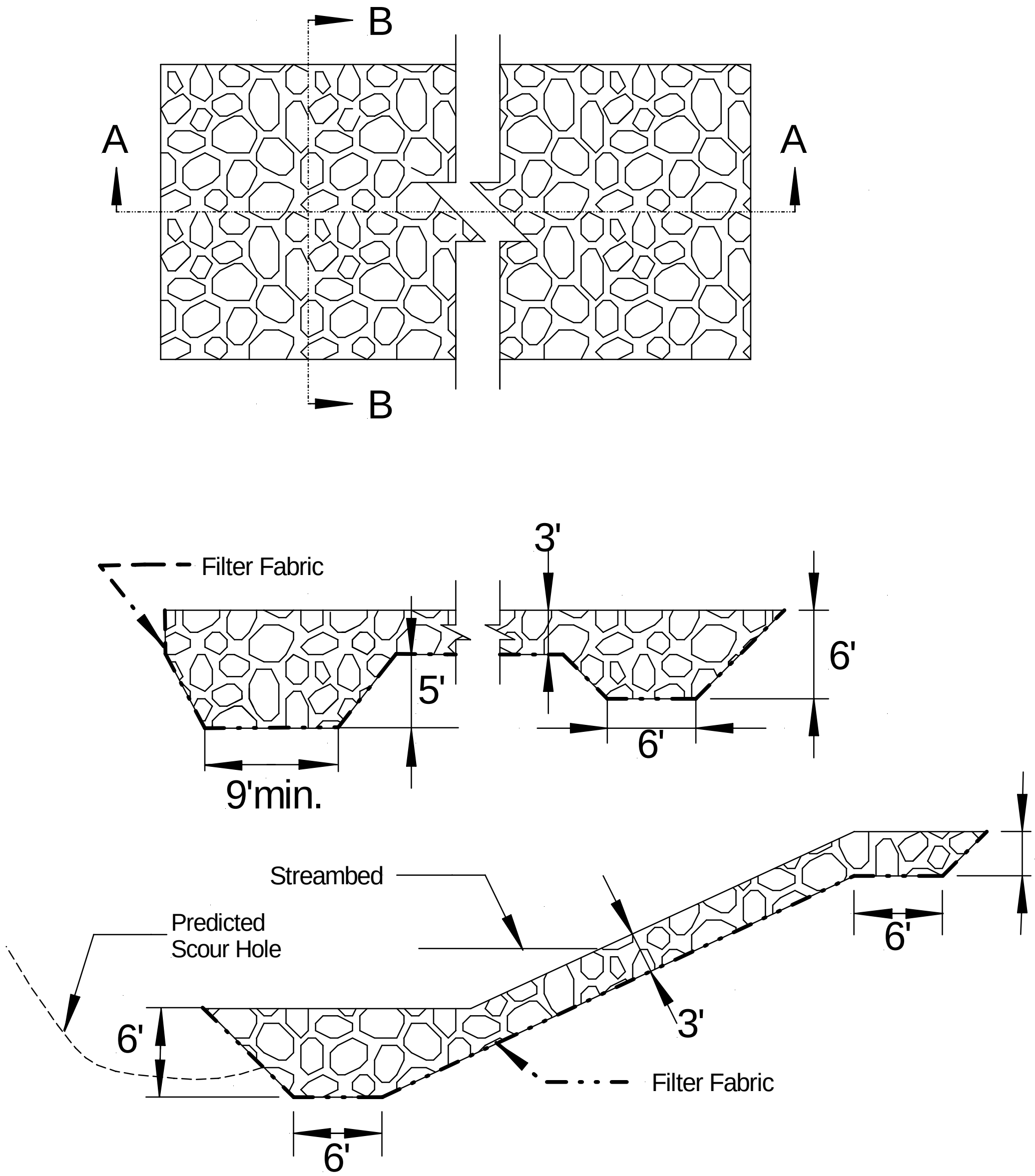


Figure 11-23 Toe And Flank Details (Example 2)

11.7 Design Guidelines (continued)

11.7.2 Wire-Enclosed Rock

As described in Section 11.5.1, wire-enclosed rock (gabion) revetments consist of rectangular wire mesh baskets filled with rock. The most common types of wire-enclosed revetments are mattresses and stacked blocks. The wire cages that make up the mattresses and gabions are available from commercial manufacturers. If desired, the wire baskets can also be fabricated from available wire fencing materials.

Rock and wire mattress revetments consist of flat wire baskets or units filled with rock that are laid end to end and side to side on a prepared channel bed and/or bank. The individual mattress units are wired together to form a continuous revetment mattress. They are commonly used on flatter slopes, less than 2:1, or as aprons. Commercial sizes are usually 6, 9 or 12 inches in thickness and 6, 9 or 12 feet wide.

Stacked block gabion revetments consist of rectangular wire baskets that are filled with stone and stacked in a stepped-back fashion to form the revetment surface. They are also commonly used at the toe of embankment slopes as toe walls that help to support other upper bank revetments and prevent undermining. The rectangular basket or gabion units used for stacked configurations are of more uniform dimensions than those typically used for mattress designs. That is, they typically have a square cross section. Commercially available gabions used in stacked configurations are available in various sizes but the most common have a 3 ft width and thickness.

11.7.2.1 Design Guidelines For Mattresses

Components of a rock and wire mattress design include layout of a general scheme or concept, bank and foundation preparation, mattress size and configuration, stone size, stone quality, basket or rock enclosure fabrication, edge treatment and filter design. Design guidance is provided below in each of these areas.

General

Rock and wire mattress revetments can be constructed from commercially available wire units or from available wire fencing material. The basic elements of a mattress is illustrated in Figure 11-24. Rock and wire mattress revetments can be used to protect either the channel bank, Figure 11-25, or the entire channel perimeter, Figure 11-26.

When used for bank protection, Figure 11-25, rock and wire mattress revetments consist of two distinct sections: a toe section and upper bank paving. As illustrated, a variety of toe designs can be used. The vertical and longitudinal extent of the mattress should be based on guidelines provided in section 11.6.7. Emphasis in design should be placed on toe design, and filter design. These designs are detailed later.

11.7 Design Guidelines (continued)

11.7.2.1 Design Guidelines For Mattresses (continued)

Bank and Foundation Preparation

Channel banks should be graded to a uniform slope. The graded surface, either on the slope or on the stream bed at the toe of the slope on which the rock and wire mattress is to be constructed, should not deviate from the specified slope line by more than 6 inches. All blunt or sharp objects (such as rocks or tree roots) protruding from the graded surface should be removed.

Mattress Unit Size and Configuration

Individual mattress units should be a size that is easily handled on site. Commercially available gabion units come in standard sizes as indicated in Table 11-6. Manufacturer's literature indicates that alternative sizes can be manufactured when required, provided that the quantities involved are of a reasonable magnitude. The mattress should be divided into compartments so that failure of one section of the mattress will not cause loss of the entire mattress. Compartmentalization also adds to the structural integrity of individual gabion units. It is recommended that diaphragms be installed at a nominal of 3 ft spacing within each of the gabion units to provide the recommended compartmentalization, Figure 11-24. Wire mattress units should be limited to slopes no steeper than 2:1.

Table 11-6 Standard Gabion Sizes

Thickness (ft)	Width (ft)	Length (ft)	Wire-Mesh Opening Size (in. × in.)
0.75	6	9	2.5 x 3.25
0.75	6	12	2.5 × 3.25
1.0	3	6	3.25 × 4.5
1.0	3	9	3.25 × 4.5
1.0	3	12	3.25 × 4.5
1.5	3	6	3.25 × 4.5
1.5	3	9	3.25 × 4.5
1.5	3	12	3.25 × 4.5
3.0	3	6	3.25 × 4.5
3.0	3	9	3.25 × 4.5
3.0	3	12	3.25 × 4.5

11.7 Design Guidelines (continued)

11.7.2.1 Design Guidelines For Mattresses (continued)

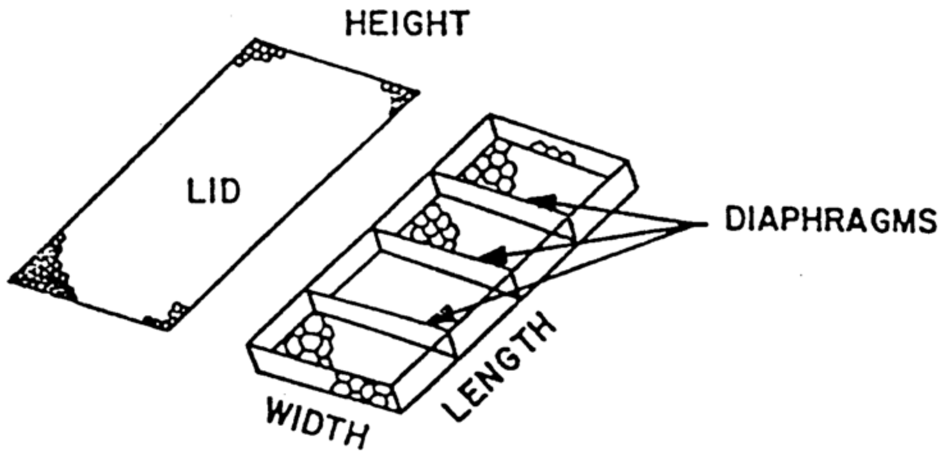


Figure 11-24 Mattress Configuration

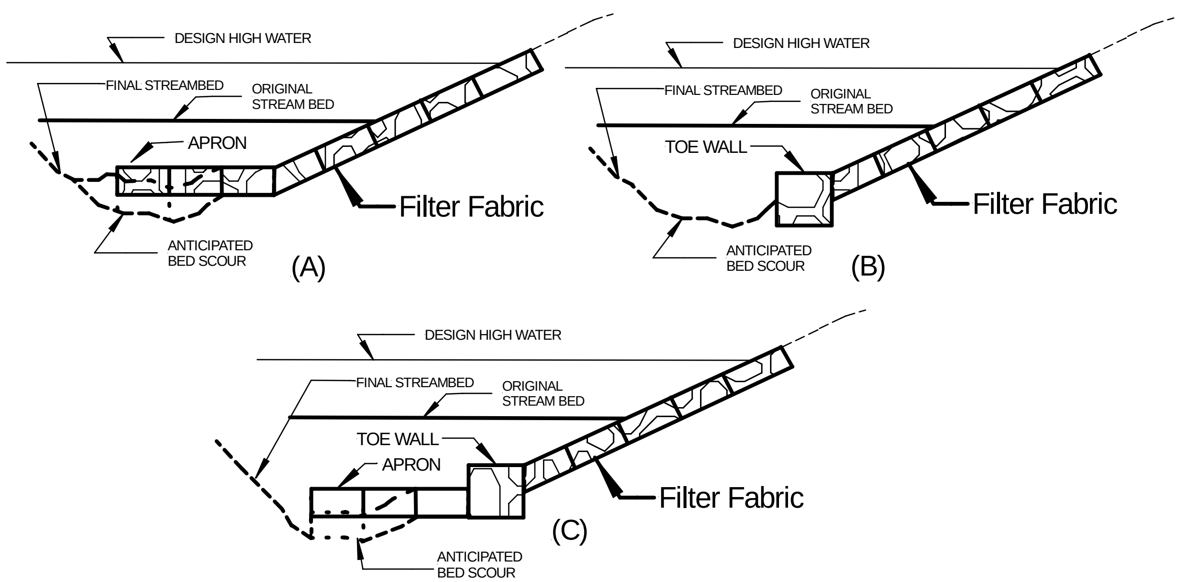


Figure 11-25 Rock And Wire Mattress Configuration: (a) mattress with toe apron; (b) mattress with toe wall; and (c) mattress with toe wall and apron

11.7 Design Guidelines (continued)

11.7.2.1 Design Guidelines For Mattresses (continued)

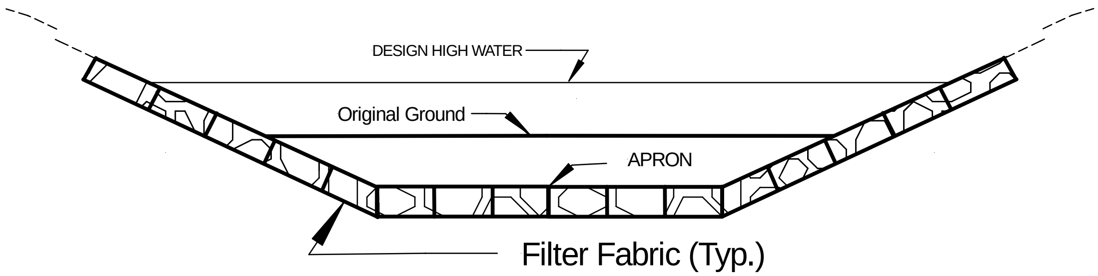


Figure 11-26 Wire-Tied Rock Mattress Installation Covering The Entire Channel Perimeter

The thickness of the mattress is determined by three factors: the erodibility of the bank soil, the maximum velocity of the water and the bank slope. The minimum thickness required for various conditions is tabulated in Table 11-7. These values are based on observations of a large number of mattress installations which assume a filling material in the size range of 3 to 6 inches. The mattress thickness should be at least as thick as two overlapping layers of stone. The thickness of mattresses used as bank toe aprons should always exceed 12 inches. The typical range is 12 to 15 inches. The thickness of mattress revetments can vary according to need by utilizing gabions of different depths.

Table 11-7 Criteria For Mattress/Gabion Thickness

Bank Soil Type	Maximum Velocity (ft/s)	Bank Slope (H:V)	Min. Required Mattress Thickness (inches)
Cohesive Soils	10	< 3:1	9
	13-16	< 2:1	12
Silts, fine sands	10	< 2:1	12
Shingle with gravel	16	< 3:1	9
	20	< 2:1	12

For any installation on a slope greater than 2:1 the minimum mattress thickness shall be 18 inches.

11.7 Design Guidelines (continued)

11.7.2.1 Design Guidelines For Mattresses (continued)

Stone Size

The maximum size of stone should not exceed the thickness of individual mattress units. The stone should be well graded within the sizes available, and 95% of the stone, by weight, should be slightly larger than the wire-mesh opening. For commercially available units, the wire-mesh opening sizes are listed in Table 11-6.

Common median stone sizes used in mattress designs range from 3 to 6 inches for mattress less than 12 inches thick. For mattresses of larger thickness, rock having a median size up to 12 inches is used.

Stone Quality

The stone should meet the quality requirements as specified in Section 913 of the ADOT standard specifications.

Basket Fabrication

Commercially fabricated basket units are formed from galvanized steel wire mesh of triple twist hexagonal weave. The netting wire and binding wire is approximately No. 12 gage. The wire for edges and corners is approximately 12 gage. Manufacturer's instructions for field assembly of basket units should be followed.

All wire used in the construction of the mesh rock enclosures including tie wire shall be galvanized in accordance with the ADOT Standard Specifications, Section 913.

Galvanized wire baskets may be safely used in fresh water and in areas where the pH of the liquid in contact with it is not greater than 10. For highly corrosive conditions such as industrial areas, polluted streams and in soils such as muck, peat and cinders, a polyvinyl chloride (PVC) coating must be used over the galvanizing. The PVC coating must have a nominal thickness of 0.02165 inches and shall nowhere be less than 0.015 inches. It shall be capable of resisting deleterious effects of natural weather exposure and immersion in salt water, and shall not show any material difference in its initial characteristics with time.

Edge Treatment

The edges of rock and wire mattress revetment installations (the toe, head and flanks) require special treatment to prevent damage from undermining. Of primary concern is toe treatment. Figure 11-25 illustrates several possible toe configurations. If a toe apron is used, its projection should be 1.5 times the expected maximum depth of scour in the vicinity of the revetment toe. In areas where little toe scour is expected, the apron can be replaced by a single-course gabion toe wall which helps to support the revetment and prevent undermining. In cases where an excessive amount of toe scour is anticipated, both an apron and a toe wall can be used.

11.7 Design Guidelines (continued)

11.7.2.1 Design Guidelines For Mattresses (continued)

To provide extra strength at the revetment flanks, it is recommended that mattress units having additional thickness be used at the upstream and downstream edges of the revetment, Figure 11-28. It is further recommended that a thin layer of topsoil be spread over the flank units to form a soil layer to be seeded when the revetment installation is complete.

Filter Design

Individual mattress units will act as a crude filter as well as a pavement unit when filled with overlapping layers of hand-size stones. However, it is recommended that a layer of permeable membrane cloth (geotextile) woven from synthetic fibers should be placed between the bank and the rock and wire mattress revetment to inhibit washout of fines.

Construction

Construction details for rock and wire mattresses vary with the design and purpose for which the protection is provided. Rock and wire mattress revetments may be fabricated where they are to be placed, or at an off-site location. Fabrication at an off-site location requires that the individual mattress units be transported to the site. In this case extreme care must be taken so that moving and placing the baskets does not damage them by breaking or loosening strands of wire or ties, or by removing any of the galvanizing or PVC coating. Because of the potential for damage to the wire enclosures, off-site fabrication is not recommended.

On-site fabrication of rock and wire mattress revetments is the most common practice. Figure 11-25 illustrates installations on a channel bank. Figure 11-26 illustrates an installation where the entire channel perimeter is lined. Installation of mattress units above the water line is usually accomplished by placing individual units on the prepared bank, lacing them together, filling them with appropriately sized rock and then lacing the tops to the individual units.

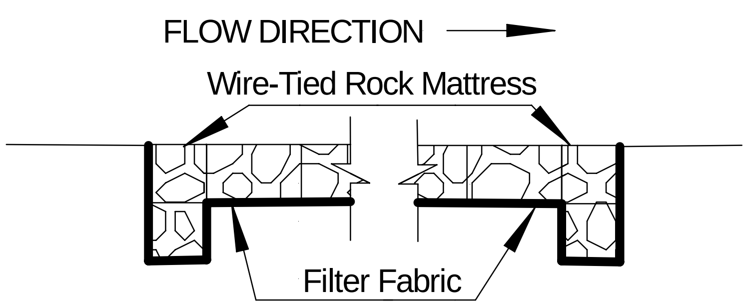


Figure 11-27 Flank Treatment for Wire–Tied Rock and Mattress Designs -Upstream Face and Downstream Face

11.7 Design Guidelines (continued)

11.7.2 Wire-Enclosed Rock (continued)

11.7.2.2 Design Guidelines For Stacked Block Gabions

Components of stacked gabion revetment design include layout of a general scheme or concept, bank and foundation preparation, unit size and configuration, stone size and quality, edge treatment, backfill and filter considerations and basket or rock enclosure fabrication. Design guidelines for stone size and quality, and bank preparation are the same as those discussed for mattress designs, other remaining areas are discussed below.

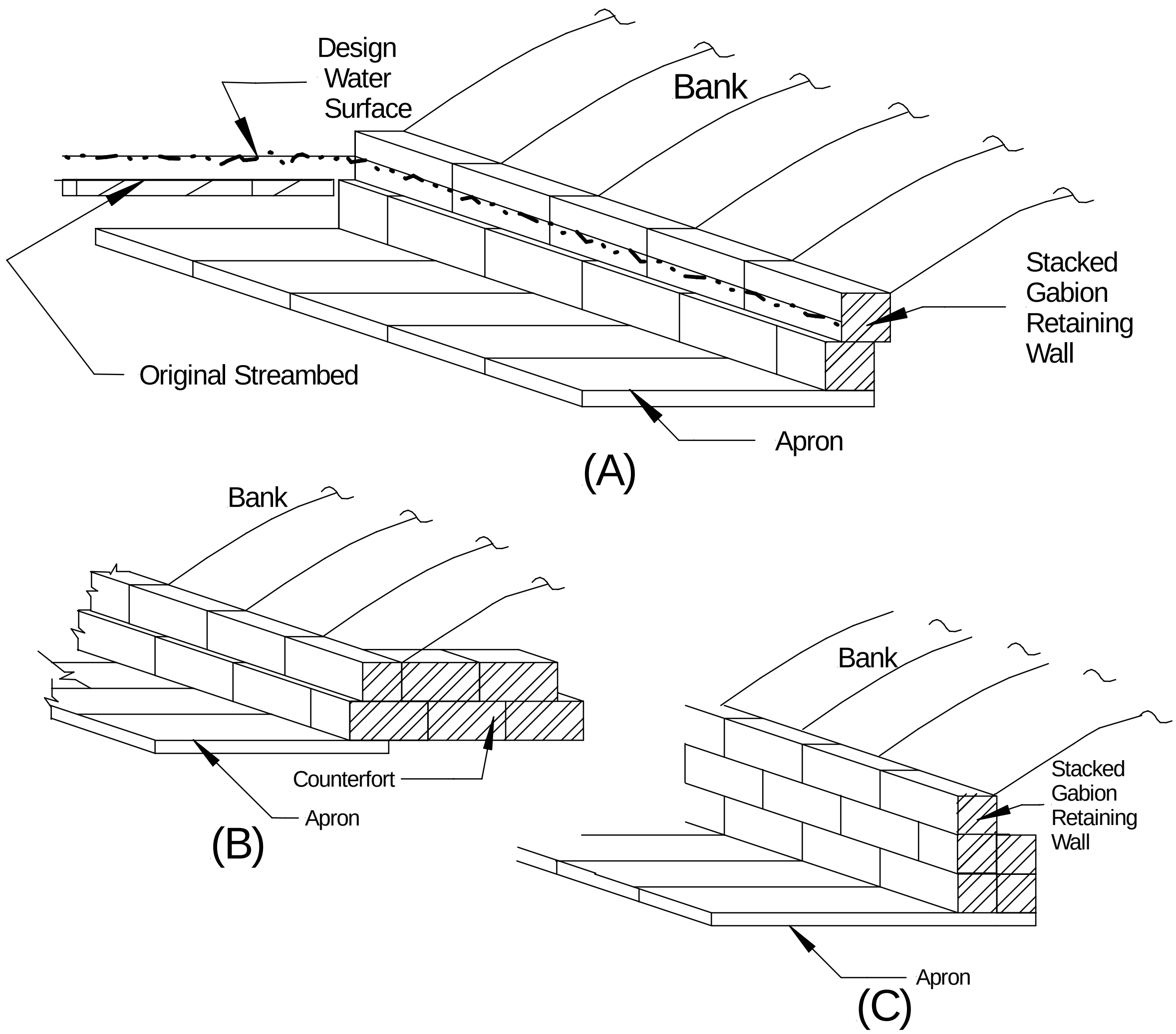


Figure 11-28 Typical Stacked Block Gabion Revetment Details: stepped back low retaining wall with apron.

11.7 Design Guidelines (continued)

11.7.2.2 Design Guidelines For Stacked Block Gabions (continued)

General

Stacked gabion revetments are typically used instead of gabion mattress designs when the slope to be protected is greater than 3H:1V or when the purpose of the revetment is for flow training. They can also be used as retaining structures when space limitations prohibit bank grading to a slope suitable for other revetments.

Stacked gabion revetments must be based on a firm foundation. The foundation or base elevation of the structure should be well below any anticipated scour depth. Additionally, in alluvial streams where channel bed fluctuations are common, an apron should be used as illustrated in Figures 11-25. Aprons are also recommended for situations where the estimated scour depth is uncertain.

Size and Configuration

Common commercial sizes for stacked gabions are listed in Table 11-6. The most common sizes have widths and depths of 3 ft. Sizes less than 1 ft thick are not practical for stacked gabion installations.

Retaining walls can be designed in either a stepped-back configuration as illustrated in Figures 11-28(a) or a batter configuration as illustrated in Figure 11-28(c). Structural details and configurations can vary from site to site.

Gabion walls are gravity structures and their design follows standard engineering practice for retaining structures. Design procedures are available in standard soil mechanics texts as well as in gabion manufacturer's literature.

Edge Treatment

The flanks and toe of stacked block gabion revetments require special attention. The upstream and downstream flanks of these revetments should include counterforts, see Figure 11-28(b). The counterforts should be placed 12 to 18 ft from the upstream and downstream limits of the structure, and should extend a minimum of 12 ft into the bank.

Backfill/Filter Requirements

Standard retaining wall design requires the use of selected backfill behind the retaining structure to provide for drainage of the soil mass behind the wall. The permeable nature of gabion structures permits natural drainage of the supported embankment. However, since material leaching through the gabion wall can become trapped and cause plugging, it is recommended that a geotextile filter be used, Figure 11-25.

11.7 Design Guidelines (continued)

11.7.2.2 Design Guidelines For Stacked Block Gabions (continued)

The toe of the revetment should be protected by placing the base of the gabion wall at a depth below anticipated scour depths. In areas where it is difficult to predict the depth of expected scour, or where channel bed fluctuations are common, it is recommended that a mattress apron be used. The minimum apron length should be equal to 1.5 times the anticipated scour depth below the apron. This length can be increased in proportion to the level of uncertainty in predicting the local toe scour depth.

Basket Fabrication

Commercially fabricated basket units are formed from galvanized steel wire mesh of triple twist hexagonal weave. The netting wire and binding wire specifications are the same as those discussed for mattress units. Specifications for galvanizing and PVC coatings are also the same for block designs as for mattresses. Figure 11-29 illustrates typical details of basket fabrication.

Construction

Construction details for gabion installations typically vary with the design and purpose for which the protection is being provided. Several typical design schematics were presented in Figures 11-28. Design details for a typical stepped-back design and a typical batter design are presented in Figure 11-30.

As with mattress designs, fabrication and filling of individual basket units can be done at the site, or at an off-site location. The most common practice is to fabricate and fill individual gabions at the design site. The following steps outline the typical sequence used for installing a stacked gabion revetment or wall:

Step 1 Prepare the revetment foundation. This includes excavation for the foundation and revetment wall.

Step 2 Place the filter and gabion mattress (for designs which incorporate this component) on the prepared grade, then sequentially stack the gabion baskets to form the revetment system.

Step 3 Each basket is unfolded and assembled by lacing the edges together and the diaphragms to the sides.

Step 4 Fill the gabions to a depth of 1 ft with stone from 6 to 12 inches in diameter. Place one connecting wire in each direction and loop it around two meshes of the gabion wall. Repeat this operation until the gabion is filled.

Step 5 Wire adjoining gabions together by their vertical edges; stack empty gabions on the filled gabions and wire them at front and back.

Step 6 After the gabion is filled, fold the top shut and wire it to the ends, sides and diaphragms.

11.7 Design Guidelines (continued)

11.7.2.2 Design Guidelines For Stacked Block Gabions (continued)

Step 7 Crushed stone and granular backfill should be placed at intervals to help support the wall structure. It is recommended that backfill be placed at three-course intervals.

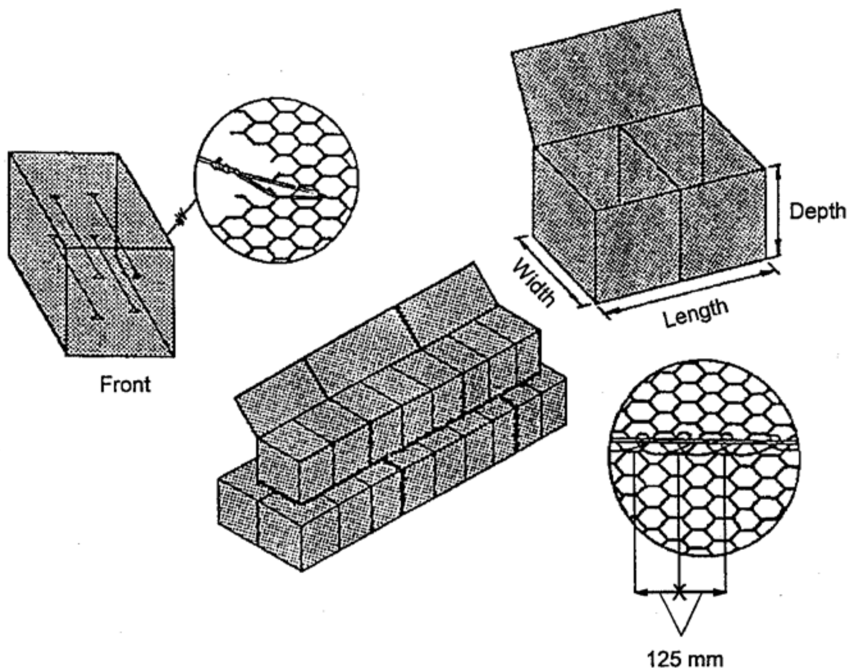
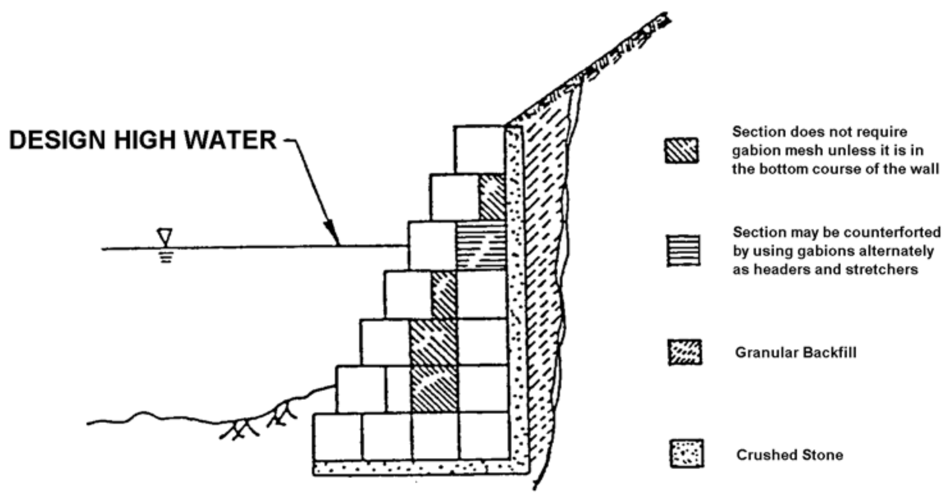


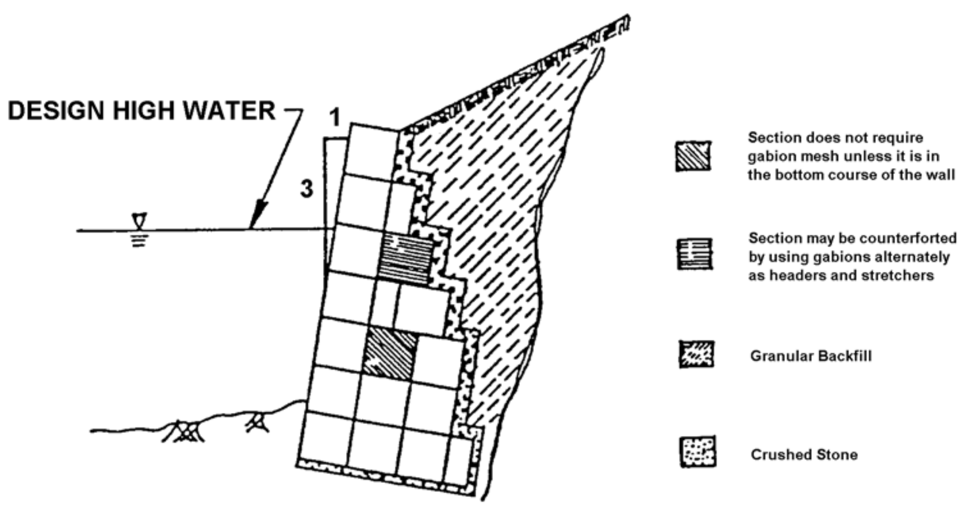
Figure 11-29 Gabion Basket Fabrication

11.7 Design Guidelines (continued)

11.7.2.2 Design Guidelines For Stacked Block Gabions (continued)



(a)



(b)

Figure 11-30 Section Details: (a) stepped back and (b) battered gabion retaining walls

11.7 Design Guidelines (continued)

11.7.3 Soil-Cement

Soil-cement is an acceptable method of slope protection for dikes, levees, channels, and highway embankments. Soil-cement can also be used to construct impervious cores as well as provide a protective facing. On most projects, soil-cement is constructed in stair-step fashion by placing and compacting the soil-cement in horizontal layers stair-stepped up the embankment (Figure 11-31). A compacted layer thickness of 6 inches is most widely used, with the recommended maximum being 9 inches for efficient, uniform compaction. This facilitates placement using common highway construction equipment. Embankment slopes of 1:1 to 2:1 are most common for stair-step construction, slopes steeper than 2:1 may need to be evaluated for stability. The width of a layer may need to be adjusted to provide a **minimum protective thickness of about 3.5 ft**. measured normal to the slope.

A wide variety of soils can be used to make durable soil-cement slope protection. The Portland Cement Association (PCA) has data on soil types, gradations, costs and testing procedures. The PCA also has data placement and compaction methods.

Use of soil-cement does not require any unusual design considerations for the embankment. Proper embankment design procedures should be followed, based on individual project conditions, to prevent subsidence or any other type of embankment distress.

11.7.3.1 Design Guidelines

Top, Toe and End Features

An important consideration in the design of soil-cement facing is to ensure that all extremities of the facing are tied into non-erodible sections. Adequate freeboard and carrying the soil-cement to the paved roadway, plus a lower-section detail as shown in Figure 11-31, will minimize erosion from behind the crest and under the toe of the facing. The ends of the facing should terminate smoothly into the bank and be tied into the bank with counterforts.

Where miscellaneous structures such as culverts extend through the facing, the area immediately adjacent to such structures are constructed by placing and compacting the soil-cement by hand or with small power tools, or by using a lean-mix concrete.

Special Conditions

Slope stability is provided to embankments by the strength and impermeability of the soil-cement facing. Special design considerations usually are not necessary in soil-cement-faced embankments. It is necessary to utilize proper design and analysis procedures to ensure the structural and hydraulic integrity of the embankment. Conditions most commonly requiring special analysis include subsidence of the embankment or rapid drawdown of the reservoir or river.

11.7 Design Guidelines (continued)

11.7.3 Design Guidelines (continued)

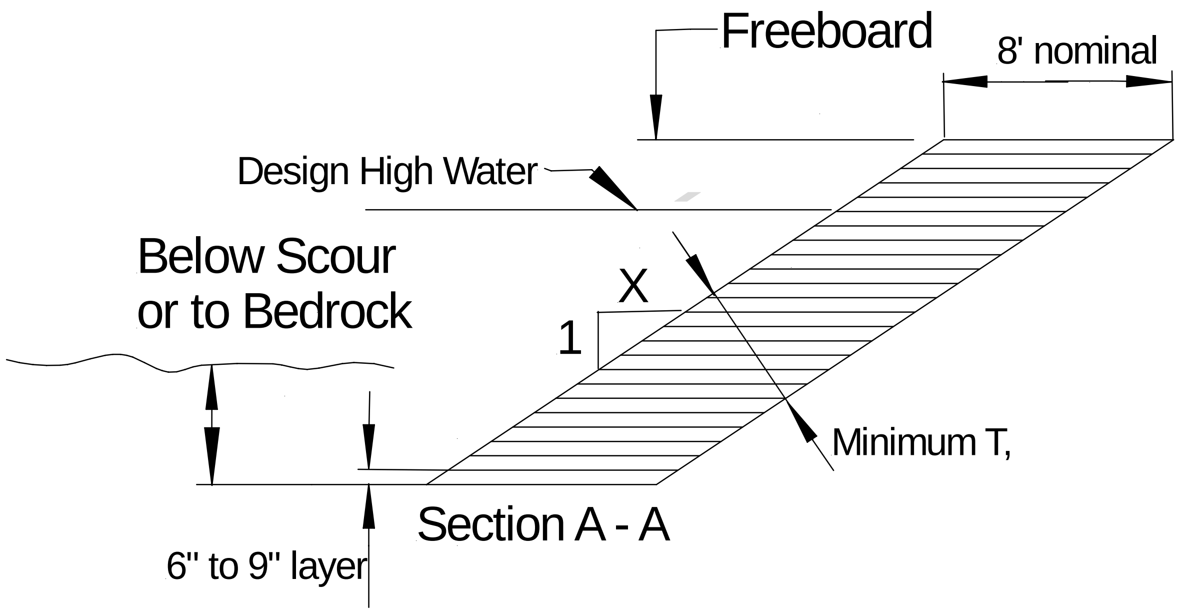


Figure 11-31 Details And Dimensions Of Three Soil-Cement Facings Design Guidelines

Subsidence

Embankment subsidence results from a compressible foundation, settlement within the embankment itself, or both. Analyzing the possible effects of such a condition involves a number of assumptions by the designer concerning the embankment behavior. Combining these assumptions with the characteristics of the facing, a structural analysis of the condition can be made. The layer effect can usually be ignored.

Note: The post construction appearance of a pattern of narrow surface cracks about 10 to 20 ft apart is evidence of normal hardening of the soil-cement. Substantial embankment subsidence conceivably could allow the facing to settle back in large sections coinciding with the normal shrinkage crack pattern. If such settlement of the soil-cement, with separation at the shrinkable cracks, takes place, the slope remains adequately protected unless the settlement is large enough to allow the outer face of a settling section to move past the inner face of an adjoining section.

11.7 Design Guidelines (continued)

11.7.3 Design Guidelines (continued)

Rapid Drawdown

Rapid drawdown exceeding 15 ft or more within a few days theoretically produces hydrostatic pressure from moisture trapped in the embankment against the back of the facing. Three design concepts that may be used to prevent damage due to rapid drawdown-induced pressure are:

1. Designing the embankment so that its least permeable zone is immediately adjacent to the soil-cement facing, which ensures that seepage through cracks in the facing will not build up a pool of water sufficient to produce damaging hydrostatic pressure;

2. evaluating the stability of the soil cement mass using gravity wall approach; and

3. providing free drainage behind, through or under the soil-cement facing to prevent adverse hydrostatic pressure.

11.7.3.2 Construction

The method of construction (central plant or mixed in place) should be considered by the designer in determining the facing cross section. Both methods have been successfully used for soil-cement slope protection. The central plant method allows faster production and provides maximum control of mixing operations. With the mixed-in-place method, mixing should be deep enough so that there will be no unmixed seams between the layers, but excessive striking of the soil-cement below the layer being mixed should be avoided. The PCA has sample specifications regarding these two construction methods.

11.7.4 Grouted Rock

Grouted rock revetment consists of rock slope-protection having voids filled with concrete grout to form a monolithic armor. See section 11.5.4 for additional descriptive information and general performance characteristics for grouted rock.

11.7.4.1 Design Guidelines

Components of grouted rock riprap design include layout of a general scheme or concept, bank preparation, bank slope, rock size and blanket thickness, rock grading, rock quality, grout quality, edge treatment, filter design and pressure relief. Grouted riprap designs are rigid monolithic bank protection schemes. When complete they form a continuous surface. A typical grouted riprap section is shown in Figure 11-32. Grouted riprap should extend from below the anticipated channel bed scour depth to the design high water level, plus additional height for freeboard. During the design phase for a grouted riprap revetment, special attention needs to be paid to edge treatment, foundation design and mechanisms for hydrostatic pressure relief.

11.7 Design Guidelines (continued)

11.7.4.1 Design Guidelines (continued)

Bank And Foundation Preparation

In general, the graded surface should not deviate from the specified slope line by more than 6 inches. However, local depressions larger than this can be accommodated since initial placement of filter material and/or rock for the revetment will fill these depressions. Since grouted riprap is rigid but not extremely strong, support by the embankment must be maintained. The foundation for the grouted riprap revetment should have a bearing capacity sufficient to support either the dry weight of the revetment alone, or the submerged weight of the revetment plus the weight of the water in the wedge above the revetment for design conditions, whichever is greater. To form a firm foundation, it is recommended that the bank surface be tamped or lightly compacted. Care must be taken during bank compaction to maintain soil permeability similar to that of the natural, undisturbed bank material.

Bank Slope

Bank slopes for grouted riprap revetments should not exceed 1.5:1.

Rock Size And Blanket Thickness

Grouted riprap is usually used because larger sizes of dumped riprap are not available. Therefore the largest size of economically available rock should be used. Blanket thickness and rock size requirements for grouted riprap installation are interrelated. The blanket thickness should be a minimum of 150% of the median rock size. The largest rock used in the revetment should not exceed the blanket thickness. Figure 11-37 illustrates a relationship between the design velocity and the minimum required riprap blanket thickness for grouted riprap designs.

Rock Grading

Table 11-9 provides guidelines for rock gradation in grouted riprap installations. Five size classes are listed.

Rock Quality

Rock used in grouted rock slope-protection is usually the same as that used in ordinary rock slope-protection. In addition, the rock used in grouted riprap installations should be free of fines in order that penetration of grout may be achieved.

11.7 Design Guidelines (continued)

11.7.4.1 Design Guidelines (continued)

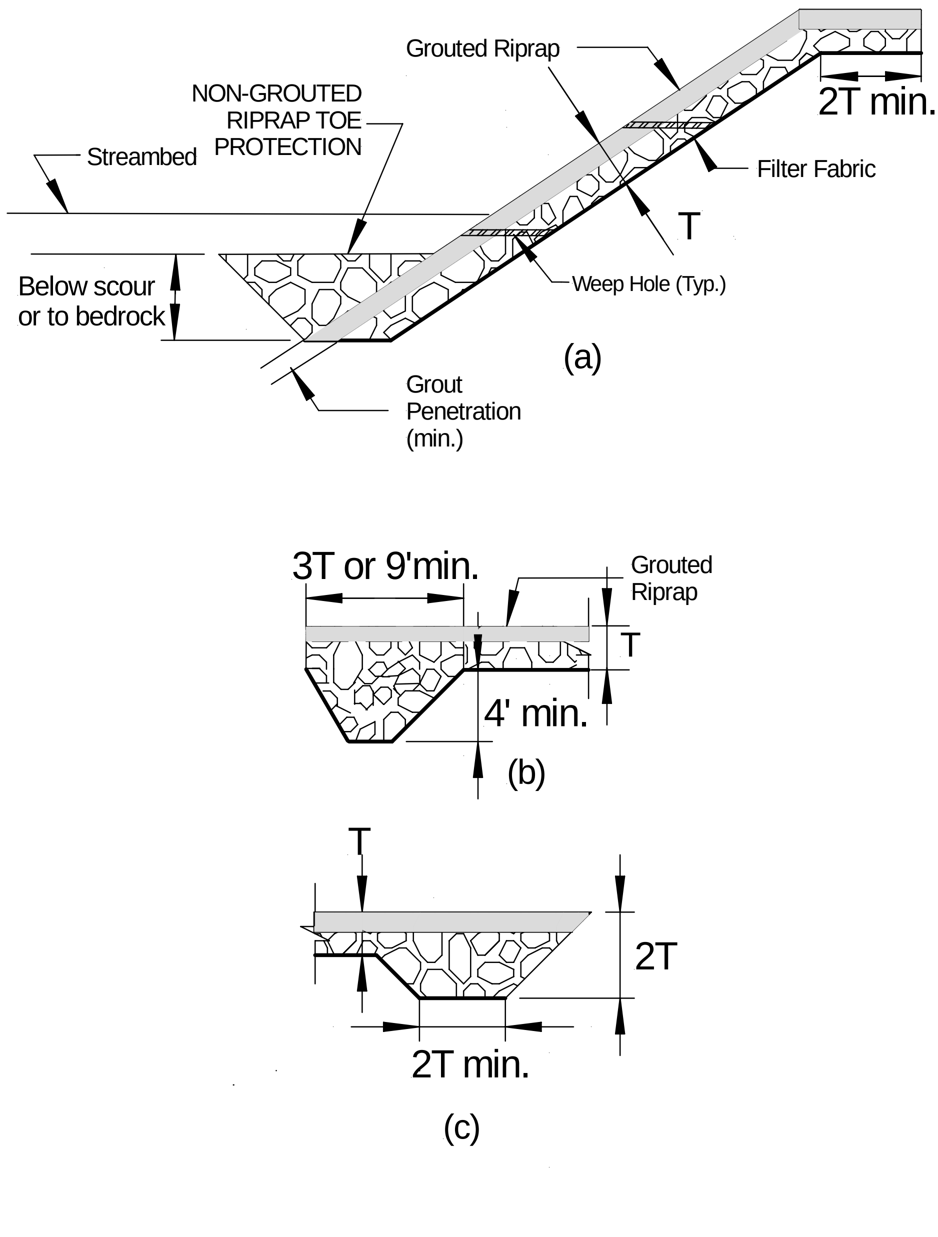


Figure 11-32 Grouted Riprap Sections: (a) section; (b) Upstream Detail; and (c) Downstream Detail

11.7 Design Guidelines (continued)

11.7.4.1 Design Guidelines (continued)

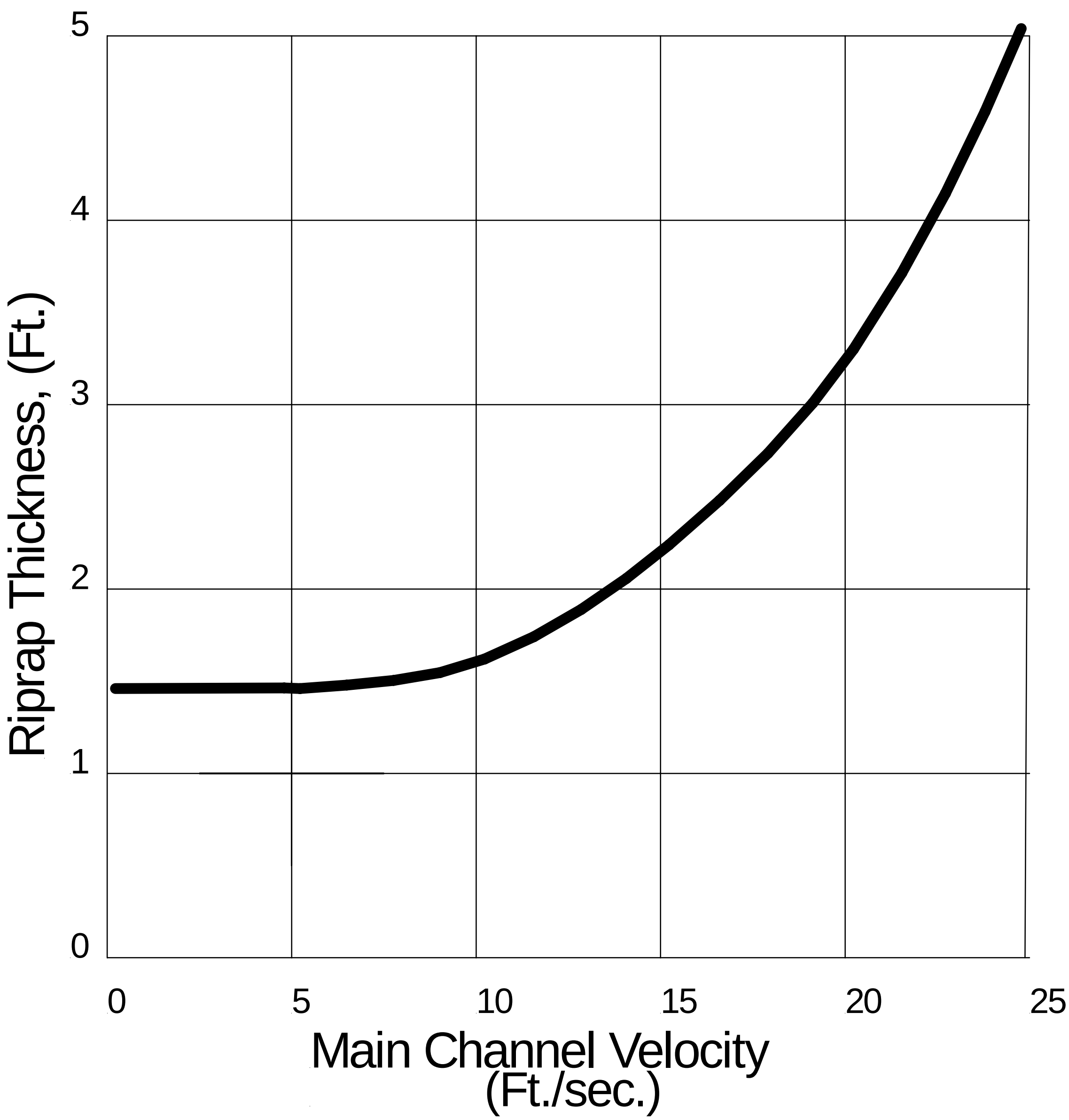


Figure 11-33 Required Blanket Thickness As A Function Of Flow Velocity

11.7 Design Guidelines (continued)

11.7.4.1 Design Guidelines (continued)

Grout Quality And Characteristics

Grout should consist of good strength concrete using a maximum aggregate size of ¾ in and a slump of 3 to 4 inches. Sand mixes may be used where roughness of the grout surface is unnecessary, provided sufficient cement is added to give good strength and workability. The thickness of grout necessary is shown in Table 11-9. The finished grout should leave face stones exposed for one-fourth to one-third their depth and the surface of the grout should expose a matrix of coarse aggregate.

Edge Treatment

The edges of grouted rock revetments (the head, toe and flanks) require special treatment to prevent undermining. The revetment toe should extend to a depth below anticipated scour depths or to bedrock. The toe should be designed as illustrated in Figure 11-32(a). After excavating to the desired depth, the riprap slope protection should be extended to the bottom of the trench and grouted. The remainder of the excavated area in the toe trench should be filled with grout-free riprap. The grout-free riprap provides extra protection against undermining at the bank toe. To prevent outflanking of the revetment, various edge treatments are required. Recommended designs for these edge treatments are illustrated in Figure 11-32, (b) and (c).

Filter Design

Filters are required under all grouted riprap revetments to provide a zone of high permeability to carry off seepage water and prevent damage to the overlying structure from uplift pressures. A 6-inch granular filter is required beneath the pavement to provide an adequate drainage zone. The filter can consist of well-graded granular material or uniformly-graded granular material with an underlying filter fabric. The filter should be designed to provide a high degree of permeability while preventing base material particles from penetrating the filter, thus causing clogging and failure of the protective filter layer.

Pressure Relief

Weep holes should be provided in the revetment to relieve hydrostatic pressure build-up behind the grout surface, Figure 11-32(a). Weeps should extend through the grout surface to the interface with the gravel underdrain layer. Weeps should consist of 3-inch diameter pipes having a maximum horizontal spacing of 6 ft and a maximum vertical spacing of 10 ft. The buried end of the weep should be covered with wire screening or a fabric filter of a gage that will prevent passage of the gravel underlayer.

11.7 Design Guidelines (continued)

11.7.4.2 Construction

The following construction procedures are recommended:

Step 1 Normal construction procedures include (a) bank clearing and grading; (b) development of foundations; (c) placement of the rock slope protection; (d) grouting of the interstices; (e) backfilling toe and flank trenches; and (f) vegetation of disturbed areas.

Step 2 The rock should be wet immediately prior to commencing the grouting operation.

Step 3 The grout may be transported to the place of final deposit by chutes, tubes, buckets, pneumatic equipment, or any other mechanical method which will control segregation and uniformity of the grout.

Step 4 Spading and rodding are necessary where penetration is achieved by gravity flow into the interstices.

Step 5 No loads should be allowed upon the revetment until good strength has been developed

Table 11-9 Recommended Grading Of Grouted Rock Slope Protection

Rock sizes	Classes (Per cent larger than given rock size)					
Equivalent Diameter, Ft.	Weight, Ton	1 Ton	½ Ton	¼ Ton	Light	Facing
3.5	2	0-5				
2.75	2	50-100	0-5			
2.25	0.5	---	50-100	0-5		
1.75	0.25	95-100	---	50-100	0-5	
1.25	0.1	---	95-100	---	50-100	0-5
1	0.0375	---	---	95-100	95-100	50-100
0.5	0.0125	---	---	---	---	95-100
Minimum Penetration of Grout (inches)		24	18	14	10	8

11.7 Design Guidelines (continued)

11.7.5 Concrete Slope Pavement

Concrete slope pavement revetments are cast-in-place, or precast and set in place on a prepared slope to provide a continuous, monolithic armor for bank protection. Cast-in-place designs are the most common of the two design methods. For additional descriptive information and general performance characteristics of concrete pavement see Section 11.5.3.

11.7.5.1 Design Guidelines

Components of concrete pavement revetment design include layout of a general scheme, bank and foundation preparation, bank slope, pavement thickness, pavement reinforcement, edge treatment, stub walls, filter design, pressure relief and concrete quality. Each of these components is addressed below.

Concrete pavement designs are rigid monolithic bank protection schemes. When complete they form a continuous surface. As illustrated in Figure 11-34, typical concrete pavement revetment consists of the bank pavement, a toe section, a head section, cutoff or stub walls, weeps and a filter layer. The various dimensions are labeled in Table 11-10.

As indicated in Figure 11-34, concrete pavements should extend vertically below the anticipated channel bed scour depth, and to a height equal to the design high water level plus additional height for freeboard. The longitudinal extent of protection should be as described in section 11.7.6. One additional consideration in concrete pavement design is the surface texture. Depending on the smoothness required for hydraulics, a float or sand finish may be specified, or if roughness is desired, plans may call for a deformed surface obtained by raking the surface after the initial set.

During the design phase for concrete pavement revetment, special attention needs to be paid to toe and edge treatment, foundation design and mechanisms for hydrostatic pressure relief. Field experience indicates that inadequacies in these areas of design are often responsible for failures of concrete pavement revetments.

Table 11-10 Dimensions For Concrete Slab

Dimension											
A	B	C	D	E	F	G	H	I	J	K	L
6	9	9	1’ 9	2’ 0	9	6	4-5	2’-3’	1’ 6	25’-30’	9

11.7 Design Guidelines (continued)

11.7.5.1 Design Guidelines (continued)

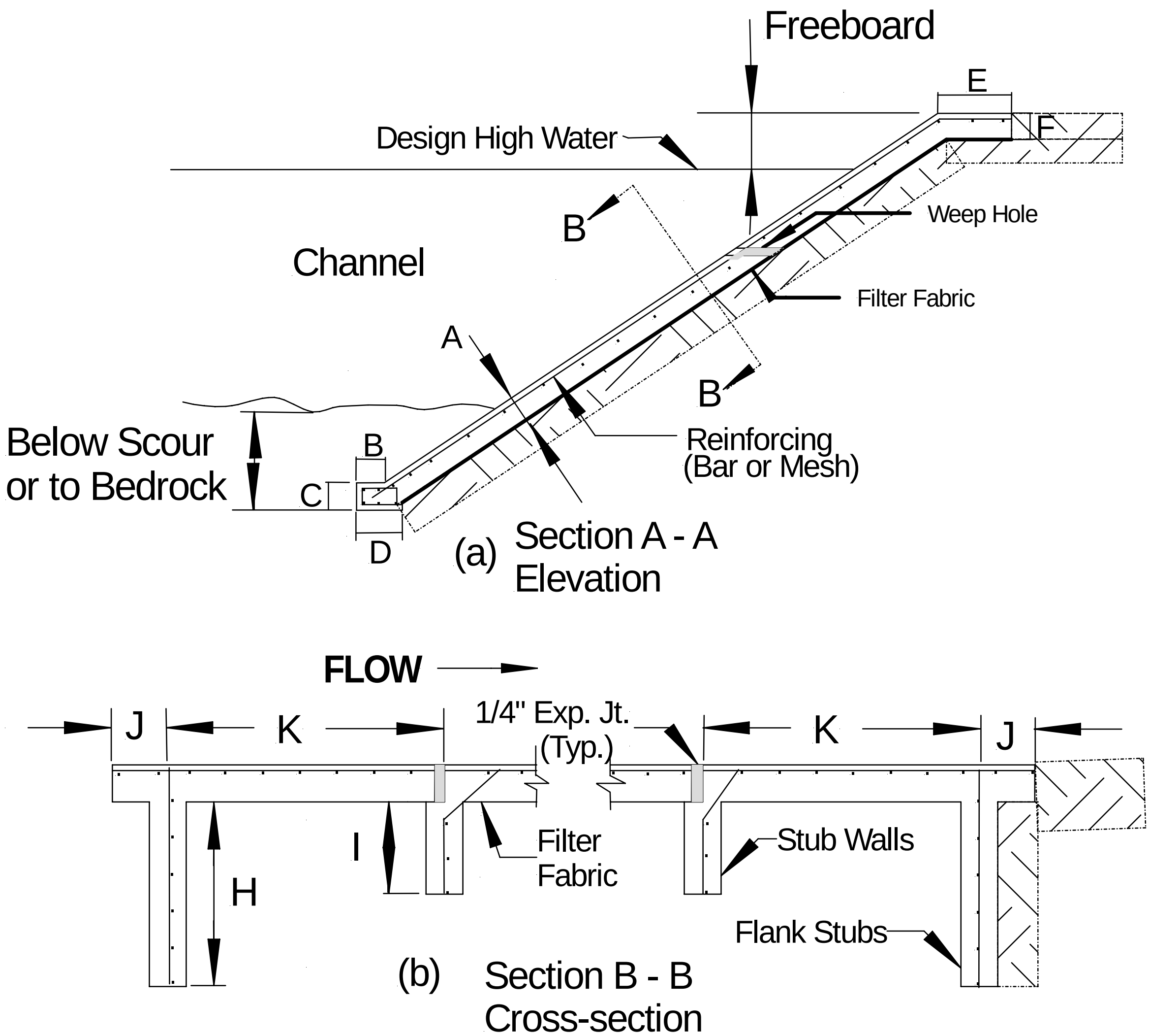


Figure 11-34 Concrete Slope Paving Detail:

(a) typical cross-section, section A-A;

(b) typical longitudinal section, section B-B

11.7 Design Guidelines (continued)

11.7.5.1 Design Guidelines (continued)

Bank And Foundation Preparation

The bank should be prepared by first clearing all trees and debris from the bank, and grading the bank surface to a slope not to exceed 1.5:1. Continuity of the final graded surface is important. After grading, the surface should be true to grade, and stable with respect to slip and settlement. To form a firm foundation, it is recommended that the bank surface be tamped or lightly compacted. Care must be taken during bank compaction to maintain a soil permeability similar to that of the natural, undisturbed bank material. After compaction, the bank surface should not deviate from the specified slope by more than several centimeters at any one point. This is particularly true if pre-cast slabs are to be placed on the bank.

The foundation for the concrete slope pavement revetment should have a bearing capacity sufficient to support either the dry weight of the revetment alone, or the submerged weight of the revetment plus the weight of water in the wedge above the revetment for design conditions, whichever is greater.

Bank Slope

The bank slope for concrete pavements should not exceed 1.5:1.

Pavement Thickness

A minimum pavement thickness of 6 inches is recommended. Dimensions are provided in Table 11-10 for Figure 11-34.

Reinforcement

The purpose of reinforcement is to maintain the continuity of pavement by aggregate interlock even though cracks develop from shrinkage, thermal stresses and flexural stresses. Reinforcement may be either mesh or bar reinforcement. Typically, #5 rebars are used in 6-inch slabs. Both size and spacing in each direction must be specified.

Concrete Quality

Concrete should be of good strength, and the concrete mixture shall be proportioned so as to secure a workable, finishable, durable, watertight and wear resistant concrete of the desired strength.

11.7 Design Guidelines (continued)

11.7.5.1 Design Guidelines (continued)

Edge Treatment

The edges of the concrete pavement (the toe, head and flanks) require special treatment to prevent undermining. Section A-A in Figure 11-34 illustrates standard head and toe designs. The head of the pavement should be tied into the bank and overlapped with soil as illustrated to form a smooth transition from the concrete pavement to the natural bank material. This minimizes scour due to the discontinuity in this area. Also, this design seals off the filter layer from any water that overtops the revetment, thereby reducing the potential for erosion at this interface. Section A-A also illustrates the standard toe design. The revetment toe should extend to a depth below anticipated scour or to bedrock. When this is not feasible without costly underwater construction, an alternative design should be considered. Several alternative designs are illustrated in Figure 11-35, including a riprap filled toe trench, a toe mattress and a sheet-pile toe wall. In all but the latter case, the concrete pavement should extend a minimum of 5 ft below the channel thalweg; the sheet-pile toe wall can be attached to the concrete pavement above, below, or at the channel bed level.

Section B-B of Figure 11-34 illustrates flank treatment. At the upstream and downstream blanks, flank stubs are used to prevent progressive undermining at the flanks.

Stub Walls

As illustrated in Figure 11-34, stub walls should be placed at regular intervals. Stub walls provide support for the revetment at expansion joints; they also guard against progressive failure of the revetment. A maximum spacing of 25 feet is suggested.

Filter Design

Filters are required under all concrete pavement revetments to provide a zone of high permeability to carry off seepage water and prevent damage to the overlying structure from uplift pressures. A 4- to 6-inch granular filter is required beneath the pavement to provide an adequate drainage zone. The filter can consist of well-graded granular material or uniformly graded granular material underlain with an underlying filter fabric. The filter should be designed to provide a high degree of permeability while preventing base material particles from penetrating the filter, thus causing clogging and failure of the protective filter layer.

11.7 Design Guidelines (continued)

11.7.5 Design Guidelines (continued)

Pressure Relief

Weep holes should be provided in the revetment to relieve hydrostatic pressure build-up behind the pavement surface (Figure 11-34). Seeps should extend through the pavement surface and into the granular underdrain or filter layer.

Weeps should consist of 6-inch diameter pipes having a maximum horizontal spacing of 6.0 ft and a maximum vertical spacing of 10 ft. The buried end of the weep should be covered with wire screening or filter fabric of a gage that will prevent passage of the gravel filter layer. Alternatively, a closed end pipe with horizontal slits can be used for the drain; in this case, the slits must be of a size that will not pass the granular filter material.

11.7.5.2 Construction

The following construction procedures and specifications are recommended:

- Normal construction procedures include (a) bank clearing and grading; (b) development of a foundation; (c) trenching and setting forms for stubs; (d) placing the filter layer; (e) forming for and placing the concrete pavement (including any special adaptations necessary for the revetment toe); (f) backfilling toe trenches (if required); and (g) vegetation of disturbed areas.
- The usual specifications for placing and curing structural concrete should apply to concrete slope paving.
- Subgrade should be dampened before placement of the concrete
- Reinforcement must be supported so that it will be maintained in its proper position in the completed paving.
- If the slope is too steep to allow ordinary hand finishing, a 3-inch thickness of mortar may be applied immediately after the concrete has set.
- Slabs should be laid in horizontal courses, with cold joints without filler between courses. These joints should be formed with 4-inch lumber, which should be removed and the joint left open upon completion.
- Vertical expansion joints should run normal to the bank at 15- to 20 ft intervals. These joints should be formed using joint filler.
- Headers or forms for use during screening or rodding operations must be firm enough and so spaced that adjustment will not be necessary during placement operations.

11.7 Design Guidelines (continued)

11.7.5.2 Construction (continued)

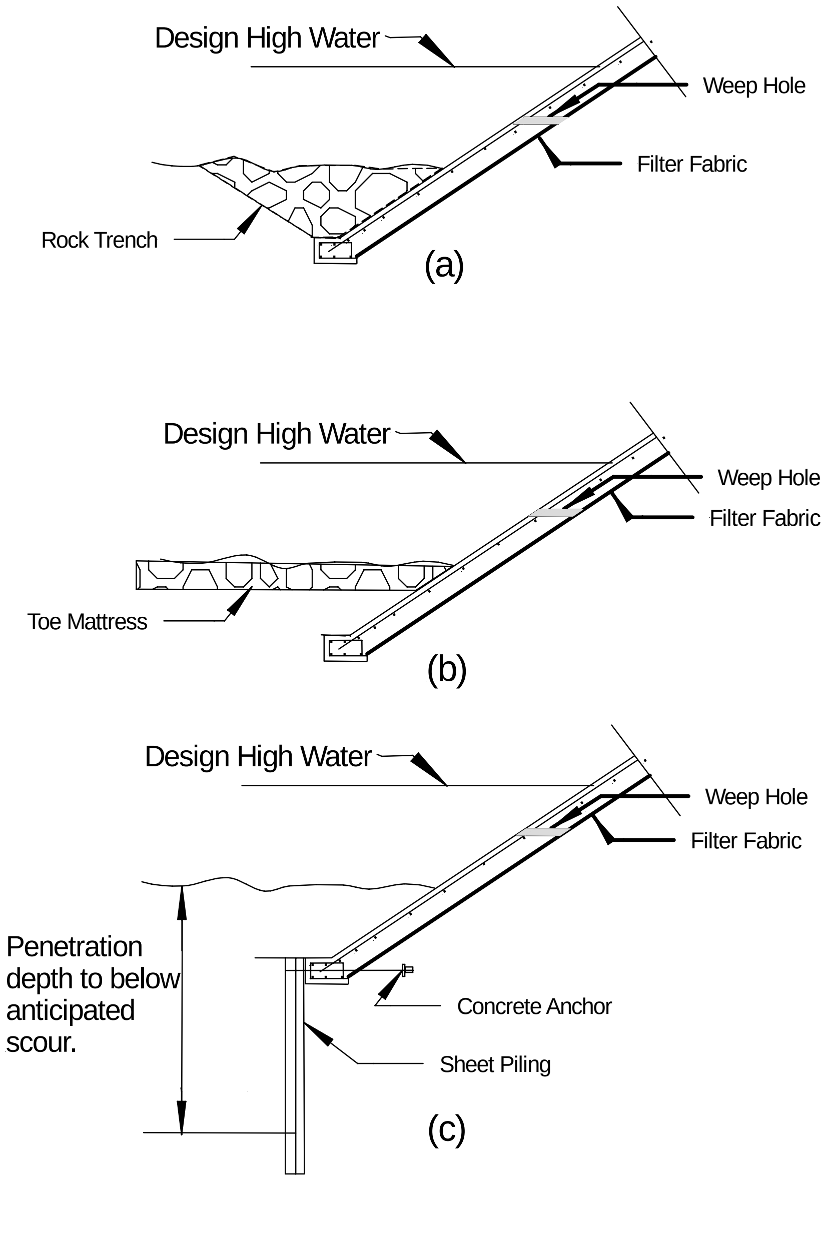


Figure 11-35 Concrete Pavement Toe Details.

11.7 Design Guidelines (continued)

11.7.6 Grouted Fabric Slope Paving

Grouted Fabric-formed revetments are a relatively new development for use on earth surfaces subject to erosion. They have been used as an alternative to traditional revetments such as concrete liners or riprap on reservoirs, canals and dikes.

Grouted fabric-formed revetments are made by pumping a highly fluid structural grout, often referred to as "fine aggregate concrete," into an insitu envelope consisting of a double-layer synthetic fabric. During filling, excess mixing water is squeezed out through the highly permeable fabric substantially reducing the water/cement ratio with consequent improvement in the quality of the hardened concrete. A major advantage of this type revetment is that fabric formed revetments may be as easily assembled underwater as in a dry location.

There are three commonly used types of fabric-formed revetments.

Type 1 Two layers of nylon fabric are woven together at 4-to 6-inch centers as indicated in Figure 11-36. These points of attachment serve as filter points to relieve hydrostatic uplift caused by percolation of ground water through the underlying soil. The finished revetment has a deeply cobbled or quilted appearance. Mat thickness typically average from 2- to 6-inch.

Type 2 Two layers of nylon or polypropylene woven fabric are joined together at spaced centers by means of interwoven tie cords, the length of which controls the thickness of the finished revetment. Plastic tubes may be inserted through the two layers of fabric prior to grout injection to provide weep holes for relief of hydrostatic uplift. The finished revetment is of relatively uniform cross section and has a lightly pebbled appearance. Mat thickness typically averages from 2- to 10 inches.

Type 3 Two layers of nylon fabric are interwoven in a variety of rectangular block patterns, the points of interweaving serving as hinges to permit articulation of the hardened concrete blocks. Revetments are reinforced by steel cables or nylon rope threaded between the two layers of fabric prior to grout injection and remain embedded in the hardened cast-in-place blocks. Block thickness is controlled by spacer cords in the middle of each block.

11.7.6.1 Design Guidelines

The specially woven fabric for grouted fabric formed revetments are manufactured by several different companies. The designer should consult with the manufacturer's literature for designing and selecting the appropriate type of material and thickness for a given hydraulic condition.

11.7 Design Guidelines (continued)

11.7.6.1 Design Guidelines (continued)

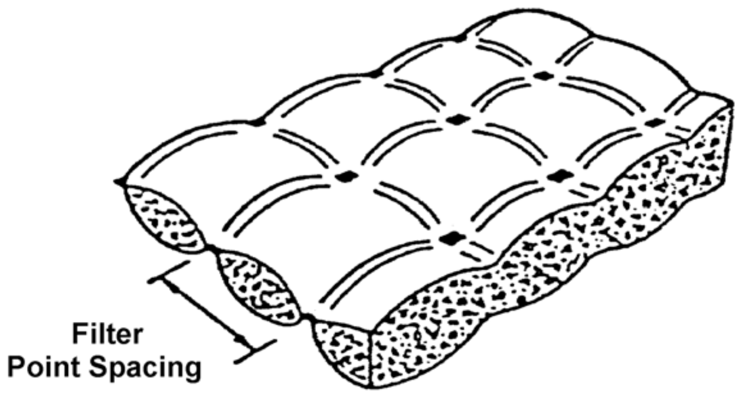


Figure 11-36 Type 1 Grouted Fabric-Formed Revetment

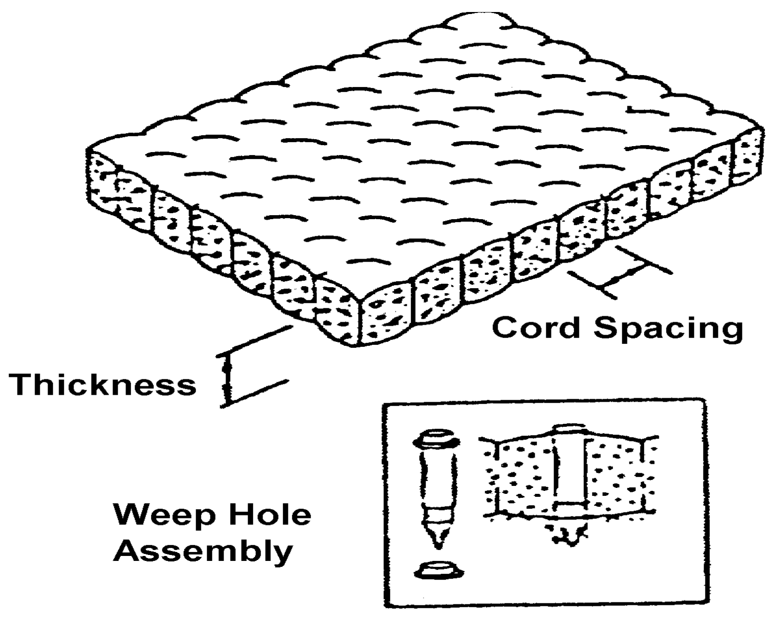


Figure 11-37 Grouted Fabric-Formed Revetment

11.8 References

U.S. Corps of Engineers. Design of Coastal Revetments, Seawalls, and Bulkheads, Engineering Manual EM-1110-2-1614. April 1985.

Federal Highway Administration. Design of Riprap Revetment, Hydraulic Engineering Circular No. 11. 1989.

Federal Highway Administration. Geosynthetic Design and Construction Guidelines, FHWA-HI-95-038. 1995.

Federal Highway Administration. Stream Stability at Highway Structures, Hydraulic Engineering Circular No. 20. February 1991.

Holtz, Robert D., Barry R. Christopher, and Ryan R. Berg. Geosynthetic Design & Construction Guidelines, Participant Notebook. July 29, 1994. Draft. National Highway Institute, U.S. Department of Transportation, Federal Highway Administration.

Appendix A– Geotextile Design and Selection Criteria

Application Evaluation

Step 1. Determine if design requirements are critical and/or severe
A. Critical/Noncritical

1. If the erosion control system fails will there be a risk of loss of life?

2. Does the erosion control system protect a significant structure and will failure lead to significant structural damage.

3. If the geotextile clogs, will failure occur with no warning? Will failure be catastrophic?

4. If erosion control system fails, will the repair costs greatly exceed installation costs?

B. Severe/Nonsevere

1. Are soils to be protected gap graded or pipable soils?

2. Are soils present which consists of primarily silts and uniform sands with 85% passing the #100 sieve?

3. Will the erosion control system be subjected to reversing or cyclic flow conditions such as wave action or tidal variations?

4. Will high gradients exist in the soil to be protected? Will rapid drawdown conditions or seeps or weeps in the soil exist, and whose blockage would produce high hydraulic pressures?

5. Will high velocity conditions exist such as in stream channels?

NOTE: IF the answer is yes to any of the above questions, the design should proceed under the critical/severe Requirements; otherwise use the noncritical/nonsevere design approach.

Step 2. Identify in-place (backfill) Material Properties
A. Grain size analyses.

1. Obtain D_{85} for each soil and select worst soil for retention (i.e., soil with smallest D_{85}).

2. Calculate $C_u = d_{60}/ d_{10}$. Note: When the protected soil contains particles from 1 inch to those passing the #200 sieve, use only the gradation of soil passing the No. 4 sieve in selecting the geotextile (i.e. scalp off the +No. 4 material).

B. Permeability test

1. Select the worse case soil (i.e., soil having the highest coefficient of permeability, k). Permeability of clean sands with 0.1 mm< d_{10} <3mm and C_u <5 can be estimated by Hazen's formula, $k=d_{10}^2$(k in cm/sec; d_{10} in mm). This formula is not to be used for fine-grained soils.

Appendix A– Geotextile Design and Selection Criteria(continued)

Application Evaluation

Step 3 Evaluate Armor Material

A. Size Armor Stone

When minimum size of stone exceeds 4 inches or greater than a 4 inch gap exists between blocks, an intermediate layer 6 inches thick should be used between the armor stone and geotextile. Gravel should be sized such that it will not wash through the armor stone (d_{85} gravel>d_{15} riprap/5).

B. Determine Armor Stone Placement Technique. Design Requirements:

I. Soil Retention

Soils	Steady State Flow	Dynamic Pulsating, and Cyclic Flow
<50% passing US #200 sieve	AOS or $0_{95} <= BD_{85}$	
	$C_u <=2$ or $>= 8$; B=1	
	$2<= C_u<= 4$; $B=0.56C_u$	$0_{95}<=0.5\ D_{85}$
	$4<= C_u <= 8$; $B=8/C_u$	
>50% Passing	Woven: $0_{95}<= D_{85}$	$0_{95}<=0.5\ D_{85}$
	Nonwoven: $0_{95} <=1.8\ D_{85}$	
	0_{95}(fabric) <= 0.3mm (No. 50 sieve)	

NOTE: 1. When the protected soil contains particles for 1-inch size to those passing the U.S. No. 200 sieve, use only the gradation of soil passing the U.S. No. 4 sieve in selecting the fabric.

2. Select fabric on the basis of largest opening value required (smallest AOS).

Appendix A– Geotextile Design and Selection Criteria (continued)

Design Requirements:

II. Permeability

A. Critical/Sever Applications:

k(fabric) >= 10k(soil)

B. Less critical/Less Severe Applications (with clean medium to coarse sands and gravels)

k(fabric) >= k(soil)

NOTE: Permeability should be based on the actual fabric open area available for flow. For example, if 50% of fabric area is to be covered by flat concrete blocks, the effective flow area is reduced by 50%.

III. Clogging Criteria

A. Critical/Severe Applications

Select fabrics meeting I, II, IIIB and perform soil/fabric filtration test before specification, prequalifying the fabric, or selection before bid opening. Alternative: use approved list specification for filtration applications. Suggested performance test method: Gradient Ration <= 3B

B. Less critical/Nonsevere Applications

1. Perform soil/fabric filtration tests.
2. Alterative: $0_{95}>3d_{15}$ for $C_u>3$
3. For $C_u<=3$, fabric with maximum opening size possible (lowest AOS) from retention criteria should be specified.
4. Apparent Open Area Qualifiers[2]

Woven Fabrics: Percent Open Area: >=4%
Nonwoven Fabrics: Porosity[2] >=30%

NOTE: 1. Filtration test are performance test and cannot be performed by the manufacturer as tests depend on specific soil and design conditions. Test to be performed by specifying agency or representative. NOTE: Experience required to obtain reproducible results in gradient ratio test.

2. Porosity requirement based on graded filter porosity.

Appendix A– Geotextile Design and Selection Criteria (continued)

Design Requirements:

IV Survivability Requirements

Physical Requirements For Erosion Control Geotextile
(From AASHTO-AGC-ARTBA Task Force 25)

Property	Class A	Class B	Test Method
Grab Strength, lbs	200	990	ASTM D4632
Elongation, % (min)	15	15	ASTM D4632
Seam Strength, lbs	180	80	ASTM D4632
Puncture Strength, lbs	80	40	ASTM D4833
Burst Strength, psi	320	140	ASTM D3787
Trapezoidal Shear, lbs	50	30	ASTM D3787
Ultraviolet Degradation at 150 hours	70% Strength retained for all Classes		ASTM D4355

NOTE:
1. Acceptance of geotextile material is to be based on TF25 ACCEPTANCE/REJECTION Guidelines (ASTM D4759)

2. ADOT may require a letter from supplier certifying that its geotextile meets specification requirements.

3. Minimum - Use material in weaker principal direction. All numerical values represent minimum average roll value (i.e., test results from any sampled roll in lot shall meet or exceed the minimum values in the table). Stated values for non-critical, non-sever condition. Lot sampled according to ASTM D4354.

4. Class A erosion Control applications are those where fabrics are used under conditions where installations stresses are more severe than Class B. (i.e., stone placement height should be less than 3 feet and stone weights should not exceed 250 pounds.)

5. Class B Erosion Control applications are those where fabric is used in structures or under conditions where the fabric is protected by a sand cushion or by “zero drop height” placement of stone.

6. Values apply to both field and manufactured seams.

CHAPTER 12

PAVEMENT DRAINAGE SYSTEMS

Chapter 12 Pavement Drainage Systems
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Chapter12 Pavement Drainage Systems
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12.1 Overview

12.1.1 Introduction

Storm drainage facilities for pavement drainage consist of curbs, gutters, storm drains, channels and culverts. This chapter discusses the aspects of pavement drainage design such as system planning, pavement drainage, gutter flow calculations, and inlet sizing and location. The pipe sizing and hydraulic grade line calculations for the design of storm drains is covered in Chapter 13. Pump stations are discussed in Chapter 14. Structures that convey flows through the highway are discussed in Chapter 9 Culverts or Chapter 10 Bridges.

12.2 Design Goals

12.2.1 General

A storm drainage system for a street or highway is a collection of structures to collect and convey storm water runoff from land areas to a discharge location in a manner that adequately drains the roadway and minimizes the potential for flooding and erosion to adjacent properties.

The system begins with a concentration system such as gutters and channels, a system of inlets that pass the collected flows into a conveyance system of pipes or channels, that has structures to allow the connection or access to them. The collected flows are eventually conveyed to an outfall. The outfall may discharge to a pump station, storage facilities, or a larger conveyance, such as a storm drain channel. The cost of drainage facilities is neither incidental nor minor on most roads. The quality of the final system usually reflects the attention given to every aspect of the design. The design of a drainage system must address the needs of the traveling public as well as those impacted by the project.

The storm drain system may be categorized as a surface system and a subsurface system. The surface system usually involves gutter flow and inlet interception. This system is used to control the location and amount of water flowing along the gutters or ponding at sags to quantities that will minimize interference with the passage of traffic at the design storm event. This is accomplished by placing inlets at such points and at such intervals to intercept and capture flows as necessary to satisfy the spread and depth criteria for the specified storm frequency.

The subsurface system includes the pipes that convey the flow and the structures that connect the inlets to the pipes. There may be additional structures that allow access to the subsurface system while not being intended for capturing of flows into the subsurface system. The subsurface system allows for the entry of water at each inlet and conveys the collected flows to the discharge location in a manner that contains the flows for the design event. This is accomplished by sizing the pipes and evaluating the energy losses so that the hydraulic grade line is just near the top of the pipes for the design storm event.

Some of the constraints in meeting the hydraulic goals are the available r/w, utilities, budget, alignment, and regulations. The successful design meets the stated hydraulic goals at the lowest total economic costs: construction, maintenance, right-of-way and environmental.

12.3 Symbols and Definitions

12.3.1 Symbols

To provide consistency within this chapter as well as throughout this manual the symbols in Table 12-1 will be used. These symbols were selected because of their wide use in storm drainage publications.

Table 12-1 Symbols And Definitions

Symbol	Definition	Units
A	Area of cross section	ft^2
A	Watershed area	acres
a	Depth of depression	inches
C	Runoff coefficient or coefficient	-
d	Depth of gutter flow at the curb line	ft
D	Diameter of pipe	ft
E_o	Ratio of frontal flow to total gutter flow Q_w/Q	-
h	Height of curb opening inlet	ft
H	Head loss	ft
I	Rainfall intensity	in./hr
K	Coefficient	-
L	Length of curb opening inlet	ft.
L	Pipe length	ft.
L	Pavement width	ft.
L	Length of runoff travel	ft.
n	Roughness coefficient in Manning formula	-
P	Perimeter of grate opening, neglecting bars and side against curb	ft.
P	Tire pressure	psi
Q	Rate of discharge in gutter	ft^3/sec
Q_i	Intercepted flow	ft^3/sec
Q_s	Gutter capacity above the depressed section (See Figure 12-1)	ft^3/sec
Q_T	Total flow	ft^3/sec
Q_W	Gutter capacity in the depressed section (See Figure 12-1)	ft^3/sec
R_h	Hydraulic radius	ft
S or S_x	Pavement cross slope	ft/ft
S	Crown slope of pavement	ft/ft
S or S_L	Longitudinal slope of pavement	ft/ft
S_w	Depressed section slope (See Figure 12-1)	ft/ft
T	Top width of water surface (spread on pavement)	ft
t_c	Time of concentration	min
T_D	Tire tread depth	in.
T_s	Spread above depressed section	ft
TXD	Pavement texture depth	in.
V	Vehicle speed	mph
V	Velocity of flow	ft/sec
W	Width of depression for curb opening inlets	ft
W_d	Rotational velocity on dry surface	rpm
WD	Water depth	in.

12.3 Symbols and Definitions (continued)

Table 12-1 Symbols And Definitions (continued)

Symbol	Definition	Units
W_w	Rotational velocity on flooded surface	rpm
y	Depth of flow in approach gutter	ft
Z	T/d, reciprocal of the cross slope	-

12.3.2 Definitions

Following are discussions of terms that will be used throughout the remainder of this chapter in dealing with different aspects of storm drainage analysis.

Bypass/Flowby (Carry over) -- Occurs at an inlet on grade. It is the flow that is not captured at an inlet on grade, bypasses, and flows to the next inlet downgrade. Inlets on grade are usually designed to allow a certain amount of flowby. Inlets located upstream of an area where pedestrians are expected to use the street may be designed for zero flowby.

Capture Ratio-- The percent of length, area or perimeter expressed as a decimal (0.5, 0.67 or 0.8) of an inlet that is effective in capturing flow after accounting for obstruction by debris. See Chapter 600 of the RDG for appropriate values.

Crown--The crown is the top inside of a pipe, also referred to as the soffit.

Culvert--A culvert is a closed conduit whose purpose is to convey surface water under a roadway, railroad or other impediment. It may have inlets connected to it.

Curb-Opening Inlet--A drainage inlet consisting of a vertical opening in the roadway curb.

Downdrain – an inlet used to convey stormwater from the roadway down the embankment slope consisting of a gap in the roadway curb face that connects to a pipe in the sloped embankment.

Embankment Curb-- The curb along the edge of pavement in fill areas to contain and convey flow along the highway. It inhibits flow from flowing over the side of the embankment.

Energy Grade Line – The locus of points that the describe the total energy of the flowing water, it includes the elevation, pressure and velocity heads. See Hydraulic Grade Line.

Equivalent Cross Slope-- An imaginary straight cross slope having conveyance capacity equal to that of the given compound cross slope.

Flanking Inlets-- Inlets placed upstream and on either side of an inlet at the low point in a sag vertical curve. The purpose of these inlets is to intercept debris as the slope decreases and to act in relief of the inlet at the low point.

12.3 Symbols and Definitions (continued)

12.3.2 Definitions (continued)

Flow-- Flow refers to a quantity of water that is flowing.

Flow-by-- See Bypass

Flume-- An open structure with vertical or nearly vertical sides that carries water.

Frontal Flow-- The portion of the flow that passes over the upstream side of a grate.

Frontal Interception-- Flow that is intercepted by a grate along its upstream side. The interception may be less than the frontal flow

Grate Inlet-- A drainage inlet composed of a grate in the roadway section or at the roadside in a low point, swale or channel.

Grate Perimeter-- The sum of the lengths of all sides of a grate, except that any side adjacent to a curb is not considered a part of the perimeter in weir flow computations.

Gutter-- That portion of the roadway section adjacent to the curb that is utilized to convey stormwater runoff. A composite gutter section consists of the section immediately adjacent to the curb, usually 1.5 or 2.0 ft at a cross-slope of say 0.0588 ft/ft, and the parking lane, shoulder, or pavement at a cross slope of a lesser amount, say 0.02 ft/ft. A uniform gutter section has one constant cross-slope. See Section 12.5.4.3 for additional information.

Hydraulic Grade Line-- The hydraulic grade line is the locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe run (pressure head plus elevation head). See also Energy Grade Line

Inlet – a structure designed to intercept and capture flow. The more common types are grate, curb-opening and slotted drain.

Inlet Efficiency-- The ratio of flow intercepted by an inlet to total flow in the gutter.

Invert-- The invert is the inside bottom of the pipe.

Lateral Line-- A that connects inlets to the main discharge line..

Pressure Head-- Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.

Sag Point-- A low point in a vertical curve. Major Sag Point -- A major sag point refers to a low point that can overflow only if water can pond to a depth of 2.5 feet or more.

12.3 Symbols and Definitions (continued)

12.3.2 Definitions (continued)

Scupper--A hole or slot through the curb, barrier or a bridge deck for the purpose passing drainage. It has no pipe connected to it. A scupper may connect to a lined flume.

Side-Flow-- Flow that is flowing along the side of a grate inlet, as opposed to frontal flow.

Side-flow Interception-- Flow that is intercepted along the side of a grate inlet, the interception may be less than the side-flow.

Slotted Drain Inlet-- A drainage inlet composed of a continuous slot built into the top of a pipe which serves to intercept, collect and transport the flow.

Storm Drain-- A storm drain is defined as that portion of the storm drainage system that receives runoff from inlets and conveys the runoff to some point where it is then discharged into a channel, waterbody, or other piped system. A storm drain may be a closed-conduit, open-conduit, or some combination of the two. Design information is provided in Chapter 13 Storm Drains.

Spillway-- An inlet used to convey stormwater from the roadway down the slope consisting of a opening in the roadway curb face that connects to a flume in the sloped embankment. See scupper and flume.

Splash-Over-- Portion of frontal flow at a grate that skips or splashes over the grate and is not intercepted.

Spread-- The top width of stormwater flow in the gutter measured laterally from the roadway curb. See Section 12.7 for computation methodology. The allowable spread is specified in section 600 of the Roadway Design Guidelines.

Trunk Line-- A trunk line is the main storm drain line. Lateral lines may be connected at inlets, access structures, or with “wyes” or “tees”. A trunk line is sometimes referred to as a "main."

Velocity Head-- Velocity head is a quantity proportional to the kinetic energy of flowing water expressed as a height or head of water, ($V^2/2g$).

12.4 System Planning

12.4.1. Introduction

System planning must occur prior to detailed design of a storm drain system. The master-planning phase should identify the areas to be drained, possible locations of the conveyance system and discharge outfall. The preliminary design is consistent with the broad-system outlines established in the master-planning phase, but provides additional detail on locations of individual drainage structures and features. Activities entail a more detailed hydraulic analysis. It will explicitly account for how the proposed facility fits into the larger-scale plans. It includes topographic mapping on which sub-basins areas and proposed storm drainage facilities have been superimposed as well as locations of other features such as detention basins, pump stations and outfall. The final design will finalize design discharges and determine the dimensions of the inlets and other hydraulic structures. The final design activities and decisions will be documented.

Detention Storage

Reduction of peak flows can be achieved by the storage of runoff in detention basins, storm drainage pipes, swales and channels and other detention storage facilities. Stormwater can then be released to the downstream conveyance facility at a reduced flow rate. The concept should be considered at locations where existing downstream conveyance facilities are inadequate to handle peak flow rates from highway storm drainage facilities. Additional benefits may include the reduction of cost by downstream pipe sizes and the improvement of water quality by removing sediment and/or pollutants. Design consideration and procedures are presented in Chapter 15 Storage Facilities.

12.4.2 General Design Approach

The design of any storm drainage system involves the accumulation of basic data, familiarity with the project site, and a basic understanding of the hydrologic and hydraulic principles and drainage policy associated with that design. The design of a storm drain system is generally a process that evolves as a project develops. The primary ingredients to this process are listed below in a general sequence by which they may be carried out.

1. Data collection (See Chapter 6 — Data Collection and 12.4.3)
2. Coordination with other agencies (12.4.4)
3. Initial System Design (12.4.5)
4. Inlet location and spacing (12.11 & 12.12)
5. Plan layout of storm drain system (Chapter 13.5)
    - locate main outfall
    - determine direction of flow
    - locate existing utilities
    - locate connecting mains
    - locate access holes
6. Size the pipes (Chapter 13.5)
7. Review hydraulic grade line (Chapter 13.6)
8. Prepare the plan
9. Provide documentation (Chapter 4)

12.4 System Planning (continued)

12.4.2.1 Data Collection

Although the primary use of storm drains on the state highway system is to drain the roadway pavement, the outfall of storm drain systems may interact with local drainage systems. If this is so, the designer must collect information regarding the local drainage system. The data needs may include land use patterns, the nature of the physical development of the area(s) to be served by the storm drainage system, the stormwater management plans for the area and the ultimate pattern of drainage (both overland and by storm drains) to the outfall location. Furthermore, there should be an understanding of the nature of the outfall since it usually has a significant influence on the storm drainage system. In environmentally sensitive areas, there may be water quality requirements to consider as well.

Actual surveys of these and other features are the most reliable means of gathering the required data. Existing topographic maps, available from the U. S. Geological Survey, the Soil Conservation Service, many municipalities, some county governments and even private developers are also valuable sources of the kind of data needed for a proper storm drainage design. Where the system outfall interacts with the local drainage system, local governmental agencies should be consulted regarding plans for the area in question. Often, in rapidly growing urban areas, the physical characteristics of an area to be served by a storm drainage system may change drastically in a very short time. In such cases, the designer is to anticipate these changes and consider them in the storm drainage design. Comprehensive Stormwater Management Plans and Floodplain Ordinances should be reviewed when they are available.

12.4.2.2 Cooperative Projects

Cooperative storm drain projects with local government agencies may be beneficial where both a mutual economic benefit and a demonstrated need exist. Early coordination with the local government agency involved is necessary to determine the scope of the project. If cooperative projects are to be undertaken, cost-sharing and design methodology must be documented in agreements.

12.4.2.3 Initial System Design

Preliminary sketches or schematics, featuring the basic components of the intended design, should be developed at the beginning of the design process. Such sketches should indicate watershed areas and land use, existing drainage patterns, plan and profile of the roadway, street and driveway layout with respect to the project roadway, underground utility locations and elevations, locations of proposed retaining walls, bridge abutments and piers, logical inlet and access hole locations, preliminary lateral and trunk line layouts and a clear definition of the outfall location and characteristics. This sketch of intended design elements should be reviewed for determination of areas that are incompatible with the project needs, including construction staging and utility conflicts. With this schematic, the designer is able to proceed with the detailed process of storm drainage design calculations, adjustments and refinements.

12.4 System Planning (continued)

12.4.2.3 Initial System Design (continued)

Layout Considerations

Consideration and planning should be directed toward avoidance of utilities and deep cuts. In some cases, traffic must be maintained or temporary bypasses constructed and temporary drainage provided for during the construction phase. Further consideration should be given to the actual trunk line layout and its constructibility. For example, will the proposed location of the storm drain interfere with in-place utilities or disrupt traffic? Some instances may dictate a trunk line on both sides of the roadway with very few laterals while other instances may call for a single trunk line. Such features are usually a function of economy but may be controlled by other physical features.

Storm drain pipes should not decrease in size in a downstream direction regardless of the available pipe gradient. Except in unusual circumstances, storm drains should discharge to a single outfall. A storm drain that branches, thereby distributing the discharge, should be avoided. Attention shall be given to the storm drain outfalls to insure that the potential for erosion is minimized.

12.5 Design Considerations

12.5.1 Introduction

The desired behavior for the roadway pavement drainage system is usually described in terms of Spread. Spread is the amount the flooded top width of the flow along the gutter. The allowable spread is specified in Chapter 600 of the Roadway Design Guide (RDG) in conjunction with a design flood frequency. The spread at the specified storm frequency is based on allowing operation of the highway without unduly burdensome cost. The major consideration for selection of a design spread is the highway classification as described by the roadway cross-section. The roadway cross-section influences the public expectations for finding water on the pavement surface. Other considerations include inconvenience, hazards and nuisances to pedestrian traffic and buildings adjacent to roadways that are located within the splash zone.

12.5.2 Hydrology

The Rational Method is the usual method to determine the peak flow rate for the design of storm drain systems. The rational method is described in the ADOT Hydrology Manual. Its use should be limited to systems with drainage areas of 160 acres or less. A minimum time of concentration of 10 minutes is generally acceptable. Drainage systems involving detention storage, pumping stations and large or complex storm systems will require the development of a runoff hydrograph. This may require the use of the HEC-1 procedures.

12.5 Design Considerations (continued)

12.5.3 Roadway Design Elements for Pavement Drainage

Roadway features that impact the handling of stormwater include:

- pavement width
- longitudinal and cross slope
- curb and gutter sections.

The pavement width, cross section shape and slope determine the time it takes for storm water to drain to the gutter section. The gutter cross-section and longitudinal slope control the quantity of flow that can be carried in the gutter section.

12.5.3.1 Longitudinal Slope

A minimum longitudinal gradient is more important for a curbed pavement than for an uncurbed pavement since it is susceptible to the spread of stormwater against the curb. Desirable gutter grades should not be less than 0.3% for curbed pavements with a minimum of 0.2%. Minimum grades can be achieved in very flat terrain by use of a rolling profile.

To provide adequate drainage in sag vertical curves, a minimum slope of 0.3% should be maintained within 50 feet of the level point in the curve. This is accomplished where the length of the curve divided by the algebraic difference in grades is equal to or less than 150; $K = L/A$, $(L_C/(g_1-g_2))$. Although ponding is not usually a problem at crest vertical curves, on extremely flat curves a similar minimum gradient should be provided to facilitate drainage.

12.5.3.2 Cross Slope

The design of pavement cross slope is often a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort. The USDOT, FHWA (FHWA-RD-79-30, 31, 1979) reports that cross slopes of 2% have little effect on driver effort in steering, especially with power steering, or on friction demand for vehicle stability.

12.5.3.3 Shoulder Gutter And/Or Curbs

Curbs are used where there is a need to control runoff from the pavement. This may be due to locations where concentration of runoff would erode fill slopes or would flow off the right-of-way at unwanted locations. Curbing may also serve other purposes that include traffic control, providing pavement delineation, containing the surface runoff within the roadway and away from adjacent properties, preventing erosion and enabling the orderly development of property adjacent to the roadway.

Curbs may be either mountable or barrier type. Mountable curbs are less than 6 inches in height and have rounded or plane sloping faces. If mountable curb is used, the gutter and shoulder grades should be the same to maximize the amount of flow than can be carried along the curb. Where barrier curbs are used, a steeper gutter cross slope can be effective at increasing gutter capacity and reducing spread on the pavement.

12.5 Design Considerations (continued)

12.5.3.3 Shoulder Gutter And/Or Curbs (continued)

A curb and gutter forms a triangular channel that can be an efficient hydraulic conveyance facility that can convey runoff of a lesser magnitude than the design flow without interruption of the traffic. When a design storm flow occurs, there is a spread or widening of the conveyed water surface and the water spreads to include not only the gutter width, but also parking lanes or shoulders, and portions of the traveled surface. This is the width the hydraulic engineer is most concerned about in curb and gutter flow, and limiting this width becomes a very important design criterion.

Shoulders should generally be sloped to drain away from the pavement, except with raised, narrow medians. Shoulder gutter and/or curbs may be appropriate to protect fill slopes from erosion caused by water from the roadway pavement. Shoulder gutter and/or curbs may be appropriate at bridge ends where concentrated flow from the bridge deck would otherwise run down the fill slope. This section of gutter should be long enough to include the transitions; it is usually located behind the guardrail at the end of the bridge. Shoulder gutters are not required on the high side of super-elevated sections or adjacent to barrier walls on high fills.

12.5.4 Roadside And Median Ditches

Roadside channels are commonly used with uncurbed roadway sections to convey runoff from the highway pavement and from areas that drain toward the highway. Due to right-of-way limitations, roadside channels cannot be used on most urban arterials. They can be used in cut sections, depressed sections and other locations where sufficient right-of-way is available and driveways or intersections are infrequent. Where practicable, the flow from major areas draining toward curbed highway pavements should be intercepted in the ditch as appropriate.

It is preferable to slope median areas and inside shoulders to a center swale, to prevent drainage from the median area from running across the pavement. This is particularly important for high-speed facilities and for facilities with more than two lanes of traffic in each direction.

12.5.5 Median/Median Barriers

Medians are commonly used to separate opposing lanes of traffic on divided highways. It is preferable to slope median areas and inside shoulders to a center depression to prevent drainage from the median area from running across the traveled pavement. Where median barriers are used and, particularly on horizontal curves with associated super-elevations, it is necessary to provide inlets and connecting storm drains to collect the water that accumulates against the barrier. Slotted drains adjacent to the median barrier and in some cases weep holes in the barrier can also be used for this purpose.

12.5 Design Considerations (continued)

12.5.6 Impact Attenuators

The location of impact attenuator systems should be reviewed to determine the need for drainage structures in these areas. Impact attenuators t usually require a clear or unobstructed opening as traffic approaches the point of impact to allow a vehicle to impact the system head on. If the impact attenuator is placed in an area where superelevation or other grade separation occurs, grate inlets and/or slotted drains may need to be placed to prevent water from running through the clear opening and crossing the highway lanes or ramp lanes. Curb, curb-type structures or swales cannot be used to direct water across this clear opening as vehicle vaulting could occur when the impact attenuator system is utilized.

12.5.7 Bridge Decks

Drainage of bridge decks is similar to other curbed roadway sections. It is often less efficient, because cross slopes are flatter, parapets collect large amounts of debris and small drainage inlets or scuppers have a higher potential for clogging by debris. Bridge deck construction usually requires a constant cross slope. The gutter spread should be checked to insure compliance with the design criteria in Section 12.9. Zero gradients and sag vertical curves and superelevation transitions with flat pavement sections should be avoided on bridges. The minimum desirable longitudinal slope for bridge deck drainage should be 0.5%.

Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. In many cases, deck drainage must be carried several spans to the bridge end for disposal.

Many bridges will not require any drainage structures at all. The Rational equation and the spread equation can be combined to determine the length of deck possible without drainage structures and without exceeding the allowable spread. In many situations, scuppers are the recommended method of deck drainage because they can reduce the problems of transporting a relatively large concentration of runoff in an area of generally limited right-of-way. They also have a low initial cost and are relatively easy to maintain. However, the use of scuppers should be evaluated for site-specific concerns. Scuppers should not be located over embankments, slope pavement, slope protection, driving lanes, or railroad tracks. Runoff collected and transported to the end of the bridge should generally be collected by inlets and down drains, although flumes may be used for extremely minor flows in some areas.

Bridge Deck Drainage

For decks with a uniform cross slope the following equation can be utilized (FHWA Report No. RD-79-31, 1979):

$$L = \frac{0.56\,(S_x^{1.67})(S^{0.5})(T^{2.67})}{nCiW} \qquad \textbf{(12.1)}$$

Where: L = length of deck, ft

S_x = cross slope, ft/ft

S = longitudinal slope, ft/ft

T = allowable spread, ft

n = Manning's n, usually 0.016

C = runoff coefficient, usually 0.95

i = rainfall intensity, in./hr

W = width of drained deck, ft

12.5 Design Considerations (continued)

12.5.8 Gutter Flow

Once concentrated by curbs, the spread of the flow along the gutter must be evaluated for compliance with the design criteria. When the spread exceeds the allowable width, the flow is discharged by use of inlets or flumes. Gutter flow calculations are necessary to relate the quantity of flow to the spread of water on the pavement section. Composite gutter sections have a greater hydraulic capacity for normal cross slopes than uniform gutter sections and are therefore preferred. Refer to Section 12.7 for additional information and procedures.

12.5.9 Inlets

The term "inlets" refers to all types of inlets such as grate inlets, curb inlets and slotted inlets. Drainage inlets are sized and located to limit the spread of water on traffic lanes to tolerable widths for the design storm in accordance with the design criteria specified in Chapter 600 of the Roadway Design Guidelines, RDG. The width of water spread on the pavement at sags should not be substantially greater than the width of spread encountered on continuous grades. Grate inlets and depression of curb opening inlets should be located outside the through traffic lanes to minimize the shifting of vehicles attempting to avoid them. All grate inlets shall be bicycle safe when used on roadways that allow bicycle travel. Curb inlets are preferred to grate inlets at major sag locations because of their debris handling capabilities.

In sag vertical curves locations where significant ponding may occur such as at underpasses or in depressed sections, it is recommended practice to place flanking inlets on each side of the inlet at the low point in the sag. Review Section 12.10.3 for a discussion on the location of inlets.

12.6 Hydrology

12.6.1 Introduction

The rational method is the most common method in use for the design of storm drains when the momentary peak flow rate is desired. Its use should be limited to systems with drainage areas of 160 acres or less. Drainage systems involving detention storage and pumping stations require the development of a runoff hydrograph. Hydrology methods are described in the ADOT Hydrology Manual.

12.6.2 Rational Method

The Rational Equation is written as follows:

(12.2)

Where: Q = discharge, ft^3/sec
C = runoff coefficient
i = rainfall intensity, in/hr
A = drainage area, acres

12.6.2.1 Runoff Coefficient, C

The runoff coefficients for various types of surfaces are discussed in the ADOT Hydrology manual. The weighted C value is to be based on a ratio of the drainage areas associated with each C value as follows:

$$\text{weighted } C = [A_1C_1 + A_2C_2 + A_3C_3] / [A_1 + A_2 + A_3] \qquad \textbf{(12.3)}$$

12.6.2.2 Rainfall Intensity, i

Rainfall intensity (i): Rainfall intensity is the intensity of rainfall in inches per hour for a duration equal to the time of concentration. Intensity is the rate of rainfall per hour for the total amount of rainfall over an interval of time such that intensity multiplied by duration equals amount of rain, i.e., an intensity of 6 inches/hr for a duration of 10 min indicates a total rainfall amount of $6 \times 10/60 = 1.0$ in. The user will need Intensity-Duration information for computation of discharge for areas that have a time of concentration greater than the minimum. See ADOT Hydrology manual for data to be used for determining the intensity of rainfall.

12.6.2.3 Time Of Concentration, t_c

The time of concentration is defined as the period required for water to travel from the most hydraulically distant point of the watershed to the point of the storm drain system under consideration. The designer is usually concerned about two different times of concentration: one for inlet spacing and the other for pipe sizing. As one progresses downstream in a storm drain system there will be a major difference between the two times. The inlet time will be a function of the overland flow path to the inlet and not be altered by the subsurface flow paths. The pipe sizing time of concentration is the sum of time increments upstream of that pipe.

12.6 Hydrology (continued)

12.6.2 Rational Method (continued)

Inlet Spacing

The time of concentration (t_c) for inlet spacing is the time for water to flow from the hydraulically most distant point of the drainage area to the inlet, which is known as the inlet time. Usually this is the sum of the time required for water to move across the pavement or overland to the gutter, plus the time required for flow to move through the length of gutter to the inlet. For pavement drainage, when the total time of concentration is less than 10 minutes, a minimum t_c of 10 minutes should be used to estimate the intensity of rainfall. The time of concentration for the second downstream inlet and each succeeding inlet should be determined independently, the same as the first inlet. In the case of a constant roadway grade and relatively uniform contributing drainage area, the resultant time of concentration for each succeeding inlet could also be constant.

Overland time of concentration is developed by one of two methods: the SCS curve number or the kinematic wave approach. Channel time of concentration can be developed using one of three methods: SCS grassy waterway channel, Manning's equation, or the HEC-22 triangular gutter approach.

12.6.3 Detention

Estimation of the effects of detention requires a reservoir routing procedure such as that presented in Chapter 15 Storage Facilities. The use of reservoir routing procedures for peak flow attenuation is valid and particularly useful in a pump station based storm drainage system in which there are substantial lengths of large diameter pipes. In such systems, the storage capacity of the pipes can have a substantial effect on the final shape of the runoff hydrograph.

12.7 Gutter Flow Calculations

12.7.1 Introduction

Gutter flow calculations are necessary in order to relate the quantity of flow (Q) in the curbed channel to the spread of water on the pavement section. The nomographs in Figure 12-1 and Figure 12-2 can be utilized to solve for flow in uniform cross slope gutters, composite gutter sections and V-shape gutter sections. Figure 12-3 is also very useful in solving composite gutter section problems. Computer programs exist for this computation as well as inlet capacity. Where vertical curb is used, composite gutter sections have a greater hydraulic capacity for normal cross slopes than uniform gutter sections and are therefore preferred. Example problems for each gutter section are shown in the following sections.

The following form of Manning's equation is used for flow in a triangular section

$$\mathbf{Q = (0.56/n)\,(S_x^{1.67})(S^{0.5})(T^{2.67})} \qquad \textbf{(12.4)}$$

Where: S_x = cross slope, ft/ft
S = longitudinal slope, ft/ft
T = spread, ft
n = Manning's n

12.7.2 Manning's n For Pavements

Type of Gutter or Pavement	Manning's n
Concrete gutter, troweled finish	0.012
Asphalt Pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter-asphalt pavement	
Smooth	0.013
Rough	0.015
Concrete pavement	
Float finish	0.014
Broom finish	0.016
For gutters with small slope, where sediment may accumulate, increase above n values by:	0.002
Reference: USDOT, FHWA, HDS-3 (1961)	

Normally use 0.016 for ADOT pavements.

12.7 Gutter Flow Calculations (continued)

12.7.3 Uniform Cross Slope Procedure

The gutter capacity of uniform cross slope section can be solved using the following procedure with the nomograph in Figure 12-1 or by equation 12.4:

CONDITION 1: **Given gutter flow (Q), find spread (T).**
Step 1: Determine input parameters, including longitudinal slope (S), cross slope (S_x), gutter flow (Q) and Manning's n.

Step 2: Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 3: Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not, use the product of Q and n.

Step 4: Read the value of the spread (T) at the intersection of the line from Step 3 and the spread scale.

CONDITION 2: **Given spread (T), find gutter flow (Q).**

Step 1: Determine input parameters, including longitudinal slope (S), cross slope (S_x), spread (T) and Manning's n.

Step 2: Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 3: Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or Qn from the intersection of that line on the capacity scale.

Step 4: For Manning's n values of 0.016, the gutter capacity (Q) from Step 3 is selected. For other Manning's n values, the gutter capacity times n (Qn) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

12.7.4 Composite Cross Slope Procedure

The flow in a composite gutter section is determined by using Figure 12-2 and Figure 12-1. Figure 12-2 is used to find the flow in a gutter section of width (W) less than the total spread (T). Such calculations are generally used for evaluating frontal flow for grate inlets.

CONDITION 1: **Given flow (Q), find spread (T).**

Example: $S = 0.01$; $S_x = 0.02$; $S_w = 0.06$; $W = 2.0$ ft; $n = 0.016$; $Q = 2.0$ ft^3/sec; try $Q_s = 0.7$ ft^3/sec

Step 1: Determine input parameters, including longitudinal slope (S), cross slope (S_x), depressed section slope (S_w), depressed section width (W), Manning's n, gutter flow (Q) and a trial value of the gutter capacity above the depressed section (Q_s).

12.7 Gutter Flow Calculations (continued)

12.7.4 Composite Cross Slope Procedure (continued)

Step 2: Calculate the gutter flow in width, W (Q_w), using the equation:

$$\mathbf{Q_w = Q - Q_s} \tag{12.5}$$

$Q_w = 2.0 – 0.7 = 1.3\ ft^3/s$

Step 3: Calculate the ratios Q_w/Q and S_w/S_x and use Figure 12-2 to find an appropriate value of W/T. (Q_w/Q = 1.3/2.0 = 0.65 S_w/S_x = 0.06/0.02 = 3 From Figure 12-2, W/T = 0.27)

Step 4: Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3. T = 2.0/0.27 = 7.4 ft.

Step 5: Find the spread above the depressed section (T_s) by subtracting W from the value of T obtained in step 3. T_s = 7.4 – 2.0 = 5.4 ft.

Step 6: Use the value of T_s from Step 4 along with Manning's n, S and S_x to find the actual value of Q_s from Figure 12-1. From Figure 12-1 $Q_s = 0.49\ ft^3/sec$.

Step 7: Compare the value of Q_s from Step 5 to the trial value from Step 1. If values are not comparable, select a new value of Q_s and return to Step 1.

Compare 0.49 to 0.70 "no good," Try Q_s = 0.80; then Q_w =2.0-0.8 = 1.2; and Q_w/Q=1.2/2.0 = 0.6; From Figure 12-2, W/T = 0.23, then T = 2.0/0.23 = 8.7 ft. and T_s = 8.7 – 2.0 = 6.7 ft. From Fig 12-1, $Q_s = 0.81\ ft^3/s$ OK

ANSWER: Spread T = 8.6 ft

CONDITION 2: **Given spread (T), find gutter flow (Q).**

EXAMPLE: Spread T = 10.0 ft; W = 2.0 ft; T_s = 10.0-2.0 = 8.0 ft; S_x= 0.04;
S = 0.005 ft/ft; S_w = 0.06; n = 0.016; d = 5 inches

Step 1: Determine input parameters, including spread (T), spread above the depressed section (T_s), cross slope (S_x), longitudinal slope (S), depressed section slope (S_w), depressed section width (W), Manning's n and depth of gutter flow (d).

Step 2: Use Figure 12-1 to determine the capacity of the gutter section above the depressed section (Q_s). Use the procedure for uniform cross slopes- Condition 2, substituting T_s for T. From Figure 12-1, $Q_s=3.0\ ft^3/s$

12.7 Gutter Flow Calculations (continued)

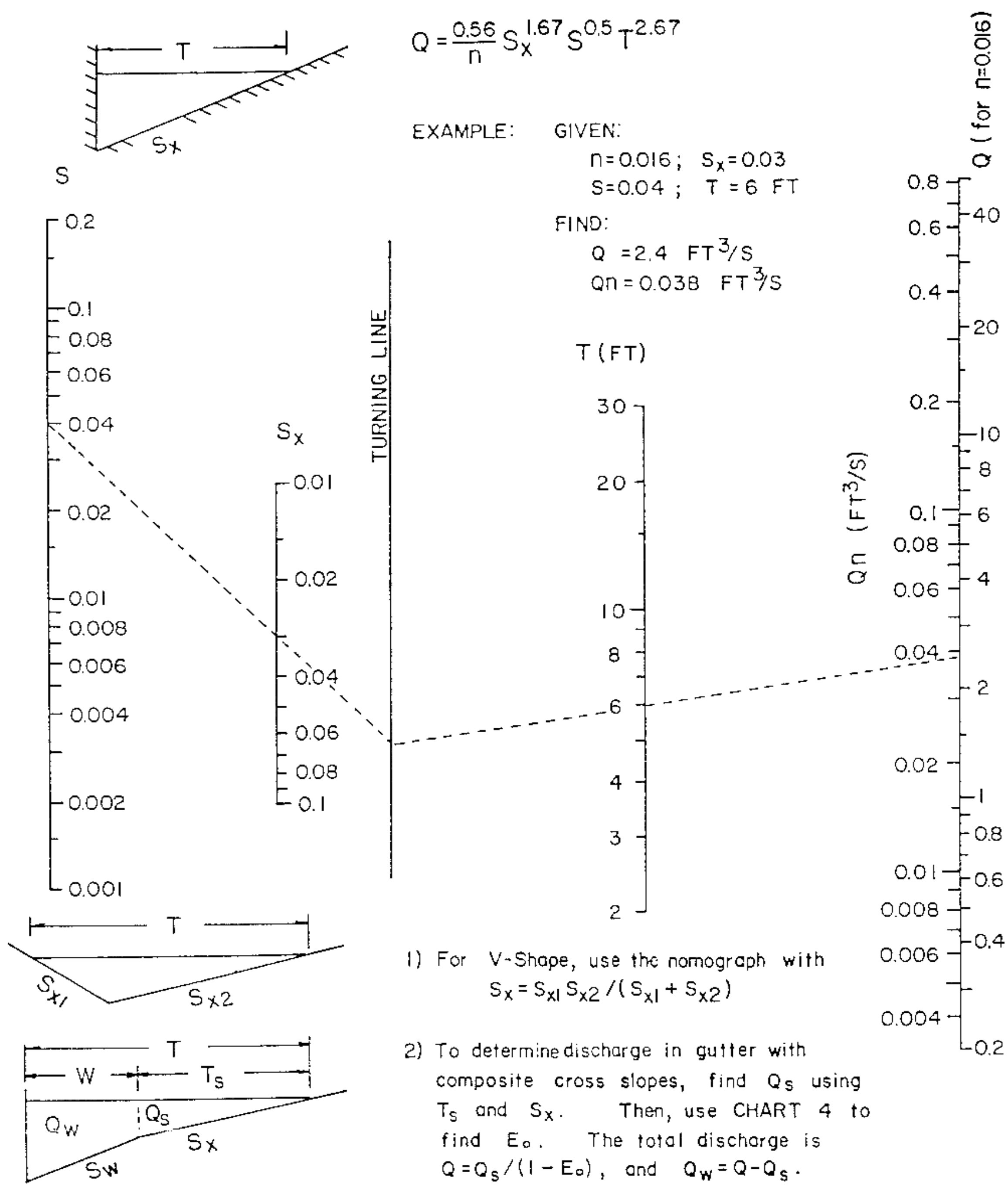


Figure 12-1 Flow In Triangular Gutter Sections

Source HEC-12

12.7 Gutter Flow Calculations (continued)

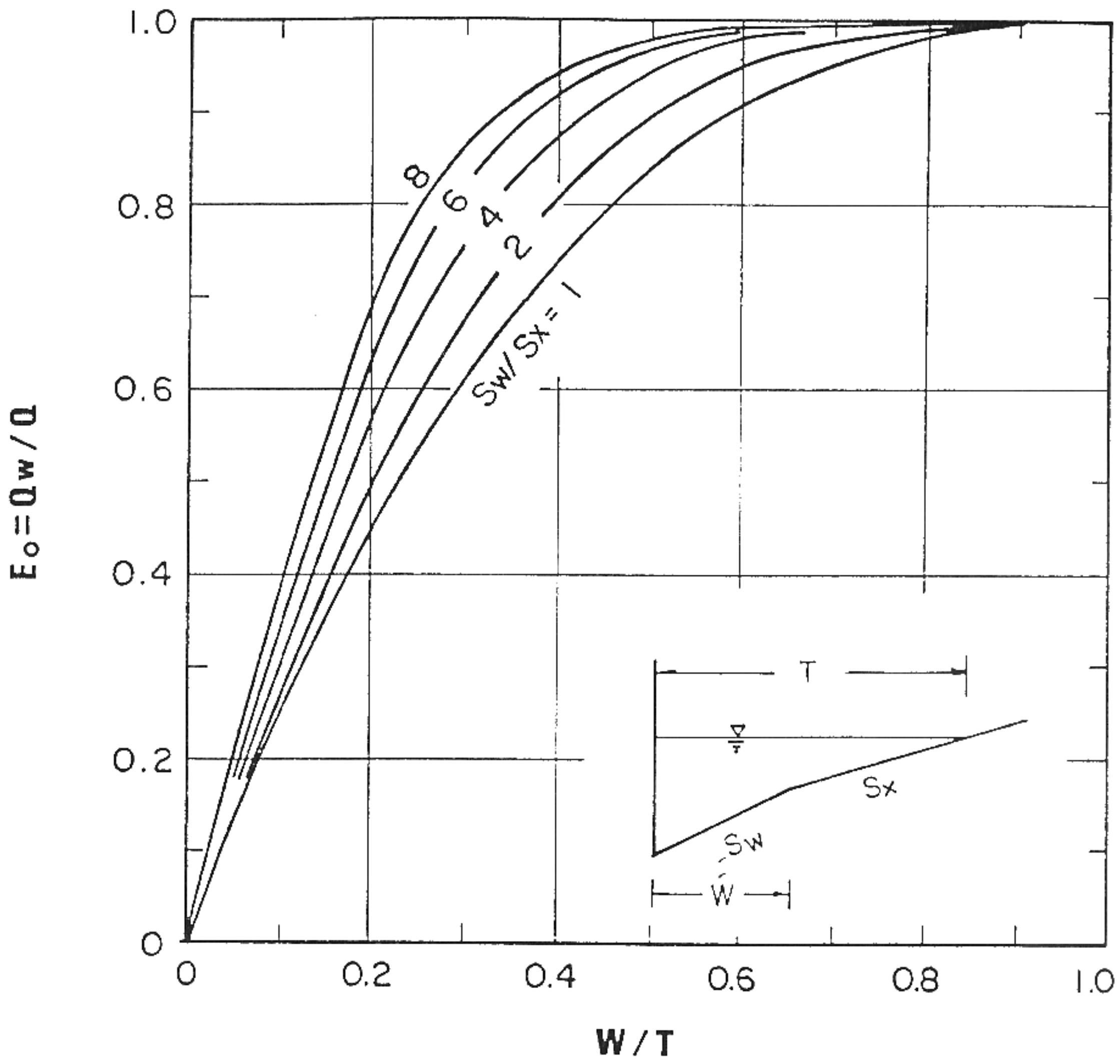


Figure 12-2 Ratio Of Frontal Flow To Total Gutter Flow

Source: HEC-12

12.7 Gutter Flow Calculations (continued)

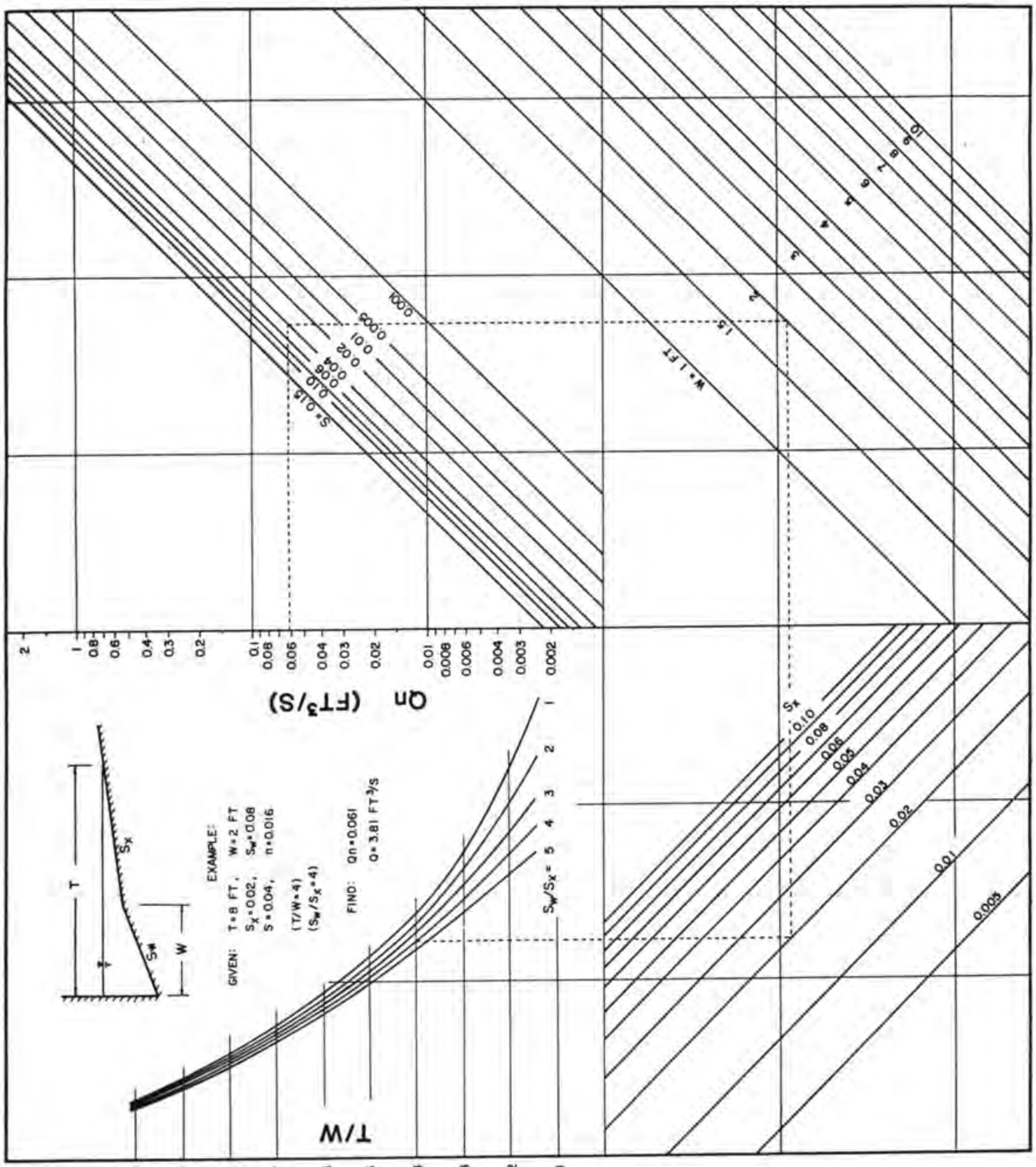


Figure 12-3 Flow In Composite Gutter Sections

Source: HEC 12

12.7 Gutter Flow Calculations (continued)

CONDITION 2: **Given spread (T), find gutter flow (Q)**. (continued)

Step 3: Calculate the ratios W/T and S_w/S_x, and from Figure 11-2, find the appropriate value of E_o, the ratio of Q_w/Q. W/T = 2.0/10.0 = 0.2; S_w/S_x = 0.06/0.04 = 1.5; From Figure 12-1 E_o = 0.46.

Step 4: Calculate the total gutter flow using the equation:

$$\mathbf{Q = Q_s/(1 - E_o)} \qquad \textbf{(12.6)}$$

Where: Q = gutter flow rate, ft^3/sec
Q_s = flow capacity of the gutter section above the depressed section, ft^3/sec
E_o = ratio of frontal flow to total gutter flow, Q_w/Q

Q = 3.0 / (1-0.46) = 5.55 ft^3/sec

Step 5: Calculate the gutter flow in width, W, using equation 12.5.

$Q_w = Q - Q_s$ = 5.55 – 3.0 = 2.25 ft^3/sec

NOTE: Figure 12-3 can also be used to calculate the flow in a composite gutter section.

12.7.5 V-Type Gutter Sections Procedures

Figure 12-1 can also be used to solve V Type channel problems. The spread, T, can be calculated for a given flow, Q, or the flow can be calculated for a given spread. This method can be used to calculate approximate flow conditions in the triangular channel adjacent to concrete median barriers. It assumes the flow is confined to the V channel with spread T_1.

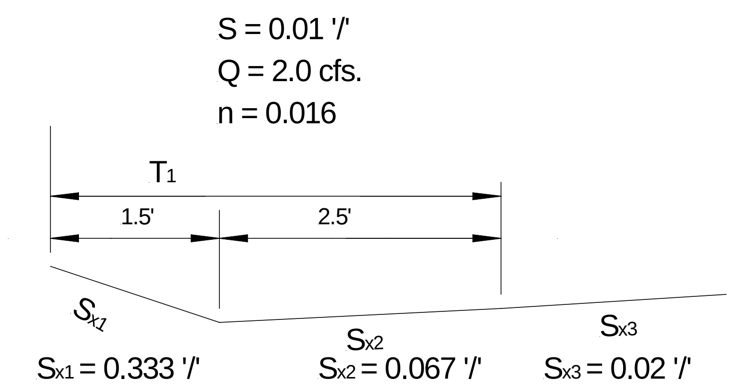


Figure 12-4 V Type Gutter

12.7 Gutter Flow Calculations (continued)

CONDITION 1: Given flow (Q), find spread (T).
Example: $S = 0.01$, $S_{x1} = 0.33$, $S_{x2} = 0.066$, $S_{x3} = 0.020$, $n = 0.016$, $Q = 2.0\ ft^3/s$, shoulder = 4.0 ft.

Step 1: Determine input parameters, including longitudinal slope, S, cross slope $S_x = S_{x1}S_{x2}/(S_{x1}+S_{x2})$, Manning's n, total flow (Q).

Step 2: Calculate S_x
$S_x = S_{x1}S_{x2}/(S_{x1} + S_{x2})$ $S_x = (0.33)(0.066)/(0.33 + 0.066) = 0.0555$

Step 3: Solve for T_1 using the nomograph on Figure 12-1.

T_1 is a hypothetical width that is correct if it is contained within S_{x1} and S_{x2}. From nomograph T_1 = 5.0 feet, however since the shoulder width of 4.0 ft is less than 5.0 ft, S_{x2} is 0.0666 and the pavement cross slope S_{x3} is 0.02, T will actually be greater than 5.0 ft; 5.0 – 1.5 = 3.5 ft which is > 2.5 ft., therefore the spread is greater than 5.0 ft.

Step 4: To find the actual spread, solve for depth at points B and C.
Point B: 3.5.0 ft @ 0.0666= 0.233 ft Point C: 0.233 - (2.5 @ 0.0666) = 0.066 ft

Step 5: Solve for the spread on the pavement. Pavement cross slope = 0.02.
$T_{0.015} = 0.066/0.02 = 3.325$ ft

Step 6: Find the actual total spread, T. T = 4.0 + 3.33 = 7.33 ft

CONDITION 2: Find Flow within Gutter.
Given depth (d), Find Flow (Q)
Example: $n = 0.016$, $S = 0.01$, $S_{x1} = 0.33$, $S_{x2} = 0.066$, $d= 0.166$ ft

Step 1: Determine input parameters such as longitudinal slope (S), Cross slope $(S_{x)} = S_{x1}S_{x2}/(S_{x1} + S_{x2})$, Manning's n and allowable spread.

Step 2: Calculate S_x
$S_x = S_{x1}S_{x2}/(S_{x1} + S_{x2}) = (0.33)(0.066)/(0.33 + 0.066) = 0.0555$

Step 3: Calculate T
$T= d/(S_{x1})+d/(S_{x2}) = 0.166 /(0.333)+0.166/(0.066) = 0.500+ = 3.00$

Step 4: Using Figure 12-1, Solve for Q
For T = 3.0 ft, $Q = 0.54\ ft^3/sec$
The equation shown on Figure 12-1 can also be used.
$Q= (0.56/0.016)*(0.0555)^{1.67}*(0.01)^{0.5}*(3.0)^{2.67} = 0.53\ ft^3/sec$

12.8 Inlets

12.8.1 General

Inlets are drainage structures utilized to collect surface water through grate, curb openings, or slotted drain and convey it to storm drains or directly outletting to culverts. This section will discuss the various types of inlets in use and recommend guidelines on the use of each type.

12.8.2 Types

Various types of inlets are in use; grates, curb openings and slotted drain. Curb opening inlets and slotted drain inlets are also used in combination with grate inlets. The gutter grade and cross slope and cross slope affect the portion of flow captured be each element. Grate inlets used at the downstream end of curb opening and slotted drain inlets provide access and increase the interception capacity for a given length of opening or slot. Combination inlets are desirable in sags because they can provide relief capacity in the event of plugging of the grate inlet.

12.8.2.1 Grate Inlets

These inlets consist of an opening in the gutter covered by one or more grates. On continuous grades they are most efficient in capturing frontal flow. In sag locations, they are more susceptible to clogging with debris, the use of standard grate inlets at sag points should be limited to minor sag point locations without debris potential. Special design (oversize) grate inlets or grate with curb opening can be utilized at major sag points if sufficient capacity is provided for clogging. In this case, flanking inlets are definitely recommended. **Grates shall be bicycle safe unless bike traffic is specifically excluded and structurally designed to handle the appropriate loads when subject to traffic.** See Sections 12.11.2 and 12.11.3 for capacity calculation methods.

12.8.2.2 Curb-Opening Inlets

These inlets are vertical openings in the curb. They are best suited for use at sag points since they can convey large quantities of water and debris. They are a viable alternative to grates in many locations where grates would be hazardous for pedestrians or bicyclists. They are not very efficient on steep continuous grades. **They shall not be used with pump station collection systems.** See Sections 12.11.4 and 12.11.5 for capacity calculation methods.

12.8.2.3 Slotted Drain Inlets

These inlets consist of a vertical slot opening in the gutter with bars perpendicular to the slot opening. For shallow flow slotted inlets function as weirs with flow entering from the side. They are used to intercept sheet flow and collect gutter flow with or without curbs. Grate inlets are used at the downstream end of slotted drain inlets to provide access and reduce the overall length of slotted drain. **Slotted drain inlets shall not be used for offsite collection nor where snow and ice are anticipated (above elevation 4000 ft. +/-).** See Sections 12.11.6 and 12.11.7 for capacity calculation methods.

12.8 Inlets (continued)

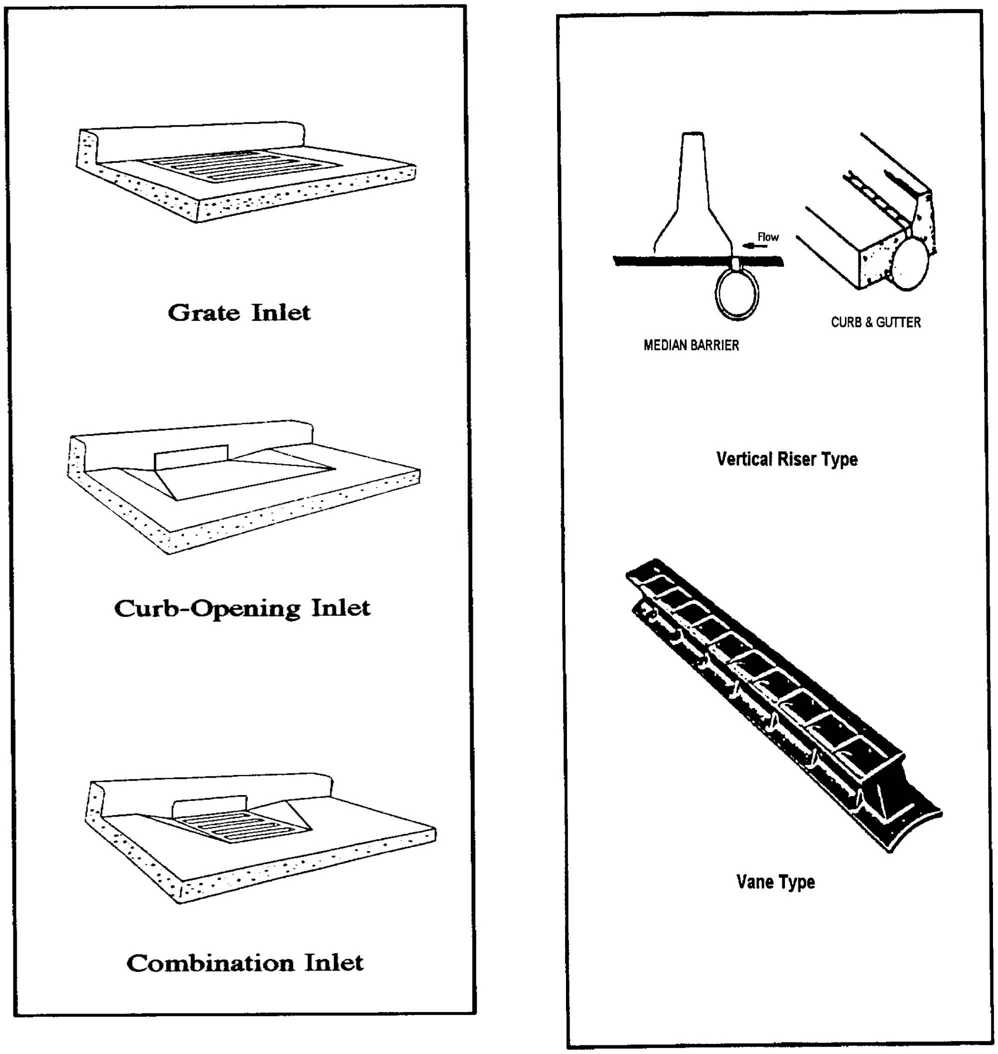


Figure 12-4 Inlets

12.8 Inlets (continued)

12.8.3 Inlet Locations

Inlets are required at locations needed to collect runoff within the design controls specified in the design criteria. In addition, there are a number of locations where inlets may be necessary with little regard to contributing drainage area. These locations should be marked on the plans prior to any computations regarding discharge, water spread, inlet capacity, or bypass-flow. Examples of such locations are as follows:

- Sag points in the gutter grade.
- Flanking inlets at sag points (see section 12.9.8)
- Upstream of median breaks, entrance/exit ramp gores, cross walks and street intersections.
- Immediately upstream and downstream of bridges.
- Immediately upstream of cross slope reversals.
- On side streets at intersections.
- At the end of channels in cut sections.
- Behind curbs, shoulders, or sidewalks to drain low areas.
- Where necessary to collect snow melt.

Inlets should not be located in the path where pedestrians are likely to walk.

12.9 Inlet Interception Calculations

12.9.1 Spacing

As discussed above a number of inlets are required to collect runoff at locations with little regard for contributing drainage area. These should be plotted on the plan first. Next, it is best to start locating inlets from the crest and working down grade to the sag points. The process is as follows: the location of the first inlet from the crest can be found by determining the length of pavement and the area back of the curb sloping toward the roadway that generates the design runoff. The design runoff can be computed as the maximum allowable flow in the curbed channel that will not exceed the design criteria. Where the contributing drainage area consists of a strip of land parallel to and including a portion of the highway, the first inlet can be calculated as using Equation 12.7, which is an alternate form of the Rational Equation.

$$L = \frac{43560\ Q_t}{C\ i\ W} \qquad \textbf{(12.7)}$$

Where: L = distance from the crest, ft
Q_t = maximum allowable flow, ft^3/sec
C = composite runoff coefficient for contributing drainage area
W = width of contributing drainage area, ft
i = rainfall intensity for design frequency, in/hr

If the drainage area contributing to the first inlet from the crest is irregular in shape, trial and error will be necessary to match a design flow with the maximum allowable flow.

12.9 Inlet Interception Calculations (continued)

12.9.1 Spacing (continued)

To space successive down grade inlets, it is necessary to compute the amount of flow which will be intercepted by the inlet, Q_i, and subtract it from the total gutter flow to compute the by-pass flow. The by-pass flow from the first inlet is added to the computed flow to the second inlet, the total of which must be less than the maximum allowable flow dictated by the criteria. Table 12-4 is an example of an inlet spacing computation sheet that can be utilized to record the spacing calculations.

Inlets on grade are usually designed for the following percentage capture:

Curb Opening Inlets---- 75%
Slotted Drain & Grate-- 90%

Inlets on grade are rarely designed to capture 100% of the flow; those instances are limited to locations where the consequence of by-pass is undesired; such as the superelevation is reversing or approaching crosswalks. Inlets that pass less than 0.5 cfs should be considered to meet this condition. Inlets that collect off-roadway flows should capture 100% of the flow that would otherwise flow onto the roadway.

12.9.2 Grate Inlets On Grade

The capacity of a grate inlet depends upon its geometry, cross slope, longitudinal slope, total gutter flow, depth of flow and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, a portion of the frontal flow may tend to splash over the end of the grate for some grates. Figure 12-6 can be utilized to determine splash-over velocities for the standard ADOT grate configurations and the portion of frontal flow intercepted by the grate. **Note that the parallel bar grates are the most efficient grates on steep slopes but are not bicycle safe.** Inlet interception capacity has been investigated by the FHWA. The grates tested in an FHWA research study are described in, "Drainage of Highway Pavements," HEC 12.

The ratio of frontal flow to total gutter flow, E_o, for straight cross slope is given by the following equation:

$$\mathbf{E_o = Q_w/Q = 1 - (1 - W/T)^{2.67}} \qquad \mathbf{(12.8)}$$

Where: Q = total gutter flow, ft^3/sec
Q_w = flow in width W, ft^3/sec
W = width of depressed gutter or grate, ft
T = total spread of water in the gutter, ft

12.9 Inlet Interception Calculations (continued)

Figure 12-2 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 is:

$$Q_s/Q = 1 - Q_w/Q = 1 - E_o \tag{12.9}$$

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the following equation:

$$R_f = 1 - 0.09(V - V_0) \tag{12.10}$$

Where: 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 12-6 provides a solution of equation 12.10 which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 12-6 is **total gutter flow** divided by the area of flow.

Figure 12-5 is a nomograph to solve for velocity in a triangular gutter section with known cross slope, slope and spread.

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

$$R_s = 1 / [1 + (0.15V^{1.8}/S_xL^{2.3})] \tag{12.11}$$

Where: V = velocity of flow in gutter, ft/sec
L = length of the grate, ft
S_x = cross slope, ft/ft

Figure 12-7 provides a solution to equation 12.11.

The efficiency, E, of a grate is expressed as:

$$E = R_fE_o + R_s(1 - E_o) \tag{12.12}$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q[R_fE_o + R_s(1 - E_o)] \tag{12.13}$$

12.9 Inlet Interception Calculations (continued)

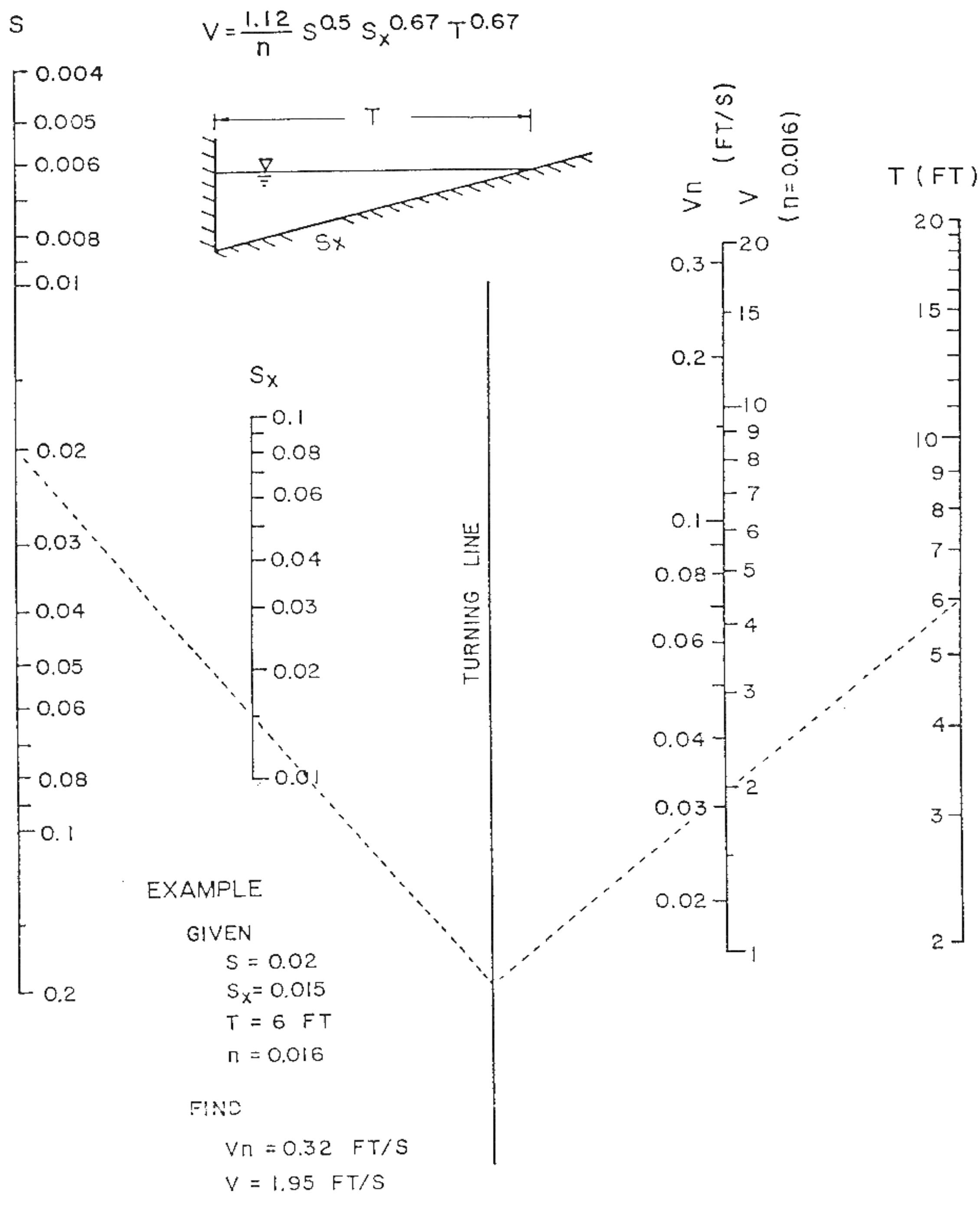


Figure 12-5 Velocity In Triangular Gutter Sections

Source: HEC-12

12.9 Inlet Interception Calculations (continued)

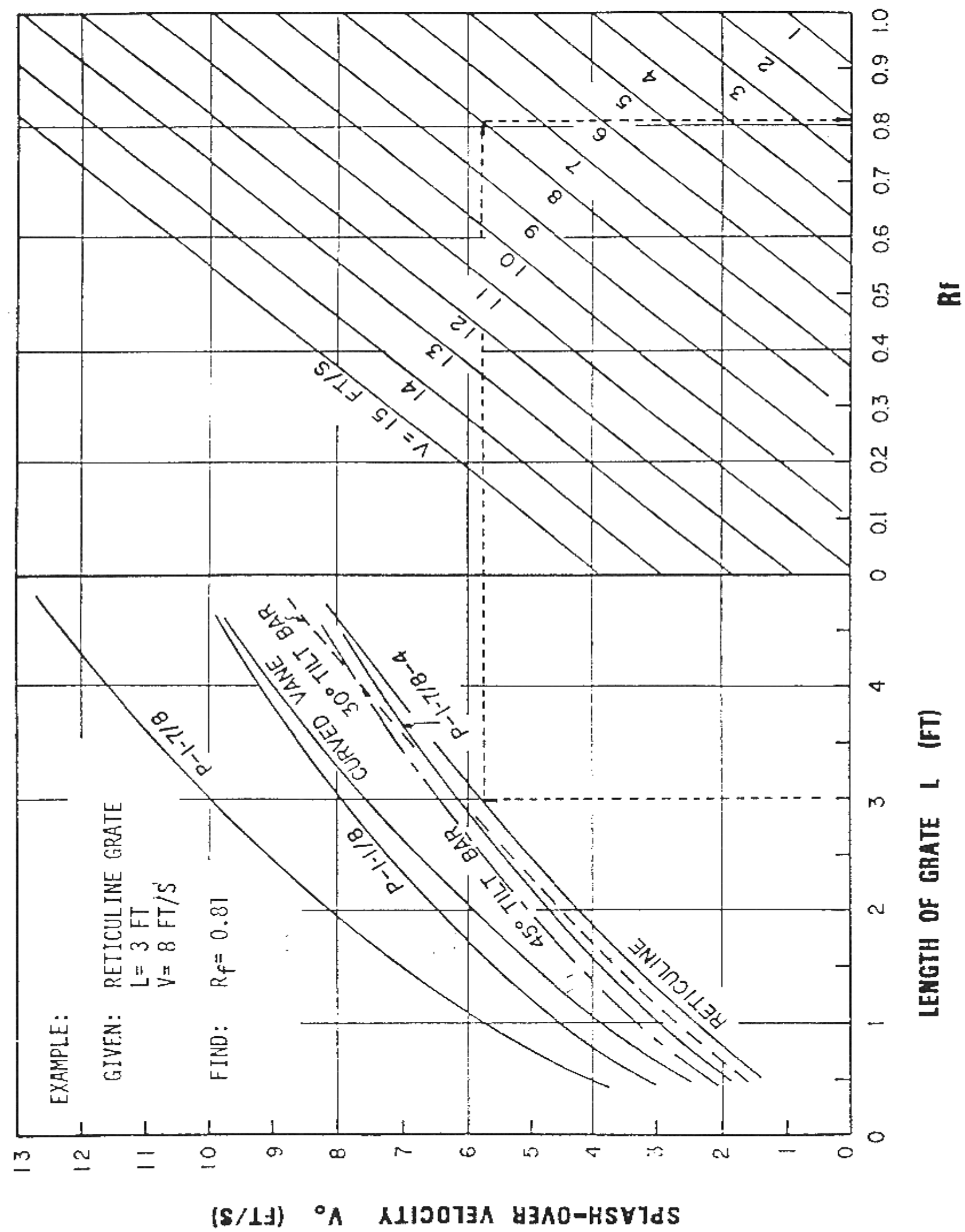


Figure 12-6 Grate Inlet Frontal Flow Interception Efficiency

Source: HEC-12

12.9 Inlet Interception Calculations (continued)

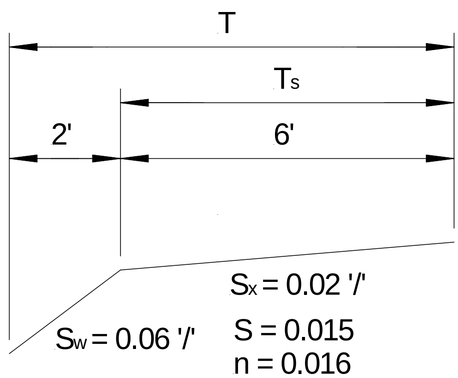
12.9.2 Grate Inlets On Grade (continued)

Example Problem

Given: Roadway in Chandler
Drainage Area: 40 ft landscape strip, C = 0.35, g= 1.5 %
2-12-ft Lanes @ 0.02 ft/ft and 2-ft gutter at 0.06 ft/ft, C=0.95
10-year design,

Allowable spread T = 2+12/2=8.0 ft, n = 0.016
g = 1.5, S_x = 0.02, S_S=0.02, S_w = 0.06

Find: Maximum allowable flow Q_T
Q_i intercepted by ADOT EF-1 grate
Q_r flowby
Location of first and second inlets from crest of hill



Sketch:

Solution:

1. Solve for Q_s using Fig. 12-1
   T_s = 6; Sx = 0.02; Q_s = 0.751 cfs
2. Use Figure 12-2 to find E_o
   S_w/S_x = 0.06/0.02 = 1.5; W/T = 2/8 = 0.25; $E_o = 0.64 = Q_w/Q_T$
3. Find total Q_T (maximum allowable flow)
   $Q_T = Q_s/(1-E_o)$ = 0.751/(1 - 0.64) = <u>2.09 cfs</u>
4. From Figure 12-5, V = 2.903 ft/s. For and ADOT EF grate, use the reticuline parameters. Splash over velocity is 6.18 ft/sec.
5. From Figure 12-6, R_f = 1.0; From Figure 12-7 R_s = 0.22
6. Using Equation 12.13
   $Q_i = Q_T [R_f E_o + R_s (1-E_o)]$
   $Q_i = 2.09[(1.0 \times 0.64) + 0.22(1 - 0.64)]$ = <u>1.50 cfs</u>
7. $Q_r = Q_T - Q_i$, Q_r = 2.09-1.50 = <u>0.59 cfs</u>

12.9 Inlet Interception Calculations (continued)

12.9.2 Grate Inlets On Grade (continued)

8. Locate first inlet from crest.

Using equation 12.7, $L = \frac{Q_t(43560)}{CiW}$

Where: L = distance from the crest, ft
Q_t = maximum allowable flow, cfs
C = composite runoff coefficient for contributing drainage area
W = width of contributing drainage area, ft
i = rainfall intensity for design frequency, in/hr

What length of roadway and embankment will generate a Q_t of 2.09 cfs.

To find i, first solve for t_c; For landscape area

C = 0.35 S = 0.5%,

Assume all travel time is in gutter. Gutter flow V = 2.9 ft/sec.

Try 700 ft, $t_c = 700/(2.9 \times 60) = 4.0$ min. Use min t_c = 10 min

For highway in Chandler, at t_c =10 min, i = 4.09 in/hr

Solve for weighted C value: C= [(40 × 0.35) + (26 x 0.95)]/66 = 0.586

L = (2.09)(43560)/(0.586)(4.09)(66) = 575 ft, since this is based on minimum travel time, use this value as max. spacing.

Place first inlet 575 ft from crest.

9. To locate second inlet:
Q_T = 2.09 cfs, $Q_{by\text{-}pass}$ = 1.60 cfs, $Q_{allowable}$ = 2.09-1.60 = 0.49 cfs
Assuming similar drainage area and t_c, i = 4.09 in/hr
L = 0.49(43560)/ (0.568)(4.09)(66) = 139 ft.

Place second inlet no more than 139 ft from first inlet.

12.9 Inlet Interception Calculations (continued)

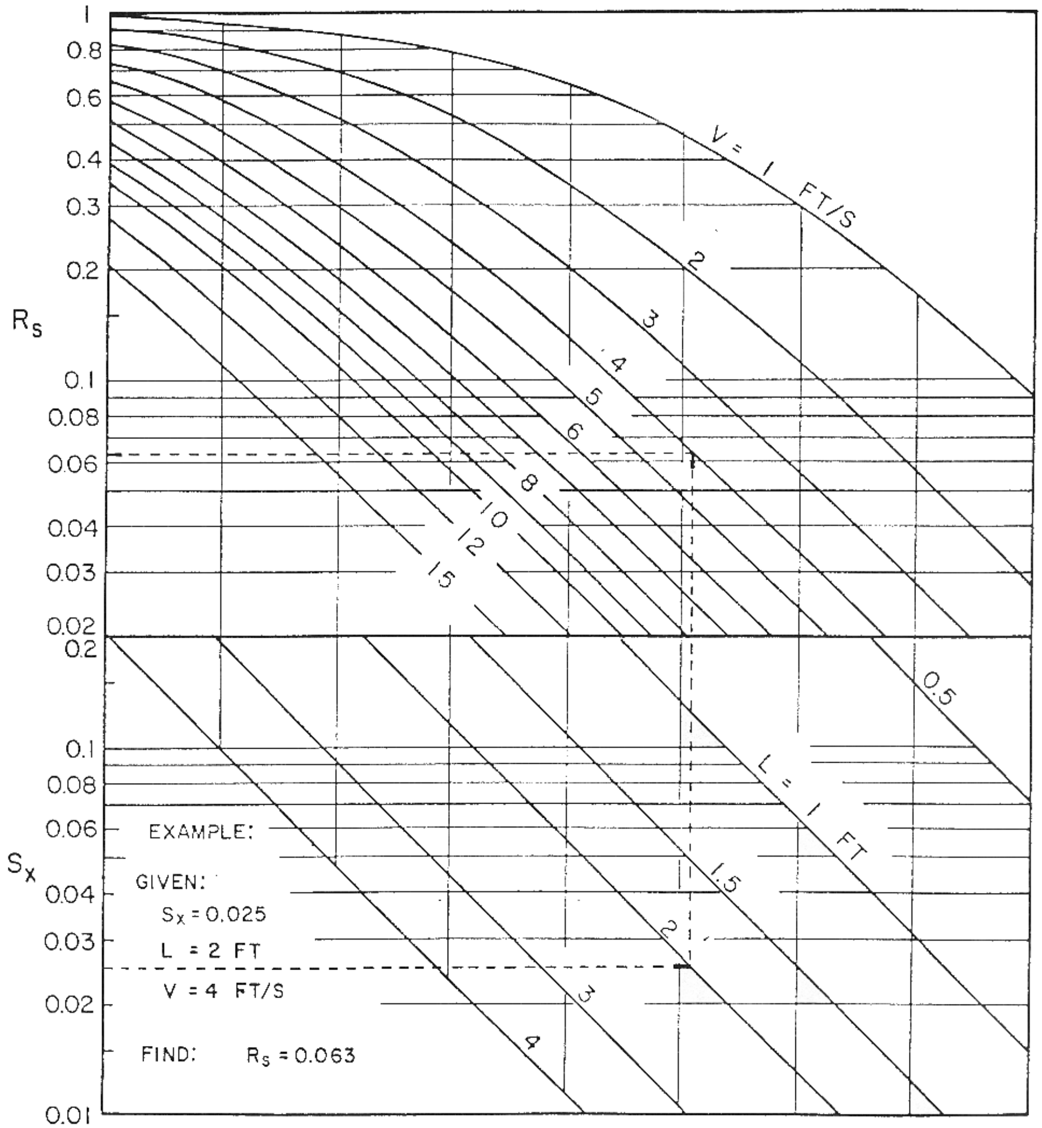


Figure 12-7 Grate Inlet Side Flow Interception Efficiency

Source: HEC-12

12.9 Inlet Interception Calculations (continued)

12.9.3 Grate Inlets In Sag

Although curb-opening inlets are generally more efficient to grate inlets at a sag, grate inlets can be used successfully. For minor sag points where debris potential is limited, grate inlets without a curb opening inlet can be utilized. An example of a minor sag point might be on a ramp as it joins a local street. Curb opening inlets with grates are preferred at sag points where debris is likely such as on a city street. For major sag points such as on divided high speed highways, a combination slotted drain and grate inlet is preferable to a grate inlet because of its hydraulic capacity. When grates are used, it is good practice to assume half the grate is clogged with debris. Where the inlets connect to a pump station, curb opening inlets should not be used.

Where significant ponding can occur, in locations such as underpasses and in sag vertical curves in depressed sections, it is good engineering practice to place a minimum of one flanking inlet on each side of the sag point inlet. The flanking inlets should be placed so they will limit spread on low gradient approaches to the low point and act in relief of the inlet at the low point if it should become clogged or if the allowable spread is exceeded. A further discussion and methodology is given in Section 12.12.8.

A grate inlet in a sag operates as a weir up to a depth of about 0.4 ft and as an orifice for depths greater than 1.4 ft. Between these depths, a transition from weir to orifice flow occurs. The capacity of a grate inlet operating as a weir is:

$$\mathbf{Q_{\mathit{i}}=3Pd^{1.5}} \qquad \textbf{(12.14)}$$

Where: 3.0 = weir coefficient
P = perimeter of grate excluding bar widths and side against curb, ft
d = depth of water at along edge of grate, for the sloping side it is measured at the c.g. of flow, ft

The capacity of a grate inlet operating as an orifice is:

$$\mathbf{Q_{\mathit{i}}=0.67A(64.4d)^{0.5}} \qquad \textbf{(12.15)}$$

Where: 0.67 = orifice coefficient
A = clear opening area of the grate, ft^2
d = depth of water at along sides of grate, measured form the c.g. of flow, ft
32.2 ft/sec^2 =G

$$\mathbf{Q_{\mathit{i}}=5.38A(d^{0.5})}$$

12.9 Inlet Interception Calculations (continued)

12.9.3 Grate Inlets In Sag (continued)

Figure 12-8 is a plot of equations 12.15 and 12.16 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 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.

Example Problem

The following example illustrates the use of Figure 12-8.

Given: A symmetrical sag vertical curve with equal bypass from inlets upgrade of the low point;
Using ADOT EF-1 grate, consider 50% clogging of the grate. Allowable spread = 8 ft.
What is Q captured.

S_x = 0.02 ft/ft, S_W=0.06 ft/ft w=2.0 ft. $T_{allow.}$ = 8 ft. n = 0.016

Find: Q without local depression.

Depth at gutter line=6(0.02)+2(0.06)=0.12+0.12=0.24ft, 2.88 inches.
Depth at face of grate = 6 (0.02)=0.12 ft., 1.44 inches = 0.12 ft.

Solution:

Center of gravity of flow is based on a depth of

d=1.44{1+[(1.44*0.33+2.88*0.67)/(1.44+2.88)]}/12
d=1.44{1+0.556)/12=2.24/12=0.187 ft., d< 0.4 ft, use weir equation.

$Q= 3(0.5)[2(2.0)(0.187^{1.5})+3.25(0.12^{1.5)}]=$
Q= 1.5[0.323+0.135]= 1.5[0.458]
Q= 0.687

The Q captured for 8 ft of spread is 0.687 cfs., if the approach flow is equal in both directions this can have a value of 0.343 cfs.

AASHTO geometric policy recommends a gradient of 0.3% within 50 ft of the level point in a sag vertical curve.
Check T at S = 0.003 for the design approach flow:
Q approach= 0.343cfs, S=0.003, T = 8.2 ft. Figure 12-1

Thus a single EF-1 grate, 50% clogged is adequate to intercept the design flow at a spread that does not exceed design spread, however the spread on the approaches to the low point will exceed design spread. However, the tendency of grate inlets to clog completely warrants consideration of a combination inlet, or curb-opening inlet in a sag where ponding can occur, and flanking inlets on the low gradient approaches.

12.9 Inlet Interception Calculations (continued)

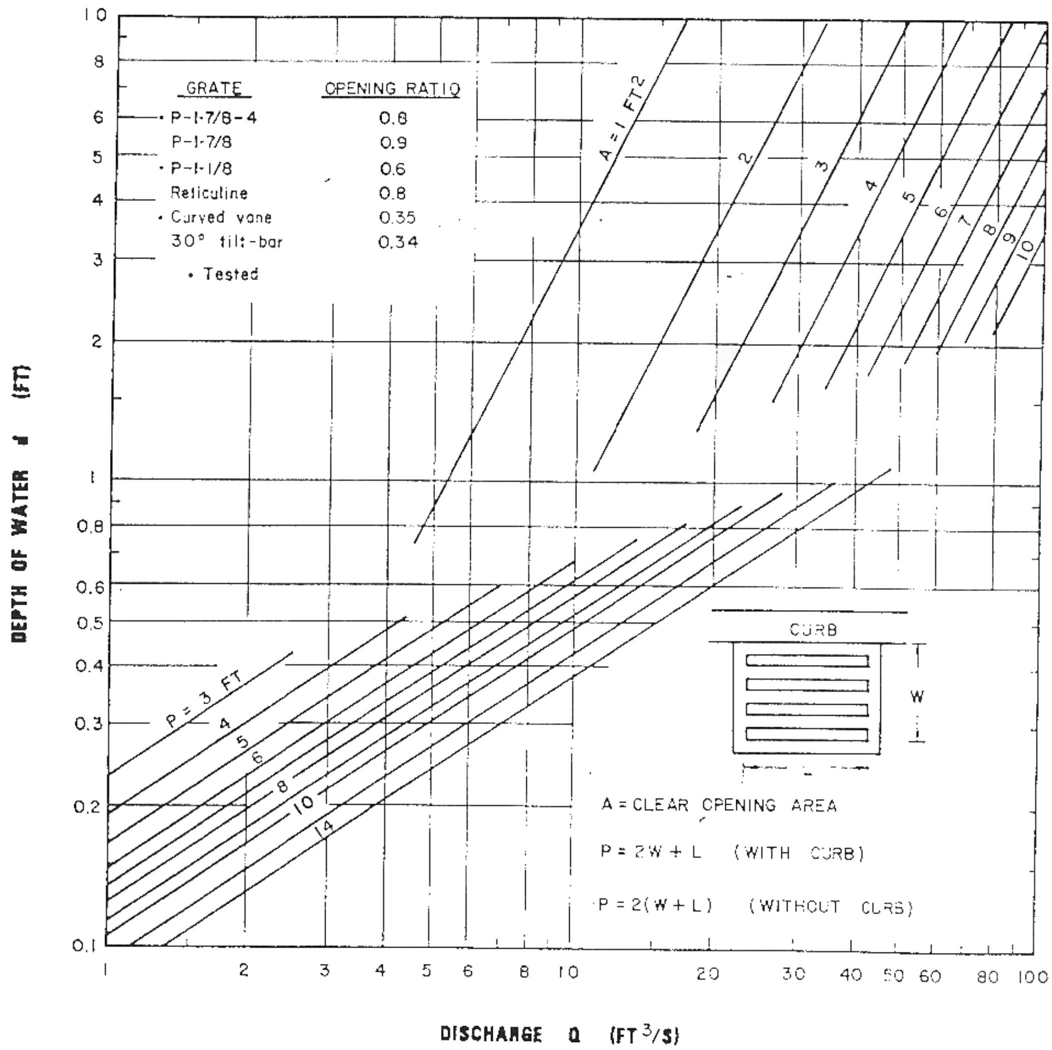


Figure 12-8 Grate Inlet Capacity In Sump Conditions

Source: HEC 12

12.9 Inlet Interception Calculations (continued)

12.9.4 Curb Opening Inlets On Grade

Curb-opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. However, their lack of filtering of trash makes them undesirable in pump station collection systems. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

$$\mathbf{L_T = 0.6Q^{0.42}S^{0.3}(1/nS_x)^{0.6}} \qquad \textbf{(12.16)}$$

Where: L_T = curb opening length required to intercept 100% of the gutter flow, ft

The length of inlet required for total interception by depressed curb-opening inlets or curb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in equation 12.16.

$$\mathbf{S_e = S_x + S'_wE_o} \qquad \textbf{(12.17)}$$

Where: S'_w = cross slope of the gutter measured from the cross slope of the pavement,
$S'_w = (a/12W)$, ft/ft
a = gutter depression, in
E_0 = ratio of flow in the depressed section to total gutter flow. It is determined by the gutter configuration upstream of the inlet.

The efficiency of curb-opening inlets shorter than the length required for total interception is expressed by:

$$\mathbf{E = 1 - (1 - L/L_T)^{1.8}} \qquad \textbf{(12.18)}$$

Where: L = curb-opening length, ft

Figure 12-9 is a nomograph for the solution of equation 12.17, and Figure 12-10 provides a solution of equation 12.18.

12.9 Inlet Interception Calculations (continued)

12.9.4 Curb Opening Inlets On Grade (continued)

Example Problem

The following example illustrates the use of this procedure.

Given: S_x = 0.03 ft/ft S = 0.035 ft/ft n = 0.016 Q = 5 ft^3/s
Find: (1) Q_i for a 11-ft curb-opening inlet, uniform cross slope
(2) Q_i for a 11-ft curb-open inlet with composite cross slope, W=2ft, Sw=Sx+2/24=0.1183
(3) Q_i for a 11-ft curb-open inlet with 2" depression and composite cross slope, a = 2 in, W = 2 ft

Solution:
(1) Q_i for a 11-ft curb-opening inlet, uniform cross slope

For uniform cross-slope, $T=\dfrac{\{(Qn/0.56)\}}{Sx^{1.67}\,S^{0.5}}^{0.375}$

$$T=\frac{(5(0.016)/0.56)}{(0.030)^{1.67}\,(S0.035)^{0.5}}^{0.375}$$

$$T=\frac{(0.1428)}{(2.8x10^{-3})(1.871x10^{-1})}^{0.375}$$

T= 8.2 ft

$$\mathbf{L_T = 0.6Q^{0.42}S^{0.3}(1/nS_x)^{0.6}} \qquad \mathbf{(12.16)}$$

Lt=42.3
L/L_T = 11/42.3 = 0.26

$$\mathbf{E = 1 - (1 - L/L_T)^{1.8}} \qquad \mathbf{(12.18)}$$

E= $1-(1-0.26)^{1.8}$
E=0.42
Q_i = EQ = 0.42 × 5 = $\underline{\underline{2.1\ ft^3/s}}$

Graphical solution
From Figure 12-1, T = 8.1 ft
From Figure 12-9, L_T = 42.3 ft
From Figure 12-10, E = 0.42
Q_i = EQ = 0.42 × 5 = $\underline{\underline{2.1\ ft^3/s}}$

12.9 Inlet Interception Calculations (continued)

12.9.4 Curb Opening Inlets On Grade (continued)

(2) Q_i for a 11-ft curb-open inlet with composite cross slope, W=2ft, Sw=Sx+2/24=0.1183
See page 23 for spread in a composite section.

Sw/Sx=0.1183/0.03=3.94
Try Q_s = 1.18; then Q_w=5.0-1.18 = 3.82; and Q_w/Q=3.82/5.0 = 0.764;
From Figure 12-2, W/T = 0.30, then T = 2.0/0.30 = 6.67 ft. and T_s = 6.67 – 2.0 = 4.67 ft.
From Fig 12-1, Qn=0.018,Q_s = 0.018/0.016=1.13 ft^3/s OK

From above, Q_w/Q= E_o = 0.764,

$$\mathbf{S_{e=}\ S_{x+}\ S'_w{*}E_0} \qquad \textbf{(12.17)}$$

$S_{e=}$ 0.03+0.083*0.764= 0.0934

$$\mathbf{L_T = 0.6Q^{0.42}S^{0.3}(1/nS_e)^{0.6}} \qquad \textbf{(12.16)}$$

Lt= 21.4; L/Lt=11.0/21.4=0.51

$$\mathbf{E = 1 - (1 - L/L_T)^{1.8}} \qquad \textbf{(12.18)}$$

E= 1-(1-0.51)$^{1.8}$
E=0.73
then Q_i = 0.73 × 5 = $\underline{\underline{3.65\ ft^3/s}}$

Graphic Solution:
From Figure 12-9 L_T = 21.4 then L/L_T = 11/21.4 = 0.51

From Figure 12-10 E = 0.73, then Q_i = 0.73 × 5 = $\underline{\underline{3.65\ ft^3/s}}$

(3) Q_i for a 11-ft curb-open inlet with 2" depression and composite cross slope, a = 2 in, W = 2 ft
The approach spread is the same as for example 2, T=6.7 ft. At the inlet the water will redistribute, so must redo the spread computation for determination of Lt.

At inlet Sw=0.1133+0.0833=0.1966; Sw/Sx=0.1966/0.03=6.55
Try Q_s = 0.29; then Q_w=5.0-0.29 = 4.71; and Q_w/Q=4.71/5.0 = 0.942;
From Figure 12-2, W/T = 0.41, then T = 2.0/0.41 = 4.88 ft. and T_s = 4.88 – 2.0 = 2.88 ft.
From Fig 12-1, Qn=0.0048,Q_s = 0.0048/0.016=0.3 ft^3/s OK

From above, Q_w/Q= E_o = 0.942,

$$\mathbf{S_{e=}\ S_{x+}\ S'_w{*}E_0} \qquad \textbf{(12.17)}$$

$S_{e=}$ 0.03+0.1133*0.942= 0.1367

$$\mathbf{L_T = 0.6Q^{0.42}S^{0.3}(1/nS_e)^{0.6}} \qquad \textbf{(12.16)}$$

Lt= 17.0; L/Lt=11.0/17.0=0.64

12.9 Inlet Interception Calculations (continued)

12.9.4 Curb Opening Inlets On Grade (continued)

$$\mathbf{E = 1 - (1 - L/L_T)^{1.8}} \qquad \mathbf{(12.18)}$$

$E = 1-(1-0.64)^{1.8}$
$E=0.84$
then $Q_i = 0.84 \times 5 = \underline{\underline{4.2\ ft^3/s}}$

$Qn = 5 \times 0.016 = 0.08\ ft^3/s$
$S_w/S_x = (0.03 + 0.0833+0.0833)/0.03 =6.55$
From Figure 12-3, T/W = 2.6 and T = 5.2 ft
Then W/T (Depress) = 2/5.2 = 0.385
From Figure 12-2, $E_o = 0.91$
$S_e = S_x + S'_w E_0 = 0.03 + 0.1666(0.91) = 0.182$

Graphic Solution:
From Figure 12-9 $L_T = 17.0$ then $L/L_T = 11/17 = 0.64$

From Figure 12-10 E = 0.84, then $Q_i = 0.84 \times 5 = \underline{\underline{4.2\ ft^3/s}}$

12.9.5 Curb Opening Inlets In Sag

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

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

$$\mathbf{Q_i = 2.3\ (L + 1.8\ W)\ d^{1.5}} \qquad \mathbf{(12.19)}$$

Where:
2.3 = curb-opening coefficient
L = length of curb opening, ft
W = width of depression, ft
D = depth of water at curb measured from the normal cross slope gutter flow line, ft

See Figure 12-11 for a definition sketch.

The weir equation for curb-opening inlets without depression becomes:

$$\mathbf{Qi = 2.3\ Ld^{1.5}} \qquad \mathbf{(12.20)}$$

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

12.9 Inlet Interception Calculations (continued)

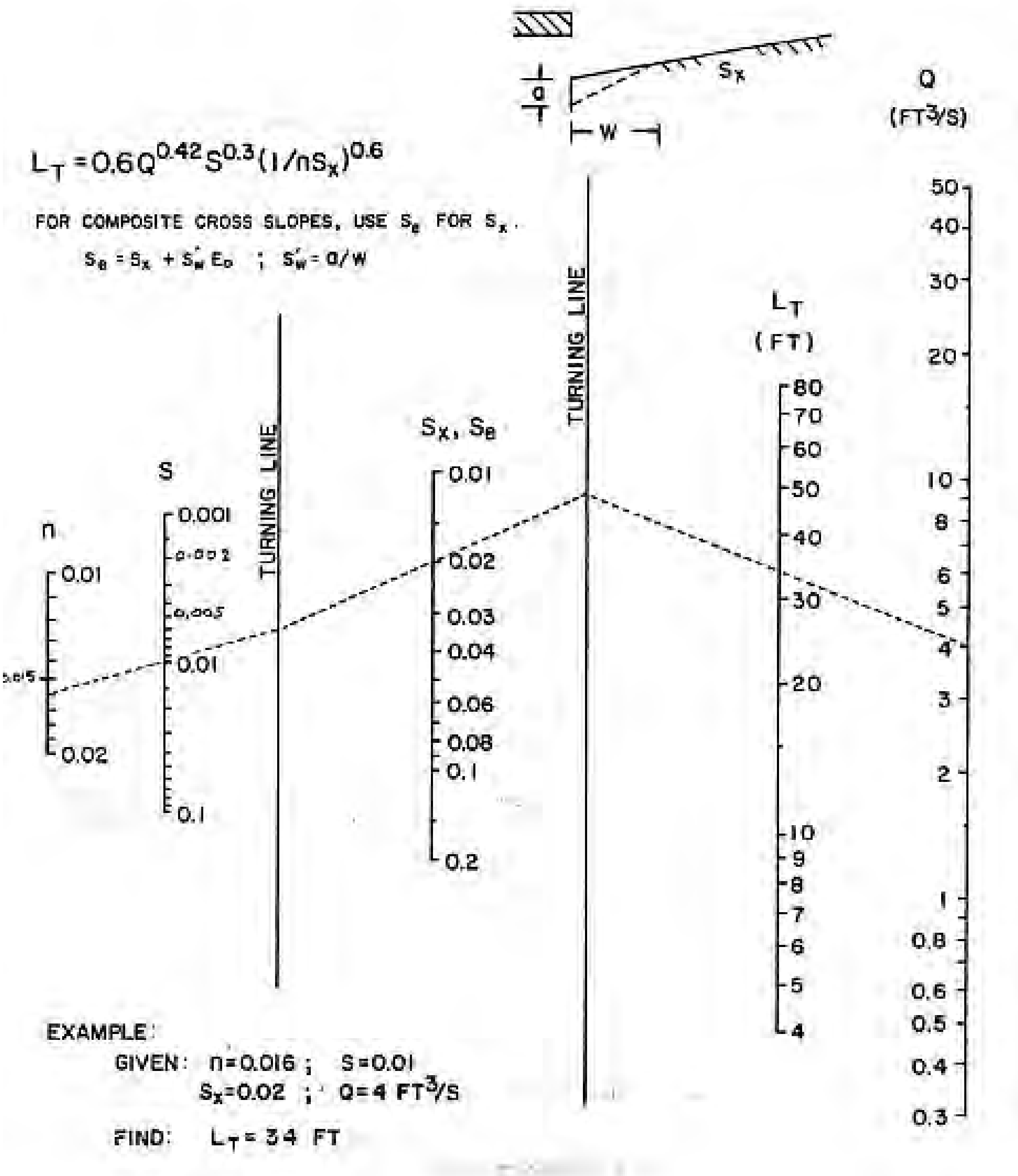


Figure 12-9 Curb-Opening And Longitudinal Slotted Drain

Inlet Length For Total Interception

Source: HEC-12

12.9 Inlet Interception Calculations (continued)

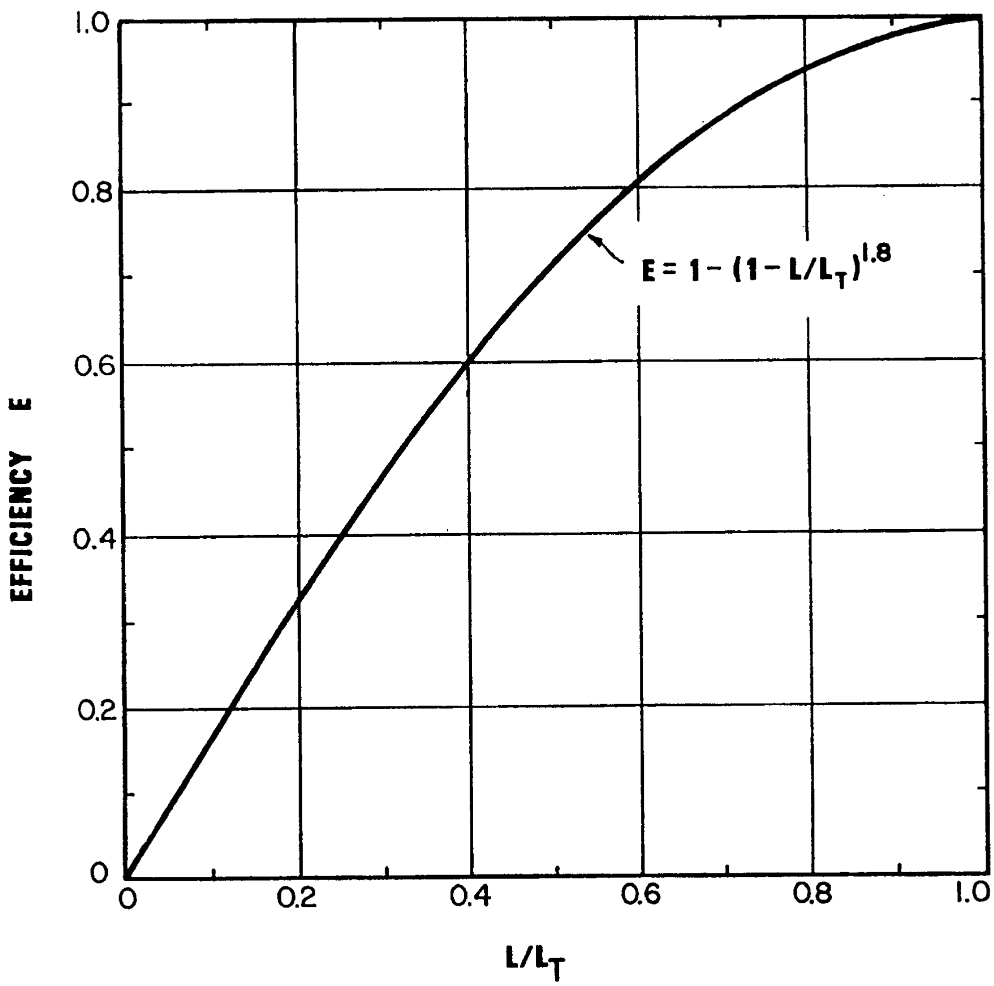


Figure 12-10 Curb-Opening And Slotted Drain Inlet Interception Efficiency

Source: HEC-12

12.9 Inlet Interception Calculations (continued)

12.9.5 Curb Opening Inlets In Sag (continued)

Curb-opening inlets operate as orifices at depths greater than approximately 1.4 × height of curb-opening. The interception capacity can be computed by:

$$Q_i = 0.67\ A[2g(d_i - h/2)]^{0.5} \qquad (12.21)$$

Where: 0.67 = orifice coefficient
H = height of curb-opening orifice, ft
A = clear area of opening, ft^2
d_i = depth at lip of curb opening, ft

Note: Equation 12.21 is applicable to depressed and undepressed curb-opening inlets and the depth at the inlet includes any gutter depression.

Example Problem:

Given: Curb-opening inlet in a sump location
L = 7.0 ft h = 5 inches

(1) Undepressed curb opening
$S_x = 0.03$ T = 8 ft

(2) Depressed curb opening
$S_x = 0.03$ W = 2.0 ft
a = 2 inches T = 8 ft.

Find: Q_i

Solution:

(1) $d = TS_s = 8 \times 0.03 = 0.24$ ft (2.88 inches) d < h therefore weir controls
$Q_i = C_W\ L\ d^{1.5} = 3.0 \times 7 \times 0.24^{1.5} = 2.9\ ft^3/s$

(2) d = 8(0.03)+2/24=0.24+0.083=0.323 ft (4.88 in.) < (1.4 h) = (1.4*7/12)=0.6 ft, therefore weir controls.

$$Q_i = 2.3\ (L + 1.8\ W)\ d^{1.5} \qquad (12.19)$$

P = L + 1.8W = 7+ 1.8(2) = 10.6 ft

$Q_i = 3.0 \times 10.6 \times 0.323^{1.5} = 5.83\ ft^3/s$ (Figure 12-11)

At d = 0.323 ft, the depressed curb-opening inlet has about 100% more capacity than an inlet without depression. In practice, the flow rate would be known and the depth at the curb would be unknown.

12.9 Inlet Interception Calculations (continued)

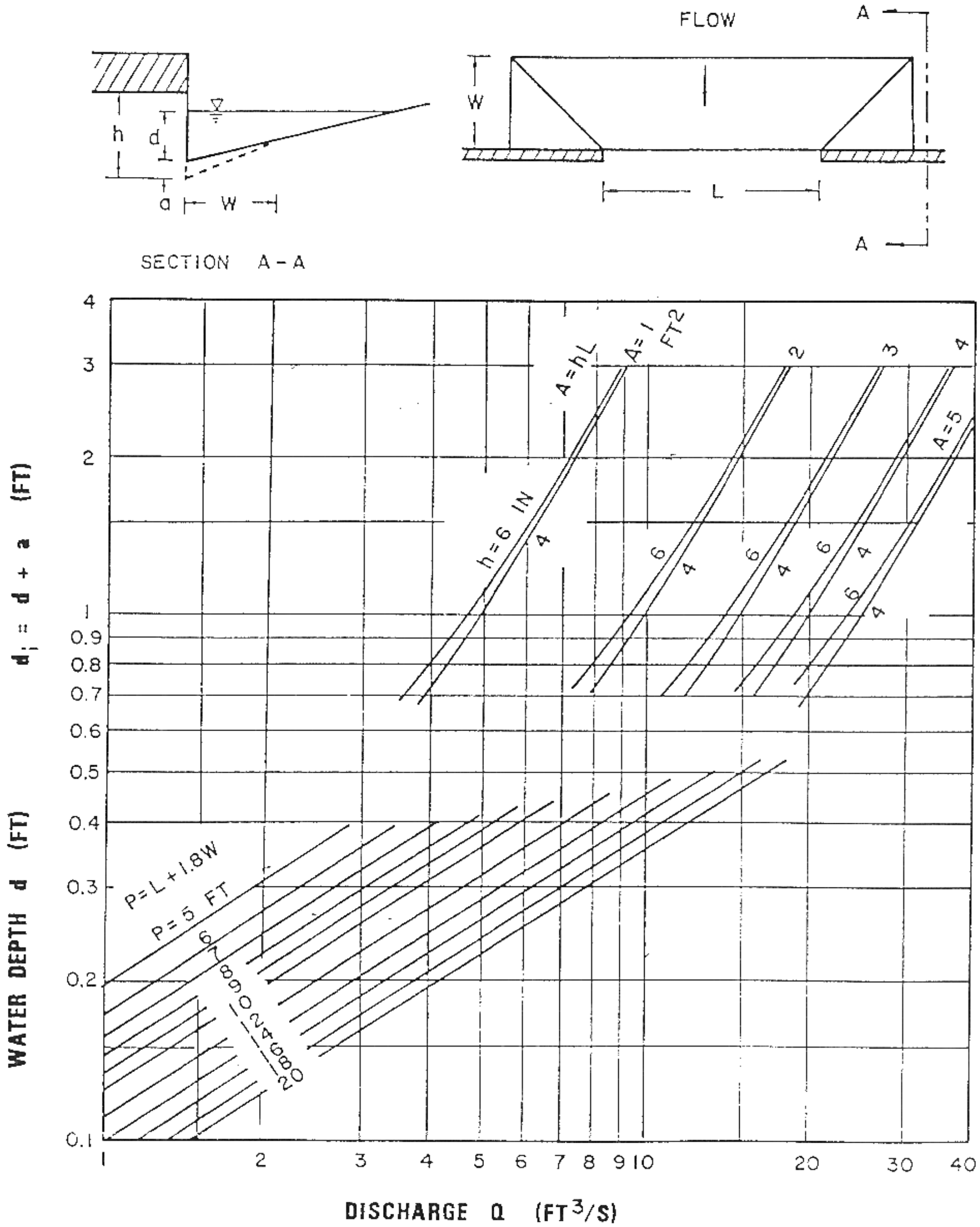


Figure 12-11 Depressed Curb-Opening Inlet Capacity In Sump Locations

Source: HEC-12

12.9 Inlet Interception Calculations (continued)

12.9.6 Slotted Inlets On Grade

Slotted inlets are effective pavement drainage inlets that have a variety of applications. They can be used on curbed or uncurbed sections and offer little interference to traffic operations. They can be placed longitudinally in the gutter or transversely to the gutter. Slotted inlets shall be connected to a clean-out structure, typically a standard catch basin, so they will be accessible to maintenance forces.

12.9.6.1 Longitudinal Placement

Flow interception by slotted inlets and curb-opening inlets 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. Slotted inlets may have economic advantages in some cases and could be very useful on widening and safety projects where right of way is narrow and existing inlet capacity must be supplemented. In some cases, curbs can be eliminated as a result of utilizing slotted inlets.

The length of a slotted inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

$$Lt=0.6Q^{0.42}S^{0.3}(1/nS_x)^{0.6} \qquad (12.22)$$

Where: L_T = slotted inlet length required to intercept 100% of the gutter flow, ft

The slot width must be at least 1.75 in for equation 12.23 to be valid.

The efficiency of slotted inlets shorter than the length required for total interception is expressed by:

$$E=1-(1-L/L_t)^{1.8} \qquad (12.23)$$

Where: L = slotted inlet length, ft

Figure 12-9 is a nomograph for the solution of equation 12.23, and Figure 12-10 provides a solution of equation 12.24.

The length of inlet required for total interception by a slotted inlet in a composite section can be found by the use of an equivalent cross slope, S_e, in equation 12.23.

$$S_e = S_x + S'_w E_o \qquad (12.17)$$

Where: S_x = pavement cross slope, ft/ft
S_w = gutter cross slope, ft/ft
S'_w = S_w - S_x
E_0 = ratio of flow in the depressed gutter to total gutter flow, Q_w/Q (See Figure 12-2)

Note that the same equations are used for both slotted inlets and curb opening inlets.

12.9 Inlet Interception Calculations (continued)

12.9.6 Slotted Inlets On Grade (continued)

Example Problem

Given: Longitudinal placement of slotted inlet adjacent to curb.
g = 1% Allowable spread = 10 ft n = 0.016
(1) Uniform cross slope, $S_x = 0.02$
(2) Composite cross slope, $S_x = 0.02$, $S_w = 0.06$, W=2.0
(3) Increase g to 3% and solve for (1) and (2)

Find: (1) Maximum allowable Q
Q_i for a 10 feet slotted inlet on straight cross slope.
(2) Maximum allowable Q
Q_i for a 10 feet slotted inlet on composite cross slope.
(3) Same as above with profile grade increased to 3%.

Solution:

(1) For T = 10 ft, Max Q = 2.394 cfs from Figure 12-1
$L_T = 27.2$ ft from Figure 12-9 $L/L_T = 10/27.2 = 0.368$
E = 0.56 from Figure 12-10; $Q_i = EQ = 0.56 \times 2.394 =$ 1.35 cfs intercepted

(2) $Q_s = 1.321$ cfs from Figure 12-1; W/T = 2.0/10.0= 0.2
$S_w/S_x = 0.06/0.02 = 3$; $E_o = 0.53$ from Figure 12-2
Max $Q = Q_s/(1 - E_o) = 1.321/(1 - 0.53) =$ 2.81 cfs
$S'_w = S_w - S_x = 0.06 - 0.02 = 0.04$
$S_e = S_x + S'_w E_o = 0.02 + (0.04 \times 0.52) = 0.041$
$L_T = 18.9$ ft from Figure 12-9 $L/L_T = 10/18.9 = 0.53$ E = 0.74 from
Figure 12-10; then $Q_i = EQ = 0.74 \times 2.81 =$ 2.08 cfs intercepted

The slotted inlet in the composite gutter section will intercept 1.55 times the flow intercepted by the slotted inlet in the uniform section.

(3) From a similar analysis for g = 3%
Uniform Section: Max Q = 4.15 cfs and $Q_i = 1.43$ cfs
Composite Section: Max Q = 4.92 cfs and $Q_i = 2.35$ cfs

12.9 Inlet Interception Calculations (continued)

12.9.7 Slotted Inlets In Sag

The use of slotted drain inlets in sag configurations is generally discouraged because of the propensity of such inlets to intercept debris in sags. However, there may be locations where it is desirable to supplement an existing low point inlet with the use of a slotted drain. Slotted inlets in sag locations perform as weirs to depths of about 2.5 inches, dependent on slot width and length. At depths greater than about 5 inches, they perform as orifices. Between these depths, flow is in a transition stage. The interception capacity of a slotted inlet operating as an orifice can be computed by the following equation:

$$\mathbf{Q_i = 0.8LW(2gd)^{0.5}} \qquad \textbf{(12.24)}$$

Where: W = width of slot, ft
L = length of slot, ft
d = depth of water at slot, ft
g = 32.2 ft/sec^2

For a slot width of 1.75 inches, the above equation becomes:

$$\mathbf{Q_i = 0.94Ld^{0.5}} \qquad \textbf{(12.25)}$$

The interception capacity of slotted inlets at depths between 0.2 ft and 0.4 ft can be computed by use of the orifice equation. The orifice coefficient varies with depth, slot width, and the length of slotted inlet. Figure 12-12 provides solutions for weir flow and a plot representing data at depths between weir and orifice flow.

12.9.8 Flanking Inlets

At major sag points where significant ponding may occur, such as underpasses or sag vertical curves in depressed sections, it is recommended practice to place a minimum of one flanking inlet on each side of the inlet at the sag point. The flanking inlets should be placed to act in relief of the sag inlet if it should become clogged. Table 12-3 shows the spacing required for various depths at curb criteria and vertical curve lengths defined by K = L/A, where L is the length of the vertical curve in meters and A is the algebraic difference in approach grades. The AASHTO policy on geometrics specifies maximum K values for various design speeds and a maximum K of 51 considering drainage.

Example Problem

Given: K = 130 ft/%, S_x = 0.04, allowable spread is 10 ft

Find: Location of flanking inlets that will function in relief of the inlet at the low point when the inlet at the low point is clogged.

d at sag = 0.4 ft., d at flanking inlet = 0.275 ft.

12.9 Inlet Interception Calculations (continued)

12.9.8 Flanking Inlets (continued)

Solution:

(1) Depth over flanking inlet to carry one-half of the design flow equals 0.63(0.4 ft) = 0.275 ft

(2) Depth from bottom of sag to flanking inlet - 0.4 ft - .275 ft = 0.125 ft

(3) Spacing of flanking inlet =58 ft (from Table 12-3, using d = 0.125 ft).
d= depth at curb line.

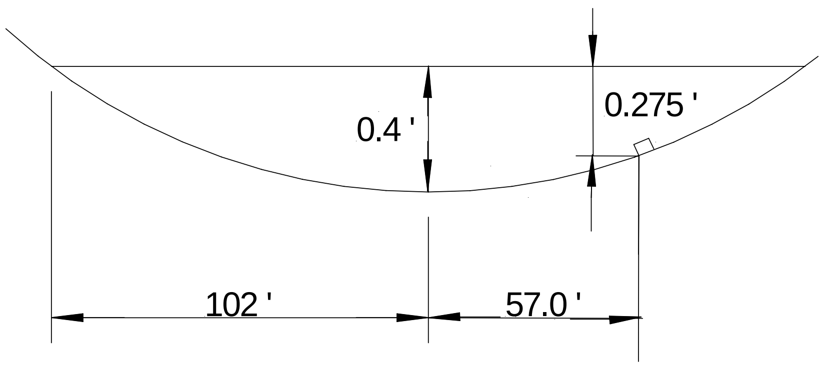


Table 12-3 Flanking Inlet Locations

Distance to flanking inlet in sag vertical curve locations using depth at curb criteria (ft).												
d↓ K→	20	30	40	50	70	90	110	130	160	165	180	220
0.1	20	24	28	32	37	42	47	51	57	57	60	66
0.2	28	35	40	45	53	60	66	72	80	81	85	94
0.3	35	42	49	55	65	73	81	88	98	99	104	115
0.4	40	49	57	63	75	85	84	102	113	115	120	133
0.5	45	55	63	69	84	95	105	114	126	128	134	148
0.5	49	60	69	77	92	104	115	12	139	141	147	162
0.7	53	65	75	84	99	112	12	135	150	152	159	175
0.8	57	69	80	89	106	120	133	144	160	162	170	188

NOTES:1. $\times = (200dK)^{0.5}$, where × = distance from the low point (ft)
2. Drainage maximum K = 165 ft/%.
3. d = depth at curb (ft).

12.9 Inlet Interception Calculations (continued)

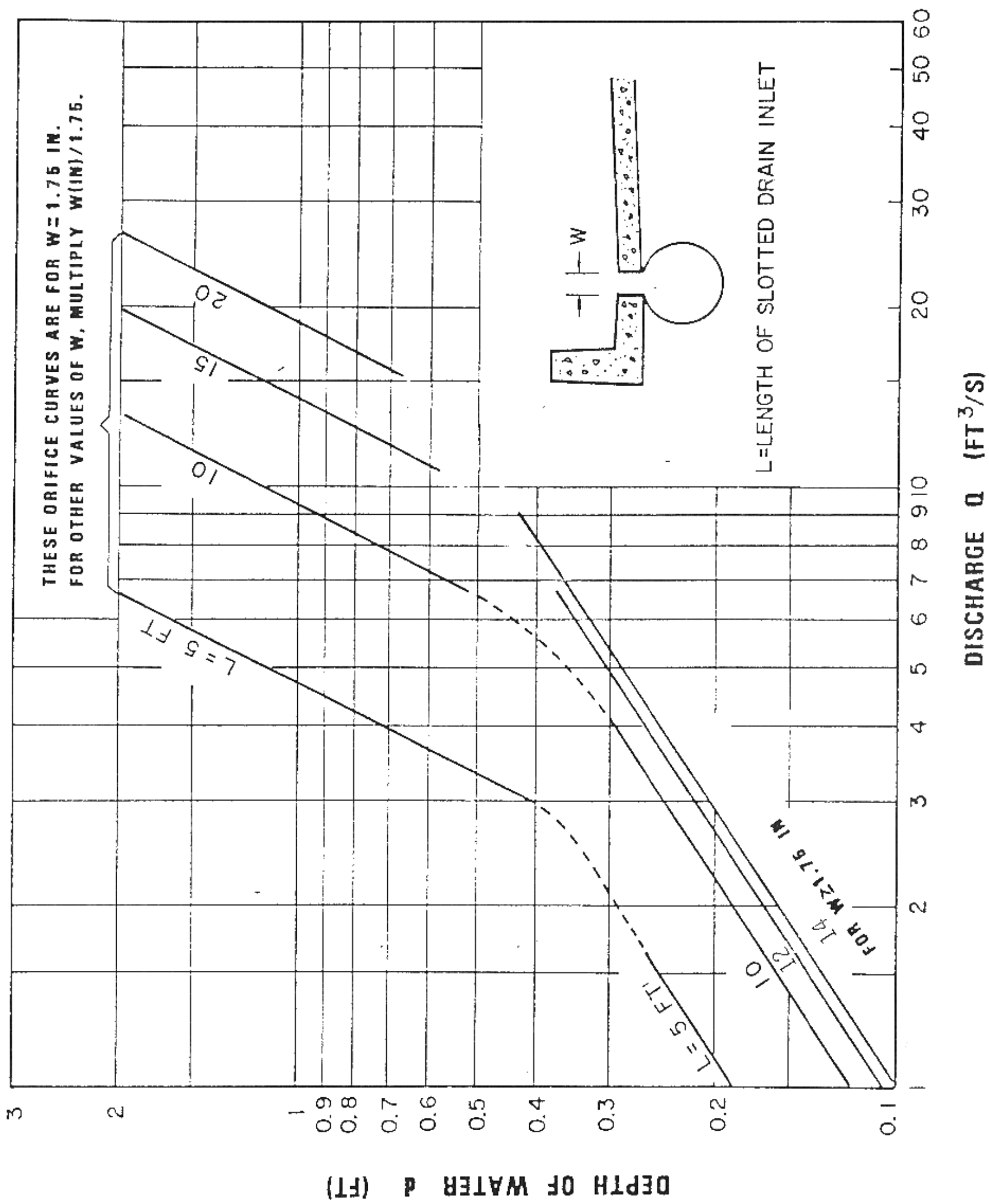


Figure 12-12 Slotted Drain Inlet Capacity In Sump Locations

Source: HEC-12

12.9 Inlet Interception Calculations (continued)

12.9.9 Inlet Spacing Computations

In order to design the location of the inlets for a given project, information such as a layout or plan sheet suitable for outlining drainage areas, road profiles, typical cross sections, grading cross sections, superelevation diagrams and contour maps are necessary. The inlet computation sheet, Table 12-4 (page 12-59), should be used to document the computations. A step-by-step procedure is as follows:

Step 1 Complete the blanks on top of the sheet to identify the job by route, location, date and your initials.

Step 2 Mark on the plan the location of inlets that are necessary even without considering any specific drainage area. See Section 12.11.3 Inlet Locations for additional information.
Step 3 Start at one end of the job, at one high point and work towards the low point, then space from the other high point back to the same low point.

Step 4 Select a trial drainage area approximately 300 to 500 ft below the high point and outline the area including any area that may come over the curb. (Use drainage area maps.) Where practical, large areas of behind the curb drainage should be intercepted before it reaches the highway. (See 12.7.5)

Step 5 Col 1 Describe the location of the proposed inlet by number and station in Col 1 & 2. Identify the curb and gutter type in the Remarks Column 19. A sketch of the cross section should be provided in the open area of the computation sheet.

Step 6 Col 3 Compute the drainage area in hectares and enter in Col 3.

Step 7 Col 4 Select a C value or compute a weighted value based on area and cover type as described in Section 12.6.2.1 and enter in Col 4.

Step 8 Col 5 Compute a time of concentration for the first inlet. This will be the travel time from the hydraulically most remote point in the drainage area to the inlet. See additional discussion in Section 12.6.2.3. The minimum time of concentration should be 10min. Enter value in Col 5.

Step 9 Col 6 Using the Intensity-Duration-Frequency curves, select a rainfall intensity at the t_c for the design frequency. Enter in Col. 6.

Step 10 Col 7 Calculate Q by multiplying Col 3 × Col 4 × Col 6. Enter in Col 7.

Step 11 Col 8 Determine the gutter slope at the inlet from the profile grade — check effect of superelevation. Enter in Col. 8.

Step 12 Col 9 Enter cross slope adjacent to inlet in Col 9 and gutter width in Col 13. Sketch composite cross slope with dimensions.

Step 13 Col 11 For the first inlet in a series (high point to low point) enter Col. 7 in Col. 11 since no previous flowby has occurred yet.

12.9 Inlet Interception Calculations (continued)

12.9.9 Inlet Spacing Computations (continued)

Step 14 Col 12 Using Figure 12-1 or the available computer model, determine the spread T and enter in Col 14 and calculate the depth d at the curb by multiplying T times the cross slope(s) and enter in Col 12. Compare with the allowable spread as determined by the design criteria in Section 12.9. If Col. 15 is less than the curb height and Col. 14 is near the allowable spread, continue on to step 16. If not OK, expand or decrease the drainage area to meet the criteria and repeat steps 5 through 14. Continue these repetitions until column 14 is near the allowable spread then proceed to step 15.

Step 15 Col 15 Calculate W/T and enter in Col 15.

Step 16 Col 16 Select the inlet type and dimensions and enter in Col 16.

Step 17 Col 17 Calculate the Q intercepted (Q_i) by the inlet and enter in Col 17. Utilize Fig. 12-1 and 12-2 or 12-3 to define the flow in the gutter. Utilize Fig. 12-2, 12-6 and 12-7 and equation 12.13 to calculate Q_i for a grate inlet and Fig. 12-9 and 12-10 to calculate Q_i for a curb-opening inlet. See Section 12.12.2 for a grate inlet example and Section 12.12.4 for a curb-opening inlet example.

Step 18 Col 18 Calculate the flowby by subtracting Col 17 from Col 11 and enter into Col 18 and also into Col. 10 on the next line if an additional inlet exists downstream.

Step 19 Col 1-4 Proceed to the next inlet down grade. Select an area approximately 270 to 300 feet below the first inlet as a first trial. Repeat steps 5 through 7 considering only the area between the inlets.

Step 20 Col 5 Compute a time of concentration for the second inlet downgrade based on the area between the two inlets.
Step 21 Col 6 Determine the intensity based on the time of concentration determined in step 19 and enter it in Col 6.

Step 22 Col 7 Determine the discharge from this area by multiplying Col 3 × Col 4 × Col 6. Enter the discharge in Col 7.

Step 23 Col 11 Determine total gutter flow by adding Col 7 and Col 10 and enter in Col 11. Column 10 is the same as Column 18 from the previous line.

Step 24 Col 12 Determine "T" based on total gutter flow (Col. 11) by using figure 12-1 or 12-3 and enter in Col. 14, (If "T" in Col. 14 exceeds the allowable spread, reduce the area and repeat steps 19-24. If "T" in Col. 14 is substantially less than the allowable spread, increase the area and repeat steps 19-24.)

Step 25 Col 16 Select inlet type and dimensions and enter in Col. 16.

Step 26 Col 17 Determine Q_i and enter in Col 17 — See instruction in step 17.

12.9 Inlet Interception Calculations (continued)

12.9.9 Inlet Spacing Computations (continued)

Step 27 Col 18 Calculate the flowby by subtracting Col 17 from Col 7 and enter in Col 16. This completes the spacing design for this inlet.

Step 28 Go back to step 19 and repeat step 19 through step 27 for each subsequent inlet. If the drainage area and weighted "C" values are similar between each inlet, the values from the previous grate location can be reused. If they are significantly different, recomputation will be required.

12.9 Inlet Interception Calculations (continued)

Storm Drainage Systems ADOT Drainage Manual INLET COMPUTATION SHEET							ROUTE_________ Location_______________________ DATE______________ COMPUTED BY________________________________ SHEET_____OF_____											
INLET		GUTTER DISCHARGE Design Frequency ______					GUTTER DISCHARGE Allowable Spread ______								INLET DISCHARGE			REMARKS
ID. NUMBER	STATION.	DRAIN .AREA "A" (Acres)	RUNOFF COEF "C"	TIME OF CONC. "T_C" (min)	Rain Intensity "I" (in/h)	Q= CIA (cfs)	GRADE, G (%)	CROSS SLOPE S_x (ft/ft)	PREV. RUNBY (cfs)	TOTAL GUTTER FLOW (cfs)	DEPTH "d" (ft)	GUTTER WIDTH "W" (ft)	SPREAD "T" (ft)	W/T	INLET TYPE	INTERCEPT "Q_i" (cfs)	RUNBY "Q_r" (cfs)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19

Table 12-4 Inlet Spacing Computation Sheet

A minimum velocity of 3 ft/s is desirable in the storm drain in order to prevent sedimentation from occurring in the pipe.

CHAPTER 13

STORM DRAINAGE SYSTEMS

Chapter 13 Storm Drain Systems
Table Of Contents

13.1 System Definition

13.1.1 Introduction

Storm drain systems are the collection of pipes and channels that serve to convey storm drain flows for their individual collection points to their discharge point. The storm drain system should be designed to operate without inhibiting the capture capacity of the inlets. For ordinary conditions, storm drains are sized on the assumption that they will flow full or practically full under the design discharge but will not flow under pressure head. Using the inlet locations determined in Chapter 12, the design of the system first involves the determination of the location and alignment of the pipes, the tailwater condition at the outfall, and then sizing the pipes to satisfy the prescribed hydraulic grade line (HGL) and energy grade line (EGL) criteria. The Manning's formula is used for capacity calculations.

13.1.2 Design Approach

The design of the storm drain system integrates the location of inlets, the connecting pipes and the outfall location including the tailwater elevation. The initial design steps begin with evaluating the location of the connecting pipes and the outfall in both plan and profile. A preliminary storm drain profile is laid out from the outfall to the inlets upstream. Consideration must be given to the invert elevations on the inlets, the most upstream inlet might not be the one that controls the slope of the trunk line. The next logical step is the computation of the rate of discharge to be carried by each reach of the storm drain, and the determination of the size and gradient of pipe required to convey this discharge. This is done by starting at the upstream reach, calculating the discharge and sizing the pipe, then proceeding downstream, reach by reach to the point where the storm drain connects with other drains or the outfall.

The design discharge at any point in the storm drain is not the simple addition of the inlet flows of all inlets above that section of storm drain unless the t_c is less than the minimum t_c. It is generally less than this total. The design rate of flow is the sum of the contributing drainage areas and the rainfall intensity for the time of concentration where the additional drainage area occurs. The time of concentration is most influential and as the time of concentration grows larger, the rainfall intensity to be used in the design grows smaller. In some cases, where a relatively large drainage area with a short time of concentration is added to the system, the peak flow may be larger for this subarea using the shorter time even though the entire drainage basin is not contributing. The prudent designer will be alert for unusual conditions and determine which time of concentration controls for each pipe segment. See Section 13.3.6 for a discussion on time of concentration.

At locations where the pipe size is increased (manholes or junction structures), the upstream pipe should be within the end area of the downstream pipe. It is usual to align the crown of pipes for situations with either open channel flow or only slight pressure flow. In summary the following items shall be considered during layout of the storm drain system:

1.) Line up crowns if possible, if not possible overlap the areas.
2.) Use wyes or tees rather than access structures to bring flows into the system. This will bring the connecting pipe at the spring line if the connecting pipe is 0.5D or less of the through pipe.
3.) When discharging into box culverts, bring the connecting pipe at least 18” above the invert.
4.) When possible position the outlet so it is pointed downstream.
5.) If need to do a 90 degree bend in a trunk line, consider 2-45 degree bends offset by 10 pipe diameters.

13.2 Symbols and Definitions

13.2.1 Symbols

To provide consistency within this chapter as well as throughout this manual the symbols in Table 13-1 will be used. These symbols were selected because of their wide use in storm drainage publications.

Table 13-1 Symbols And Definitions

Symbol	Definition	Units
A	Area of cross section	ft^2
A	Watershed area	acres
a	Depth of depression	inches
C	Runoff coefficient or coefficient	-
d	Depth of gutter flow at the curb line	ft
D	Diameter of pipe	ft
H	Head loss	ft
I	Rainfall intensity	in./hr
K	Coefficient	-
L	Length of curb opening inlet	ft.
L	Pipe length	ft.
L	Length of runoff travel	ft.
n	Roughness coefficient in Manning formula	-
Q_i	Intercepted flow	ft^3/sec
Q_T	Total flow	ft^3/sec
R_h	Hydraulic radius	ft
S or S_x	Pavement cross slope	ft/ft
S	Crown slope of pavement	ft/ft
S or S_L	Longitudinal slope of pavement	ft/ft
S_w	Depressed section slope	ft/ft
T	Top width of water surface (spread on pavement)	ft
t_c	Time of concentration	min
V	Velocity of flow	ft/sec
y	Depth of flow in approach gutter	ft

13.2 Symbols and Definitions (continued)

13.2.2 Definitions

Following are discussions of terms that will be used throughout the remainder of this chapter in dealing with different aspects of storm drainage analysis.

Bypass/Flowby (Carry over) -- Occurs at an inlet on grade. It is the flow that is not captured at an inlet on grade, bypasses, and is carried to the next inlet downgrade. Inlets on grade are usually designed to allow a certain amount of flowby, unless located upstream of an area where pedestrians are expected to use the street.

Crown-- The crown, sometimes known as soffit, is the top inside of a pipe.

Culvert-- A culvert is a closed conduit whose purpose is to convey surface water under a roadway, railroad or other impediment. It may have inlets connected to it.

Flow-- Flow refers to a quantity of water that is flowing.

Hydraulic Grade Line-- The hydraulic grade line is the locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe run (pressure head plus elevation head).

Invert-- The invert is the inside bottom of the pipe.

Lateral Line-- A lateral line, sometimes referred to as a connector, has inlets connected to it but has no other storm drains connected. It is usually tributary to the trunk line.

Pressure Head-- Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.

Storm Drain-- A storm drain is a closed or open conduit that conveys storm water that has been collected by inlets to an outfall. It generally consists of laterals, connectors, and trunk lines or mains. Culverts connected to the storm drainage system are considered part of the system.

Trunk Line-- A trunk line is the main storm drain line. Lateral lines may be connected in at inlet structures or access holes. A trunk line is sometimes referred to as a "main."

Velocity Head-- Velocity head is a quantity proportional to the kinetic energy of flowing water expressed as a height or head of water, ($V^2/2g$).

13.3 Design Concepts

13.3.1 Hydraulic Capacity

The most widely used formula for determining the hydraulic capacity of storm drains for gravity and pressure flows is the Manning's formula and it is expressed by the following equation:

$$\mathbf{V=\frac{1.486R^{0.67}S^{0.5}}{n}} \qquad \textbf{(13.1)}$$

Where: V = mean velocity of flow, ft/sec
n = Manning's roughness coefficient
R = hydraulic radius, ft = area of flow divided by the wetted perimeter (A/WP)
S = the slope of the energy grade line, ft/ft

In terms of discharge, the above formula becomes:

$$Q= V*A = \mathbf{\frac{1.486*A*R^{0.67}S^{0.5}}{n}} \qquad \textbf{(13.2)}$$

Where: Q = rate of flow, ft^3/sec
A = cross sectional area of flow, ft^2

For storm drains flowing full, the above equations become:

$$V= \frac{0.59D^{0.67}S^{0.5}}{n} \qquad \textbf{(13.3)}$$

$$\mathbf{Q= \frac{0.46\ D^{2.67}\ S^{0.5}}{n}} \qquad \textbf{(13.4)}$$

Where: D = diameter of pipe, ft

The nomograph solution of Manning's formula for full flow in circular storm drains is shown on Figure 13-1 and Figure 13-2. Figure 13-3 is provided to assist in the solution of the Manning's equation for part full flow in storm drains.

The slope required for full flow can be determined by rearranging equation 13-4 as

$$S= [Qn/(0.46D^{2.67})]^2 \qquad \textbf{(13.5)}$$

Manning's n for commonly used pipe

Concrete, precast	0.012
Concrete, site cast	0.014
HDPE, smooth interior	0.012

13.3 Design Concepts (continued)

13.3.1 Hydraulic Capacity (continued)

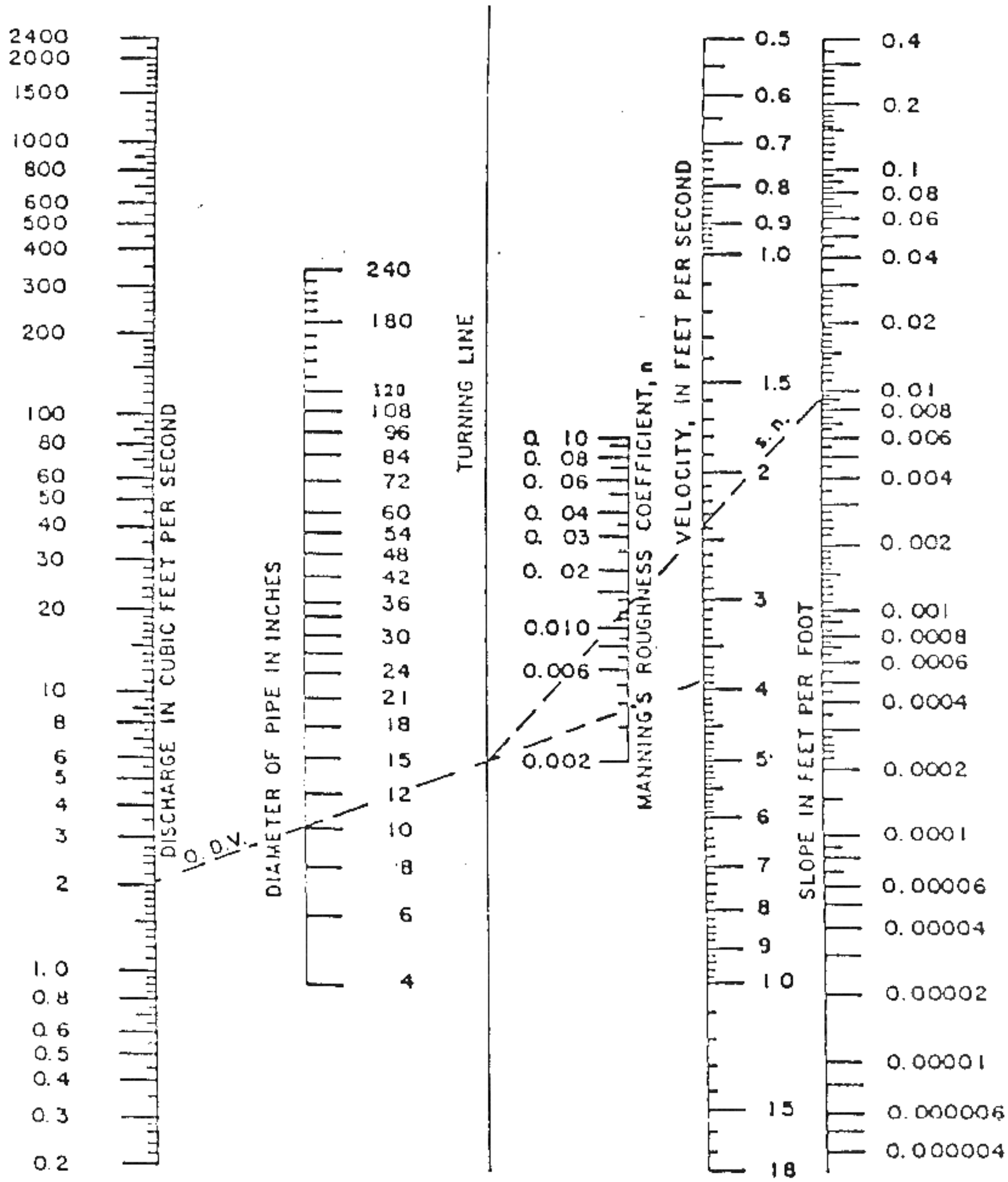


Figure 13-1 Manning's Formula For Flow In Storm Drain

13.3 Design Concepts (continued)

13.3.1 Hydraulic Capacity (continued)

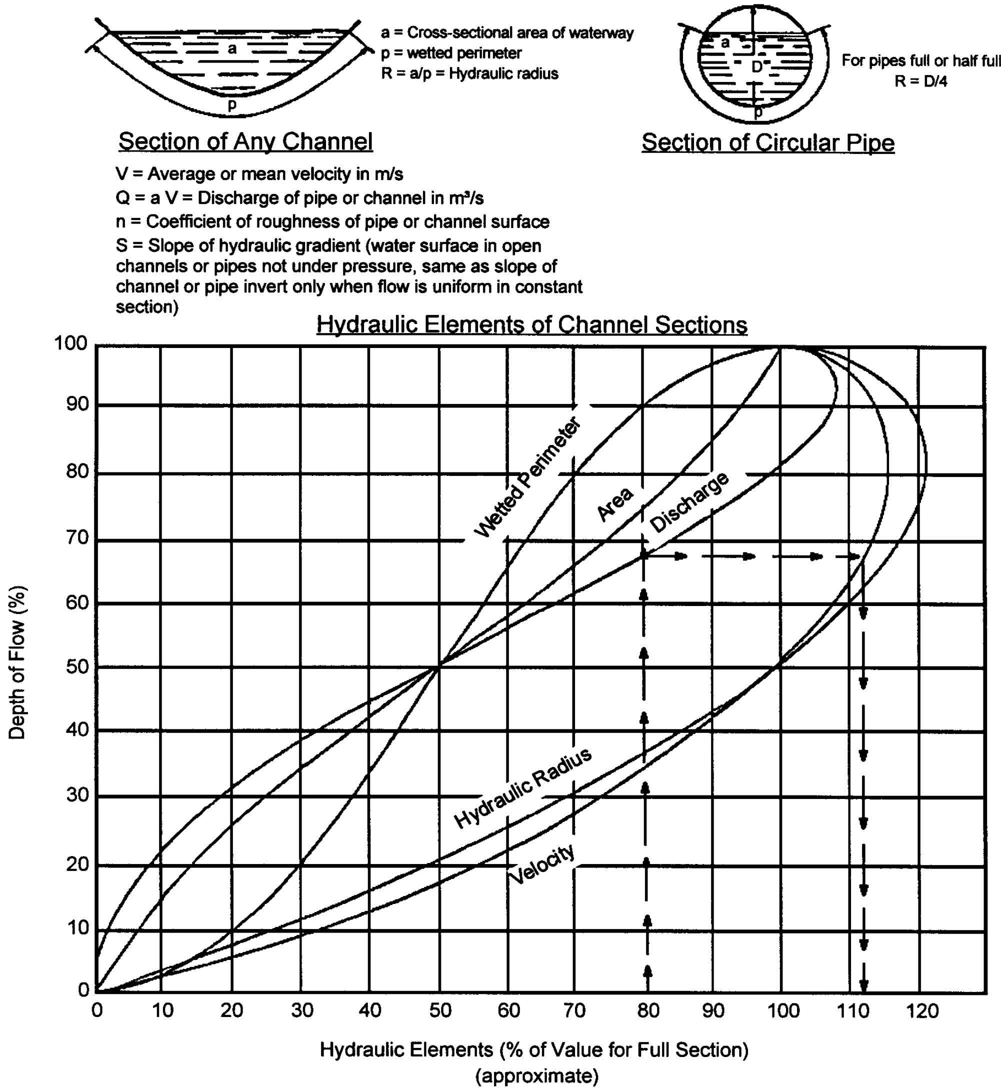


Figure 13-2 Values Of Hydraulic Elements Of Circular Section

For Various Depths Of Flow

13.3 Design Concepts (continued)

13.3.2 Minimum Grade

For very flat grades the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. The storm drainage system should be checked to be sure there is sufficient velocity in all of the drains to deter settling of particles. It is desirable that all storm drains be designed such that velocities of flow will not be less than 3 ft/sec at design flow. Minimum slopes required for a velocity of 3 ft/sec can be calculated by the Manning's formula or use values given in Table 13-1.

$$S = [Vn/0.59D^{0.67}]^2 \qquad \textbf{(13.6)}$$

For concrete pipe, n=0.012 and a velocity of 3 ft/sec, this may be rewritten as

$$S = (0.003723)/D^{1.33} \qquad \textbf{(13.7)}$$

Table 13-1

Minimum Slopes Necessary To Ensure v=3 ft/sec

In Storm Drains Flowing Full

Pipe Size, in.	Q, Full pipe ft^3/sec	Minimum Slopes ft/ft n = 0.012	n = 0.014	n = 0.024
18	5.30	0.0022	0.0030	0.0087
21	7.22	0.0018	0.0024	0.0071
24	9.43	0.0015	0.0020	0.0059
27	11.93	0.0013	0.0017	0.0051
30	14.73	0.0011	0.0015	0.0044
33	17.82	0.00097	0.0013	0.0039
36	21.21	0.00086	0.0012	0.0034
42	28.86	0.00070	0.00095	0.0028
48	37.70	0.00059	0.00080	0.0023
54	47.71	0.00050	0.00068	0.0020
60	58.90	0.00044	0.00059	0.0017
66	71.27	0.00038	0.00052	0.0015
72	84.82	0.00034	0.00046	0.0014

13.3 Design Concepts (continued)

13.3.3 Access Structures

13.3.3.1 Location

Access structures are utilized to provide entry to continuous underground storm drains for inspection and cleanout. In some locations grate inlets are used in lieu of access structures, when entry to the system can be provided at the inlet, so that the benefit of extra stormwater interception can be achieved with minimal additional cost. Typical locations where access structures should be specified are:

- where more than two storm drains converge,
- at intermediate points along tangent sections,
- where pipe size changes,
- where an abrupt change in horizontal or vertical alignment occurs
- where an abrupt change in elevation occurs.

Access structures should not be located in traffic lanes; however, when it is impossible to avoid locating an access structure in a traffic lane, care should be taken to insure it is not in the normal vehicle wheel path.

13.3.3.2 Spacing

The spacing of access structures should be in accordance with the following criteria:

Table 13-2

Pipe Size, inches	Maximum Spacing, ft.
Under 33”	330’
33 in. to 39 in.	440’
42 in. to 69 in.	660’
72 in. or greater	1200’

13.3.3.3 Sizing

ADOT C-Standards present details for normal manhole applications. When determining the minimum base required for various pipe sizes and locations, two general criteria must be met.

- The base structure must be large enough to accept the maximum pipe.
  A minimum width of D+3.5 feet is required.

- Access structure must be large enough to provide a minimum space between pipes. If pipes are located at substantially different elevations, pipes may not conflict and the above analysis is unnecessary. A minimum spacing of 12 inches shall be provided between the exterior walls of adjacent pipes.

13.3 Design Concepts (continued)

13.3.4 Curved Alignment

Curved storm drains are permitted where necessary. Long radius bend sections are available from many suppliers and are the preferable means of changing direction in pipes 48 inches and larger. Short radius bend sections are also available and can be utilized if there isn't room for the long radius bends. Deflecting the joints to obtain the necessary curvature is not desirable except in very minor curvatures. A deflection of 1.5 degrees is often possible at t joint, the designer should check with the pipe manufacturer for specifics regarding the proposed pipe sizes. Utilizing large access holes solely for changing direction may not be cost effective on large size storm drains.

13.3.5 Inverted Siphons

An inverted siphon carries the flow under an obstruction such as sanitary sewers, water mains, or any other structure or utility lane that is in the path of the storm drain line. The storm drain invert is lowered at the obstacle and is raised again after the crossing. The criteria for designing inverted siphons can be found in most of the hydraulics textbooks.

13.3.6 Time of Concentration

Pipe Sizing

The time of concentration for pipe sizing is defined as the time required for water to travel from the most hydraulically distant point of the watershed to the point of the storm drain system under consideration. It generally consists of two components: (1) the time to flow to the inlet which can consist of overland and channel or gutter flow and (2) the time to flow through the storm drain to the point under consideration.

Travel time within the storm drain pipes can be estimated by the relation:

$$t_t = L / 60V \qquad \textbf{(13.9)}$$

Where: t_t = travel time, min
L = length of pipe in which runoff must travel, ft
V = estimated or calculated normal velocity, ft/sec

To summarize, the time of concentration for any point on a storm drain is the inlet time for the inlet at the upper end of the line plus the time of flow through the storm drain from the upper end of the storm drain to the point in question. In general, where there is more than one source of runoff to a given point in the storm drainage system, the longest t_c is used to estimate the intensity (I).

13.3 Design Concepts (continued)

Pipe Sizing (continued)

There could be exceptions to this generality, for example where there is a large inflow area at some point along the system, the t_c for that area may produce a larger discharge than the t_c for the summed area with the longer t_c. The designer should be cognizant of this possibility when joining drainage areas and determine which drainage area governs. To determine which drainage area controls, compute the peak discharge for each t_c. Note that when computing the peak discharge with the shorter t_c, not all the area from the basin with the longest t_c will contribute runoff. One way to compute the contributing area, A_c, is as follows:

$$A_c = A\,[t_{c1} / t_{c2}] \qquad \textbf{(13.10)}$$

Where: $t_{c1} < t_{c2}$ and A is the area of the basin with the longest t_c.

13.4 Energy Losses

13.4.1 System Performance

The performance of a storm drain system is evaluated by determining the location of the hydraulic grade line during the design storm event. The hydraulic grade line is the location of the piezometric surface along the storm drain system. Usually it is helpful to **compute the EGL first, then subtract the velocity head (V^2/2g) to obtain the HGL.** Water flow is driven by the difference in energy head from one location to another. As water flows it encounters many obstructions that cause a loss of energy. The computation of the EGL requires the evaluation of the energy losses in the system.

13.4.2 Tailwater

For most design applications, **the tailwater will either be above the crown of the outlet** or can be considered to be between the crown and critical depth. To determine the EGL, begin with the tailwater elevation or (d_c + D)/2, whichever is higher, **add the velocity head for full flow** and proceed upstream to compute all losses such as exit losses, friction losses, junction losses, bend losses and entrance losses as appropriate.

An exception to the above might be a very large outfall with low tailwater when a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the tailwater, whichever is higher.

When estimating tailwater depth on the receiving stream, the prudent designer will consider the joint or coincidental probability of two events occurring at the same time. For the case of a tributary stream or a storm drain, its relative independence may be qualitatively evaluated by a comparison of its drainage area with that of the receiving stream. A short duration storm that causes peak discharges on a small basin may not be critical for a larger basin. Also, it may safely be assumed that if the same storm causes peak discharges on both basins, the peaks will be out of phase. To aid in the evaluation of joint probabilities, refer to the Table below.

Table 13-3

AREA RATIO	**FREQUENCIES FOR COINCIDENTAL OCCURRENCE**			
	10-Year Design		100- Year Design	
	Mainstream	Tributary	Mainstream	Tributary
10,000 to 1	1 10	10 1	2 100	100 2
1,000 to 1	2 10	10 2	10 100	100 10
100 to 1	5 10	10 5	25 100	100 25
10 to 1	10 10	10 10	50 100	100 50
1 to 1	10 10	10 10	100 100	100 100

Reference: USCE, Norfolk District, 1974

13.4 Energy Losses (continued)

13.4.3 Exit Losses

The exit loss is a function of the change in velocity at the outlet of the pipe. For a sudden expansion such as an endwall, the exit loss is:

$$H_o=1.0[V^2/2g - V_d^2/2g] \qquad \textbf{(13.11)}$$

Where: V = average outlet velocity, ft/sec
V_d = channel velocity downstream of outlet, ft/sec

Note that when V_d = 0 as in a reservoir, the exit loss is one velocity head. For part full flow where the pipe outlets in a channel with moving water, the exit loss may be reduced to virtually zero.

13.4.4 Bend Losses

For bends outside of structures, the bend loss coefficient for storm drain or large radii is minor but can be evaluated using the formula:

$$h_b = 0.0033\ (\Delta)\ (V_o^2 / 2g) \qquad \textbf{(13.12)}$$

Where: Δ = angle of curvature in degrees, less than or equal to 90 degrees.

13.4.5 Pipe Friction Losses

The friction slope is the energy gradient in ft/ft for that run. The friction loss is simply the energy gradient multiplied by the length of the run in feet. Energy losses from pipe friction may be determined by rewriting the Manning's equation with terms as previously defined:

$$S_f = [Qn/1.486\ AR^{2/3}]^2 = [Qn/0.46\ D^{2/3}]^2 \qquad \textbf{(13.13)}$$

$$\text{or}\ \ S_f = [Vn/0.59D^{0.67}]^2 \qquad \textbf{(13.14)}$$

The head losses due to friction may be determined by the formula:

$$H_f=L*S_f \qquad \textbf{(13.15)}$$

The Manning's equation can also be written to determine friction losses for storm drains as follows:

$$H_f= \frac{V^2\ n^2\ L}{2.21gR^{4/3}} \qquad \textbf{(13.16)}$$

$$H_f= \frac{V^2\ n^2\ L}{0.35\ D^{4/3}} \qquad \textbf{(13.17)}$$

13.4 Energy Losses (continued)

13.4.5 Pipe Friction Losses (continued)

Where: H_F = total head loss due to friction, ft
- n = Manning's roughness coefficient
- D = diameter of pipe, ft
- L = length of pipe, ft
- V = mean velocity, ft/sec
- R = hydraulic radius, ft
- g = 32.2 ft/sec^2
- S_f = slope of hydraulic grade line, ft/ft

13.4.6 Expansion and Contraction losses

Pipe size transitions are sometimes made without the use of a junction structure. The energy loss is taken as $h = k^*\Delta V^2/2g$

Values of K are in the tables in Appendix E.

Open Channel Flow

Expansion

$$h = \frac{k_e(V_1^2 - V_2^2)}{2g}$$

Contraction

$$h = \frac{k_c^*(V_2^2 - V_1^2)}{2g}$$

Pressure Flow

$$h = \frac{k^*(V_2^2)}{2g}$$

13.4.7 Structure Losses

There are two types of structures to consider for structure losses. Pipes may be connected using manufactured wyes or tees or the may be connected at access structures such as manholes. These two types of structures have different losses and different methods for estimating those losses.

Manufactured Wye or Tee Losses.
The loss at a manufactured wye or tee is

$$H_j = \frac{(Q_oV_o) - (Q_iV_i) - Q_lV_l\cos(\square)}{(0.5(A_o + A_i))\ g} + \frac{V_i^2 - V_o^2}{2g} \qquad \textbf{(13.18)}$$

Where:
- Q_o = outlet discharge, cfs
- V_o= outlet velocity, ft/sec
- A_o= outlet cross-sectional area, sq. ft.

13.4 Energy Losses (continued)

13.4.7 Structure Losses (continued)

Q_i= inlet discharge, cfs
V_i= inlet velocity, ft/sec
A_i= inlet cross-sectional area, sq.ft.

Q_l= connector/lateral pipe discharge, cfs
V_l= connector/lateral pipe velocity, ft/sec.
□= angle between the inflow and outflow pipes.

Access Structure Losses

The head loss encountered in going from one pipe to another through an access hole is commonly represented as being proportional to the velocity head at the outlet pipe. Using K to signify this constant of proportionality, the energy loss is approximated as $H_j = K \times (V_o^2/2g)$. Experimental studies have determined that the K value can be approximated as follows:

$$K = K_o C_D C_d C_Q C_p C_B \qquad \textbf{(13.19)}$$

Where: K = adjusted loss coefficient
K_o = initial head loss coefficient based on relative access hole size.
C_D = correction factor for pipe diameter (pressure flow only)
C_d = correction factor for flow depth (non-pressure flow only)
C_Q = correction factor for relative flow
C_B = correction factor for benching
C_p = correction factor for plunging flow

Relative Structure Hole Size and Angle of Deflection

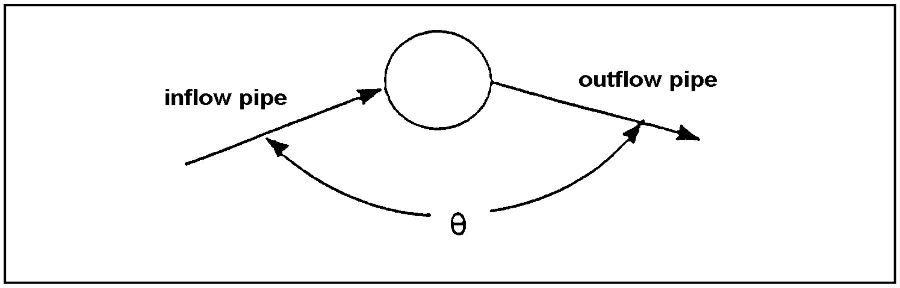
K_o is estimated as a function of the relative access hole size and the angle of deflection between the inflow and outflow pipes.

$$K_o = 0.1(b/D_o)(1-\sin(□)) + 1.4(b/D_o)^{0.15}\sin(□) \qquad \textbf{(13.20)}$$

Where: □ = the angle between the inflow pipe under consideration and the outflow pipe.
b = access hole diameter, ft
D_o = outlet pipe diameter, ft

13.4 Energy Losses (continued)

13.4.7 Structure Losses (continued)



Deflection Angle

If the structure is a rectangular box, it should be treated as an equivalent circular structure.

Pipe Diameter

A change in head loss due to differences in pipe diameter is **only significant in pressure flow situations when the depth in the access hole to outlet pipe diameter ratio, d/D_o, is greater than 3.2**. Therefore, it is only applied in such cases.

$$C_D = (D_o / D_i)^3 \quad \textbf{(13.21)}$$

Where: D_i = incoming pipe diameter, ft
D_o = outgoing pipe diameter, ft

Flow Depth

The correction factor for flow depth is significant only in cases of **free surface flow or low pressures, when d/D_o ratio is less than 3.2 and is only applied in such cases.** Water depth in the access hole is approximated as the level of the hydraulic gradeline at the upstream end of the outlet pipe.

The correction factor for flow depth, C_d, is calculated by the following:

$$C_d = 0.5(d/D_o)^{0.6} \quad \textbf{(13.22)}$$

Where: d = water depth in access hole above invert of outlet pipe, ft
D_o= outlet pipe diameter, ft

13.4 Energy Losses (continued)

13.4.7 Structure Losses (continued)

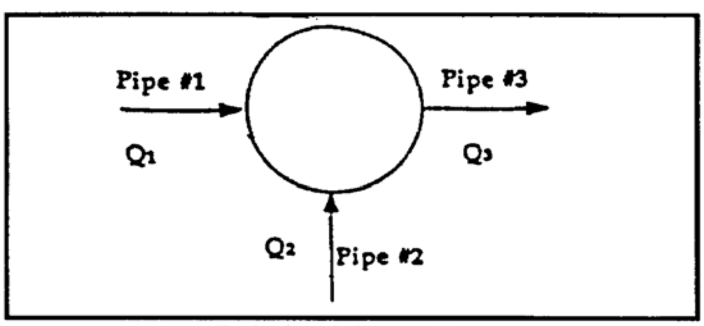
Relative Flow

The correction factor for relative flow, C_Q, is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. It is computed as follows:

$$C_Q = (1 - 2\sin(\square))(1 - Q_i / Q_o)^{0.75} + 1 \qquad \textbf{(13.23)}$$

where: C_Q = correction factor for relative flow
$\square$ = the angle between the inflow and outflow pipes
Q_i = flow in the inflow pipe under consideration, ft^3/sec
Q_o = flow in the outlet pipe, ft^3/sec

As can be seen from the equation, C_Q is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. To illustrate this effect, consider the access structure shown in the Figure and assume the following two cases to determine the impact flows of pipe 2 entering the access hole.



Relative Flow Effect

Case 1:
$Q_1 = 3.3\ ft^3/sec$, $Q_2 = 1.1\ ft^3/sec$, $Q_3 = 4.4\ ft^3/sec$

$C_{3\text{-}1} = (1-2*\sin(180°))(1 - 3.3/4.4)^{0.75} + 1$
$C_{3\text{-}1} = (1-0)(1-0.75)^{0.75} + 1 = 1.35$

$C_{2\text{-}1} = (1-2*\sin(90°))(1 - 1.1/4.4)^{0.75} + 1$
$C_{2\text{-}1} = (1-2)(1-0.25)^{0.75} + 1 = 0.19$

Case 2:
$Q_1 = 1.1\ ft^3/sec$, $Q_2 = 3.3\ ft^3/sec$, $Q_3 = 4.4\ ft^3/s$

$C_{3\text{-}1} = (1-2*\sin(180°))(1 - 1.1/4.4)^{0.75} + 1$
$C_{3\text{-}1} = (1-0)(1-0.25)^{0.75} + 1 = 1.81$

$C_{2\text{-}1} = (1-2*\sin(90°))(1 - 3.3/4.4)^{0.75} + 1$
$C_{2\text{-}1} = (1-2)(1-0.75)^{0.75} + 1 = 0.65$

13.4 Energy Losses (continued)

13.4.7 Structure Losses (continued)

Plunging Flow

This correction factor corresponds to the effect of another inflow pipe or surface flow from an inlet, plunging into the access hole, on the inflow pipe for which the head loss is being calculated. Using the notations in the figure below for the example, C_p is calculated for pipe # 1 when pipe # 2 discharges plunging flow. **The correction factor is only applied when h > d.**

The correction factor for plunging flow, C_p, is calculated by the following:

$$C_p = 1+0.2[h/ D_o][(h- d)/ D_o] \quad \textbf{(13.24)}$$

Where: C_p = correction for plunging flow
h = vertical distance of plunging flow from flow line of incoming pipe to the center of outlet pipe, ft
D_o = outlet pipe diameter, ft
d = water depth in access hole, ft

Benching

Normally, the bottom of an access structure constructed in accordance with C-18.10 can be considered to be full-round. The correction for benching in the access hole, C_B, is obtained from Table 13-8. Benching tends to direct flows through the access hole, resulting in reductions in head loss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed.

Losses at inlets

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 is estimated by using the inlet control coefficients presented in the Culvert Chapter. For inlets operating under outlet control, entrance losses can be calculated using

$$h_i = k_e (V^2/2g) \quad \textbf{(13.25)}$$

Where: h_i = headloss at inlet, ft.
k_e = entrance loss coefficient. Values are listed in the Culvert Chapter.

Summary

In summary, to estimate the head loss through an access structure from the outflow pipe to a particular inflow pipe, multiply the above correction factors together to get the head loss coefficient, K. This coefficient is then multiplied by the velocity head in the **outflow pipe** to estimate the minor loss for the connection.

13.4 Energy Losses (continued)

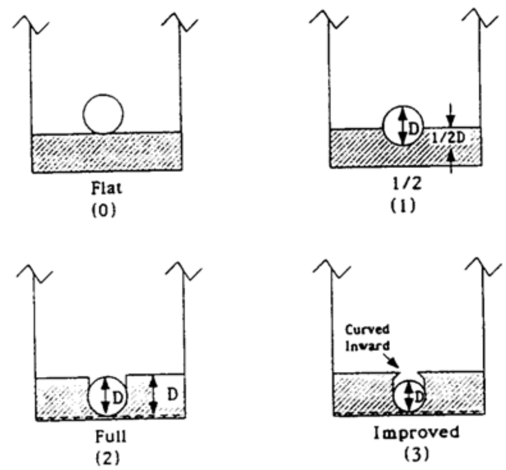
13.4.7 Structure Losses (continued)

Schematic Representation Of Benching Types

Table 13-4

Bench Type	Correction Factors, C_B	
	Submerged*	Unsubmerged**
Flat floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07
Improved	0.40	0.02

*pressure flow, $d/D_o > 3.2$
**free surface flow, $d/D_o < 1.0$



13.5 Design Procedure

The design of storm drainage systems is generally divided into the following operations:

- Storm drain design computation should be documented on forms. A sample is as illustrated in Figure 13-4.

Step 1 Determine inlet location and spacing as outlined in Chapter 12.

Step 2 Prepare plan layout of the storm drainage system establishing the following design data:
a. Location of storm drains.
b. Direction of flow.
c. Location of access holes.
d. Location of existing utilities such as water, gas, or underground cables.

Step 3 Set a preliminary slope between the outfall and the upstream end of the system. On a profile, plot the outfall elevations, and the invert elevation of any inlet that would control the invert elevations of the storm drain.

Step 4 From the inlet design get the drainage areas and runoff coefficient. For the first inlet in the system get the time of concentration to the first inlet. Using an Intensity-Duration-Frequency (IDF) curve, determine the rainfall intensity. Calculate the discharge by multiplying $C \times I \times A$.

Step 5 Size the pipe to convey the discharge assuming full flow by varying the slope and pipe size as necessary.

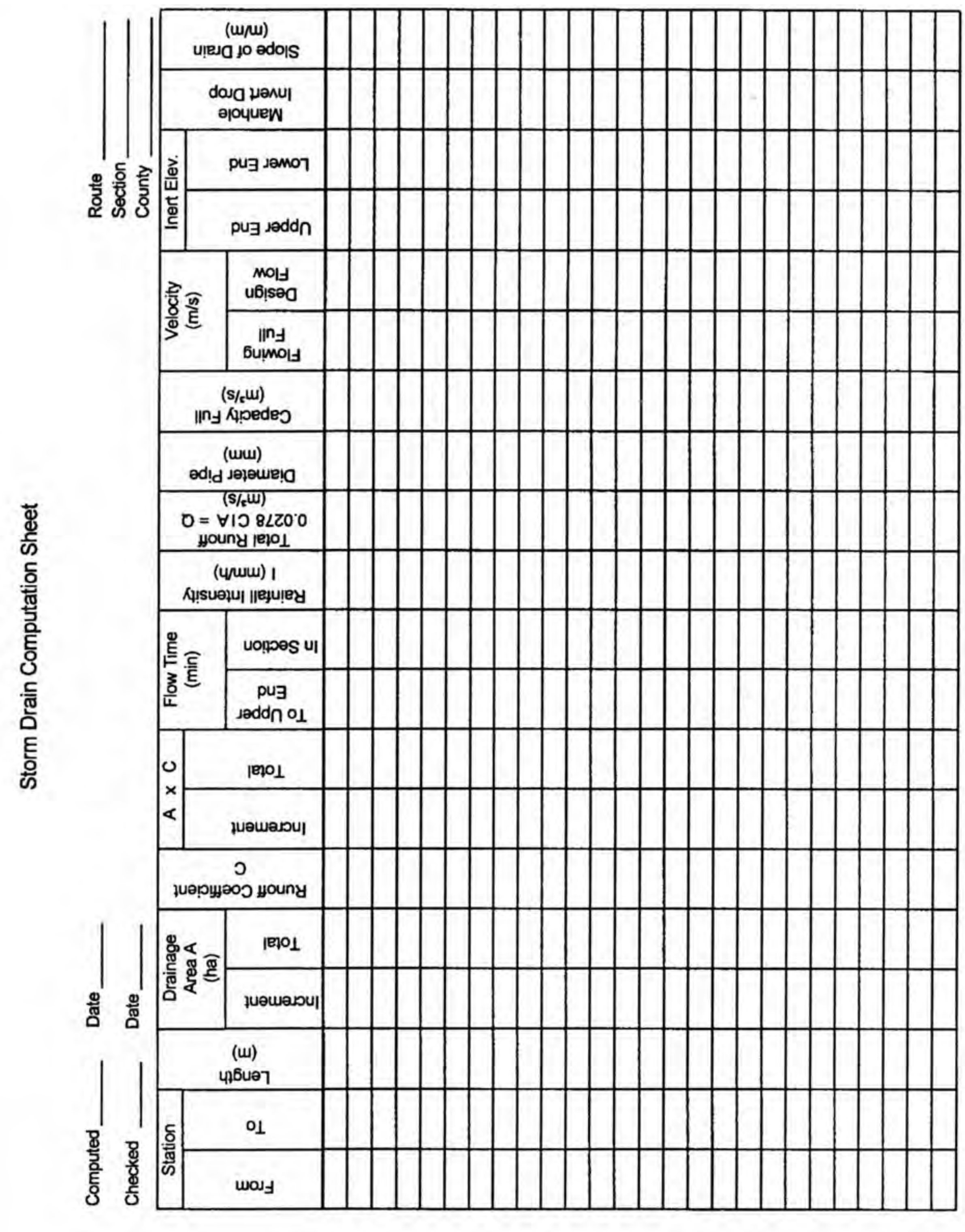
Step 6 Calculate travel time in the pipe to the next inlet or access hole by dividing pipe length by the velocity. This travel time is added to the time of concentration for a new time of concentration and a new rainfall intensity at the next entry point.

Step 7 At the next inflow point, calculate the additional CA, the sub area (A) multiplied by the subarea runoff coefficient (C), then add to the previous CA. Multiply the total CA by the rainfall intensity at the computed time of concentration to determine the new discharge. If the local CA is large, compare this discharge with the CA for the subbasin and the rainfall intensity using the inlet time of concentration. Use the larger of the two discharges. Determine the size of pipe and slope necessary to convey the discharge.

Step 8 Continue this process to the storm drain outlet. Utilize the equations and/or nomographs to accomplish the design effort.

Step 9 Complete the design by calculating the hydraulic grade line as described in Section 13.6.

13.5 Design Procedure (continued)



Storm Drain Computation Sheet

Computed ______ Date ______

Checked ______ Date ______

Route ______

Section ______

County ______

Station		Length (m)	Drainage Area A (ha)		Runoff Coefficient C	A x C		Flow Time (min)		Rainfall Intensity I (mm/h)	Total Runoff 0.0278 C I A = Q (m^3/s)	Diameter Pipe (mm)	Capacity Full (m^3/s)	Velocity (m/s)		Inert Elev.		Manhole Invert Drop	Slope of Drain (m/m)
From	To		Increment	Total		Increment	Total	To Upper End	In Section					Flowing Full	Design Flow	Upper End	Lower End		

Figure 13-4 Storm Drain Computation Sheet

13.6 Hydraulic Grade Line

13.6.1 Introduction

The hydraulic grade line (HGL) is the last feature to be established relating to the hydraulic design of storm drains. The hydraulic grade line guides the designer in determining the acceptability of the proposed system by establishing the elevations along the system to which the water will rise when the system is operating at a design frequency flow discharge. A special concern with storm drains designed to operate under pressure flow conditions is that inlet surcharging and possible access structure lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on open channel conditions must be carefully planned as well, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system. As hydraulic calculations are performed, frequent verification of the existence of the desired flow condition should be made. Storm drain systems can often alternate between pressure and open channel flow conditions from one section to another.

The calculation of the hydraulic grade line is usually for subcritical flow. Therefore, the computation begins at the system outfall with the tailwater elevation and a known HGL. If the outfall is an existing storm drain system, the EGL calculation must begin at the outlet end of the existing system and proceed upstream through this in-place system, then upstream through the proposed system. To the known energy grade is added the pipe friction losses, resulting in the energy grade line at the upstream end of the pipe. Adding the energy losses in the junction gives the energy grade line for the downstream end of the upstream pipes.

C**ompute the EGL first, then subtract the velocity head (V^2/2g) to obtain the HGL.** See Figure 9-4 for a sketch of a culvert outlet that depicts the difference between the HGL and the energy grade line (EGL). In general, if the HGL is above the crown of the pipe, pressure flow hydraulic calculations are appropriate. Conversely, if the HGL is below the crown of the pipe, open channel flow calculations are appropriate. Open channel flow should be assumed only when the flow depth is less than 80% of the pipe diameter. The design should normally be sized assuming full flow and normally analyzed for pressure flow. Flow events greater than the design event will result in pressure flow. The process is repeated throughout the storm drain system. At each junction the HGL should be checked versus the inlet or ground elevation. If all HGL elevations are acceptable then the hydraulic design is adequate. If the HGL exceeds the desirable elevation, then adjustments to the design must be made to lower the water surface elevation. See Figure 13-18 for a sketch depicting the results of and inadequate designs of a storm drain system.

13.6.2 HGL Calculation Procedure

A step-by-step procedure necessary to manually calculate the location of the hydraulic gradeline, HGL, is presented below. The procedure can be documented by the use of Table 13-5.

Step 1 Enter in Col. 1 the station for the junction immediately upstream of the outflow pipe. EGL computations begin at the outfall and are worked upstream taking each junction into consideration.

Step 2 Enter in Col. 2 the tailwater elevation, refer to Section 13.4.2 for procedure.

Step 3 Enter in Col. 3 the diameter (D_o) of the outflow pipe.

13.6 Hydraulic Grade Line

13.6.2 HGL Calculation Procedure (continued)

Step 4 Enter in Col. 4 the design discharge (Q_o) for the outflow pipe.

Step 5 Enter in Col. 5 the length, L_o, of the outflow pipe.

Step 6 Enter in Col. 6 the outlet velocity of flow, V_o.

Step 7 Enter in Col. 7 the velocity head, $V_{o2}/2g$.

Step 8 Enter in Col. 8 the exit loss, H_o.

Step 9 Enter in Col. 9 the friction slope (S_{fo}) in ft/ft of the outflow pipe. This can be determined by using the equation 13.13. **Note: Assumes full flow conditions.**

Step 10 Enter in Col. 10 the friction loss (H_f) that is computed by multiplying the length (L_o) in Col. 5 by the friction slope (S_{fo}) in Col 9. On curved alignments, calculate curve losses by using the formula H_c = 0.0033 (Δ)($V_o^2/2g$), where Δ = angle of curvature in degrees, and add to the friction loss.

Step 11 If the connection of pipes is made with a wye or tee, enter n/s in columns 11 through 17 and enter H_j in column 18 as computed by equation 13.18, then go to step 19 or if the connection is made with an access structure, enter in Col. 11 the initial head loss coefficient, K_o, based on relative access hole size as computed by equation 13.20.

Step 12 Enter in Col. 12 the correction factor for pipe diameter, C_D, as computed by equation 13.21.

Step 13 Enter in Col. 13 the correction factor for flow depth, C_d, as computed by equation 13.22. Note this factor is only significant in cases where the d/D_o ratio is less than 3.2.

Step 14 Enter in Col. 14 the correction factor for relative flow, C_Q, as computed by equation 13.23.

Step 15 Enter in Col. 15 the correction factor for plunging flow, C_p, as computed by equation 13.24. The correction factor is only applied when $h>d$.

Step 16 Enter in Col. 16 the correction factor for benching, C_B, as determined in Table 13-4.

Step 17 Enter in Col. 17 the value of K as computed by equation 13.19.

Step 18 Enter in Col. 18 the value of the total access hole loss, $K\ V_o^2/2g$.

Step 19 If the tailwater submerges the outlet end of the pipe, enter in Col. 19 the sum of Col. 2 (TW elevation) and Col. 7 (exit loss) to get the EGL at the outlet end of the pipe. If the pipe is flowing full, but the tailwater is low, the EGL will be determined by adding the velocity head to (d_c + D)/2.

13.6 Hydraulic Grade Line

13.6.2 HGL Calculation Procedure (continued)

Step 20 Enter in Col. 20 the sum of the friction head (Col 10), the access hole losses (Col 18), and the energy grade line (Col 19) at the outlet to obtain the EGL at the inlet end. This value becomes the EGL for the downstream end of the upstream pipe.

Step 21 Determine the HGL (Col 21) throughout the system by subtracting the velocity head (Col 7) from the EGL (Col 20).

Step 22 Check to make certain that the HGL is below the level of allowable high water at that point. If the HGL is above the finished grade elevation, water will exit the system at this point for the design flow.

Connector pipe profiles will usually have an entrance loss equal to a pipe culvert, $(1+k_e)*(V_1^2/2g.)$

The above procedure applies to pipes that are flowing full, as should be the condition for design of new systems. If a part full flow condition exists, the EGL is located one velocity head above the water surface.

13.6 Hydraulic Grade Line

13.6.2 HGL Calculation Procedure (continued)

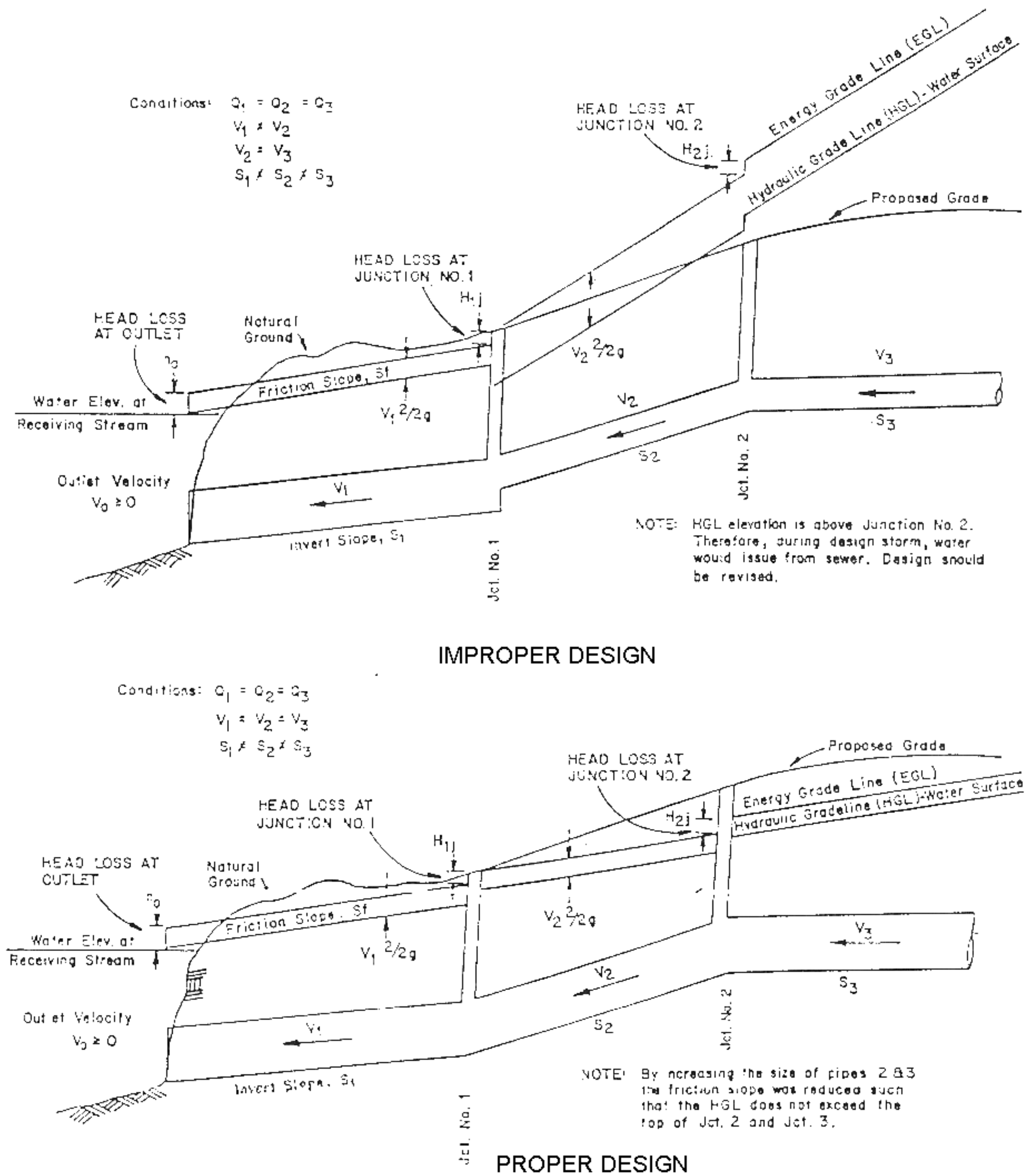


Figure 13-5 Use Of Energy Losses In Developing a Storm Drain System

13.6 Hydraulic Grade Line

13.6.2 HGL Calculation Procedure (continued)

Table 13-5 Hydraulic Grade Line Computation Form

Station	TW	D_0	Q_o	L_o	V_o	$V_o^2/2g$	H_o	S_{fo}	H_f	K_o	C_D	C_d	C_Q	C_p	C_B	K	$K(V_o^2/2g)$	EGL_0 2+7	EGL_i 10+18-+19	HGL EGL-7	TOC Elev
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)

13.7 Computer Programs

To assist with storm drain system design, many computer programs have been developed for the computation of hydraulic grade line. These computer programs can be used to check design adequacy and to analyze the performance of a storm drain system under assumed inflow conditions. The user must understand how the computer program calculates

- the time of concentration,
- how it interpolates the rainfall intensity,
- whether it uses the full flow or design flow velocity for travel time computations
- how it calculates the junction losses

Appendix 13.A presents an example problem using manual methods. It may be used to evaluate computer programs

13.8 References

American Association of State Highway and Transportation Officials. Volume 9, Highway Drainage Guidelines, Storm Drainage Systems. 1992.

Bridge Deck Drainage Guidelines. FHWA Report No. RD-014. December 1986.

Federal Highway Administration. Drainage of Highway Pavements, Hydraulic Engineering Circular No. 22.

Federal Highway Administration. Design of Bridge Deck Drainage, Hydraulic Engineering Circular No. 21. 1993.

Federal Highway Administration. Design of Urban Highway Drainage — The State of The Art, FHWA-TS-79-225. 1979.

Federal Highway Administration. Pavement and Geometric Design Criteria For Minimizing Hydroplaning. FHWA Report No. RD-79-31. December 1979.

Dah-Chen Woo. Public Roads, Vol.52, No. 2 Bridge Drainage System Needs Criteria. U. S. Department of Transportation. September 1988.

Appendix 13.A Example Problem

Storm Drain Example: Crandall Boulevard

Determine the appropriate pipe sizes and profile for the system. Evaluate the HGL.
Use concrete pipe with a Manning's n of 0.013, a minimum pipe diameter of 18 inches.
Tabled below is the data for the system.
Drainage area information is in Table 13-8.1
Rainfall information is in Table 13-8.2
Pipe geometric data is in Table 13-8.3

Step 1. Prepare preliminary layout and number the pipes. Use numbering system from outfall to upstream. Determine the Design Discharge and initial pipe sizes.

TABLE 13-A.1 Drainage basin data.

Inlet No.	Inlet Station	Drainage Area, Acre	"C"	Effective Area "CA"	Time of Concentration, min.
A (Pipe 8)	101+00	6.48	0.51	3.30	10
B (Pipe 9)	102+40	4.95	0.53	2.62	12
C (Pipe 10)	102+40	4.39	0.49	2.15	12
D (Pipe 11)	103+70	3.76	0.53	1.99	10
E (Pipe 12)	105+00	11.03	0.62	6.84	15
F (Pipe 13)	107+80	13.54	0.59	7.99	18
G (Pipe 14)	B106+25	7.42	0.58	4.30	16
H (Pipe 15)	B108+75	6.2	0.8	4.96	12

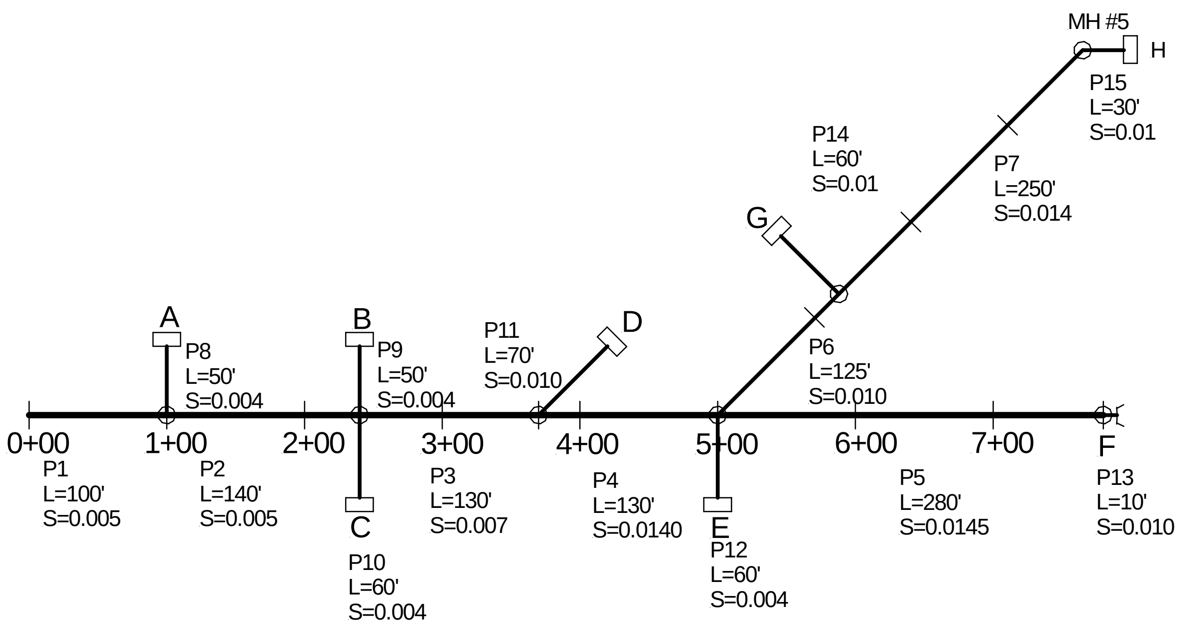
TABLE 13-A.2 Rainfall Data

Time, Min	10	15	20	30	40	50	60
Intensity (in/hr)	4.80	3.96	3.33	2.76	2.25	1.85	1.55

Appendix 13.A Example Problem (continued)

TABLE 13-A.3 Pipe Data

Pipe ID	Diameter, Ft.	Slope, ft/ft	Length, Ft	D/S Invert	U/S Invert
1	48	0.005	100	101.00	101.50
2	48	0.005	140	101.50	102.20
3	48	0.007	130	102.20	103.11
4	42	0.0140	130	103.61	105.43
5	36	0.0145	280	105.93	109.99
6	30	0.010	125	106.43	107.68
7	24	0.014	250	108.18	111.68
8	24	0.004	50		
9	24	0.004	50		
10	24	0.004	60		
11	24	0.01	70		
12	24	0.004	60		
13	36	0.01	10	109.99	110.09
14	24	0.01	60		
15	24	0.01	30	111.68	111.98



STORM DRAIN LAYOUT

Appendix 13.A Example Problem (continued)

Step 2. Determine the Design Discharge and initial pipe sizes.

Begin at upstream of line "B"

Pipe 7. L=250 ft., slope = 0.014
Area "H". CA=4.96.
Inlet Tc=12.0 min. Travel time in connector pipe = 0.06 min.
I=4.4 in/Hr. Q= 4.96*4.4 = 21.82 cfs.

Minimum pipe diameter for full flow:
$D=1.33[Qn/S^{0.5}]^{0.375}$

$D=1.33[21.82*0.013/(0.014)^{0.5}]^{0.375}$

D= 1.85 ft. use 24" pipe.

For 24" pipe, full flow capacity $Q=\frac{0.46\ D^{2.67}S^{0.5}}{n} = \frac{0.46\ (2.0)^{2.67}*(0.014)^{0.5}}{0.013} = 26.65$ cfs

Full flow velocity $V=\frac{0.59\ D^{0.67}S^{0.5}}{n} = \frac{0.59\ (2.0)^{0.67}*(0.014)^{0.5}}{0.013} = 8.48$ ft/sec

From chart 13-18, determine design velocity: Q/Qf=21.82/26.65=0.82, therefore V/Vf=1.13

Design velocity = 1.13*8.48=9.58 ft/sec = 575 ft/min.

Travel time = L/V = (250/575) =0.43 min.

Pipe 6. L=125 ft., slope = 0.01
Area "G"
Additional Ca= 4.30. Total CA=9.26
Inlet Tc=16 min. Travel time in connector pipe = 0.06 min.
System Tc=12.06 + 0.43 = 12.49 minutes, use Tc of 16.06 minutes.
I= 3.82 in/hr. Q=9.26* 3.82 = 35.39 cfs.

Minimum pipe diameter for full flow:
$D=1.33[Qn/S^{0.5}]^{0.375}$

$D=1.33[35.39*0.013/(0.01)^{0.5}]^{0.375}$

D= 2.36 ft., use 2.5' pipe.

For 30" pipe, full flow capacity $Q=\frac{0.46\ D^{2.67}S^{0.5}}{n} = \frac{0.46\ (2.5.5)^{2.67}*(0.010)^{0.5}}{0.013} = 40.86$ cfs

Appendix 13.A Example Problem (continued)

Step 2. Determine the Design Discharge and initial pipe sizes. (continued)

Full flow velocity $V=\frac{0.59\ D^{0.67}S^{0.5}}{n} = \frac{0.59\ (2.5)^{0.67}*(0.010)^{0.5}}{0.013} = 8.32$ ft/sec

From chart 13-18, determine design velocity: Q/Qf=35.39/40.86 = 0.87, therefore V/Vf=1.15

Design velocity = 1.15*8.32=9.57 ft/sec = 574 ft/min.

Travel time = L/V = (125/574) =0.22 min.

Pipe 5. Length =280, slope =0.0145
Area "F"
CA=7.99. Inlet Tc=18 min. Travel time in connector pipe = 0.06 min. Tc=18.06 min.
I=3.60 in/Hr. Q= 7.99*3.6=28.76 cfs.

Minimum pipe diameter for full flow:
$D=1.33[Qn/S^{0.5}]^{0.375}$

$D=1.33[28.76*0.013/(0.0145)^{0.5}]^{0.375}$

D= 2.03 ft., use 36" pipe. Based on allowable headwater for pipe culvert in a headwall.

For 36" pipe, full flow capacity $Q=\frac{0.46\ D^{2.67}S^{0.5}}{n} = \frac{0.46\ (3.0)^{2.67}*(0.0145)^{0.5}}{0.013} = 79.76$ cfs

Full flow velocity $V=\frac{0.59\ D^{0.67}S^{0.5}}{n} = \frac{0.59\ (3.0)^{0.67}*(0.0145)^{0.5}}{0.013} = 11.37$ ft/sec

From chart 13-18, determine design velocity: Q/Qf=28.76/79.76=0.36, therefore V/Vf=0.92

Design velocity = 0.92*11.37=10.46 ft/sec = 628 ft/min.

Travel time = L/V = (280/628) =0.45 min.

Pipe 4. Length= 130, slope = 0.0140
Area "E" Additional CA=6.84
Upstream end, CA= 9.26+7.99=17.25. Total CA=24.09
Inlet Tc= 15 min., Travel time in connector pipe = 0.06 min.
System Tc=18.51 use system Tc of 18.51 minutes. I=3.5 in/hr.
Q=24.09*3.50 = 84.32 cfs.

Minimum pipe diameter for full flow:
$D=1.33[Qn/S^{0.5}]^{0.375}$

$D=1.33[84.32*0.013/(0.0140)^{0.5}]^{0.375}$

Appendix 13.A Example Problem (continued)

Step 2. Determine the Design Discharge and initial pipe sizes. (continued)

Pipe 4. (continued)

D= 3.06 ft., use 3.5' pipe.

For 42" pipe, full flow capacity $Q=\frac{0.46\ D^{2.67}S^{0.5}}{n} = \frac{0.46\ (3.5)^{2.67}*(0.0140)^{0.5}}{0.013} = 118.7$ cfs

Full flow velocity $V=\frac{0.59\ D^{0.67}S^{0.5}}{n} = \frac{0.59\ (3.5)^{0.67}*(0.0140)^{0.5}}{0.013} = 12.43$ ft/sec

From chart 13-18, determine design velocity: Q/Qf=84.32/118.7=0.71, therefore V/Vf=1.10

Design velocity = 1.10*12.43=13.67 ft/sec = 820 ft/min.

Travel time = L/V = (130/820) =0.16 min.

Pipe 3. Length = 130, slope = 0.007
Area "D" Additional CA=1.99

Total CA=24.09+1.99= 26.08 Ac.
Inlet Tc= 10.0 min., system Tc=18.51+0.16 = 18.67, use system Tc of 18.67 minutes.
I= 3.5 in/hr.
Q=26.08*3.5 = 91.29 cfs.

Minimum pipe diameter for full flow:
$D=1.33[Qn/S^{\mathbf{0.5}}]^{\mathbf{0.375}}$

$D=1.33[91.29*0.013/(0.007)^{\mathbf{0.5}}]^{\mathbf{0.375}}$

D= 3.59 ft, use 4.0' pipe.

For 48" pipe, full flow capacity $Q=\frac{0.46\ D^{2.67}S^{0.5}}{n} = \frac{0.46\ (4.0)^{2.67}*(0.007)^{0.5}}{0.013} = 119.9$ cfs

Full flow velocity $V=\frac{0.59\ D^{0.67}S^{0.5}}{n} = \frac{0.59\ (4.0)^{0.67}*(0.007)^{0.5}}{0.013} = 9.54$ ft/sec

From chart 13-18, determine design velocity: Q/Qf=91.29/119.9=0.76, therefore V/Vf=1.13

Design velocity = 1.13*9.54=10.78 ft/sec = 647 ft/min.

Travel time = L/V = (130/647) =0.20 min.

Appendix 13.A Example Problem (continued)

Step 2. Determine the Design Discharge and initial pipe sizes. (continued)

Pipe 2: Length = 140, slope = 0.005
Area “B” & “C” Additional CA= 2.62+2.15 = 4.77 Ac.
Total CA= 26.08+4.77=30.85

Inlet Tc= 12 min. system time = 18.67 + 0.20= 18.87. use system time
I= 3.49 in/hr.
Q=30.85*3.49=107.7 cfs.

Minimum pipe diameter for full flow:
$D=1.33[Qn/S^{\mathbf{0.5}}]^{\mathbf{0.375}}$

$D=1.33[107.7*0.013/(0.005)^{\mathbf{0.5}}]^{\mathbf{0.375}}$

D= 4.07 ft., use 4.0” pipe.

For 48” pipe, full flow capacity $Q=\dfrac{0.46\ D^{2.67}S^{0.5}}{n} = \dfrac{0.46\ (4.0)^{2.67}*(0.005)^{0.5}}{0.013} = 101.3$ cfs
since full flow capacity is less than design Q, velocity is Q/A.

Design Velocity = 107.7/12.566=8.57 ft/sec. = 514 ft/min.

Travel time = L/V = (140/514) =0.27 min.

Pipe 1 Length=100, slope 0.005
Area “A” Additional CA= 3.30 Ac.
Total CA= 30.85+3.30=34.15

Inlet Tc= 10 min., system time = 18.87+0.27= 19.14, use system Tc of 19.14 minutes.
I= 3.43 in/hr.
Q=34.15*3.43=117.2 cfs.

Minimum pipe diameter for full flow:
$D=1.33[Qn/S^{\mathbf{0.5}}]^{\mathbf{0.375}}$

$D=1.33[117.2*0.013/(0.005)^{\mathbf{0.5}}]^{\mathbf{0.375}}$

D= 4.21 ft. Use 4.0 pipe.

For 48” pipe, full flow capacity $Q=\dfrac{0.46\ D^{2.67}S^{0.5}}{n} = \dfrac{0.46\ (4.0)^{2.67}*(0.005)^{0.5}}{0.013} = 101.3$ cfs

since full flow capacity is less than design Q, velocity is Q/A.

Appendix 13.A Example Problem (continued)

Step 2. Determine the Design Discharge and initial pipe sizes. (continued)

Pipe 1 (continued)
Design Velocity = 107.7/12.566=8.57 ft/sec. = 514 ft/min.

Travel time = L/V = (100/514) =0.19 min.

In the next section junction losses will be computed based on the assumed pipe sizes and full flow velocities.

Junction Losses

Step 3. Determine the junction loss coefficients for the initial system design.

At Station 101+00; Upstream end of Pipe 1

Connector pipe at a "T".

$$H=\frac{Q_oV_o-Q_iV_i-Q_lV_l\cos(\theta)}{0.5(A_o+A_i)g}+\frac{V_i^2-V_o^2}{2g}$$

Inlet Pipe 2, Q=107.70
Outlet Pipe 1, Q=117.18
Connector Pipe "A", Q=9.48
$\theta = 90°$

$$H=\frac{117.18*9.32-107.7*8.57-9.48*5.36*\cos(90°)}{0.5(12.57+12.57)*32.2}+\frac{8.57^2-9.32^2}{64.4} = 0.21\text{ ft}$$

$K= H/(V_0^2/2g) = 0.21/(9.32^2/32.2)=0.155$

Pipe	Q	Diameter	A	V	$V^2/2g$	Theta	Hj, ft	K
Inlet Pipe 2	107.70	48	12.57	8.57	1.141	180°	0.21	0.155
Pipe "A"	9.48	18	1.77	5.36	0.447	90°		
Outlet Pipe 1	117.18	48	12.57	9.32	1.350	0°		

At Station 102+40; 2 connector pipes at a manhole

Loss co-efficient, $K= K_0\ C_D\ C_d\ C_Q\ C_B\ C_P$
Where K_0= initial head loss coefficient based on relative access hole size.
C_D= Correction for pipe diameter.
C_d= Correction for flow depth.

Appendix 13.A Example Problem (continued)

At Station 102+40; 2 connector pipes at a manhole (continued)

C_Q= Correction for relative flow.
C_B= Correction for benching.
C_P= Correction for plunging flow.

K_0= initial head loss coefficient

K_0=0.1 (b/Do)(1-sin(□)+1.4 (b/Do)$^{0.15}$ sin (□)

For Pipe 3, □=180□
K_0= 0.1(48/48)(1-sin(180□))+1.4(48/48)$^{0.15}$sin(180□)
= 0.1(1)(1-(0))+1.4(1)$^{0.15}$(0)=0.1

For Pipe "B", □=90□
K_0= 0.1(48/48)(1-sin(90□))+1.4(48/48)$^{0.15}$sin(90□)
= 0.1(1)(1-1))+1.4(1)0.15(1)=0+1.4 = 1.4

For Pipe "C", □=90□
= 0.1(48/48)(1-sin(90□))+1.4(48/48)$^{0.15}$sin(90□)
= 0.1(1)(1-1))+1.4(1)0.15(1)=0+1.4 = 1.4

C_D= Correction for pipe diameter.

Since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/Do=1, then C_D= 1.

C_d= Correction for flow depth.

C_d= 0.5(d/Do)$^{0.6,}$ since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/Do=1, then C_d = 0.5

C_Q= Correction for relative flow.

C_Q= 1+(1-2*sin(□)*(1-Q_i/Q_o)$^{0.75}$

Inlet Pipe 3, Q=91.29 cfs, □= 180□
Connector Pipe "B", Q=9.01 cfs, □=90□
Connector Pipe "C", Q=7.40 cfs, □=90□
Outlet Pipe 21, Q=107.7

For the inlet pipe 3,
C_Q= 1+(1-2*sin(180□))*(1-(91.29/107.7)$^{0.75}$)
C_Q= 1+(1-0)*(1-(0.848) $^{0.75}$) = 1+(1*0.116) =1.116

Appendix 13.A Example Problem (continued)

At Station 102+40; 2 connector pipes at a manhole (continued)

For connector pipe “B”, Q=9.01 cfs, θ=90
C_Q= 1+(1-2*sin(90))*(1-(9.01/107.7)$^{0.75}$)
C_Q= 1+(1-2)*(1-(0.084) $^{0.75}$) = 1+(-1*0.844) =0.155

For connector pipe “C”, Q=7.4 cfs, θ=90°
C_Q= 1+(1-2*sin(90°))*(1-(7.40/107.7)$^{0.75}$)
C_Q= 1+(1-2)*(1-(0.069) $^{0.75}$) = 1+(-1*0.865) =0.135

C_Q= Correction for relative flow.

Pipe	Q	Diameter	A	V	$V^2/2g$	Theta	C_Q
Inlet Pipe 3	91.29	48	12.57	7.26	0.819	180	1.116
Pipe “B”	9.01	24	3.14	2.87	0.128	90	0.155
Pipe “C”	7.40	24	3.14	2.23	0.086	90	0.135
Outlet Pipe 4	107.7	48	12.57	1.141	1.141	0	

C_B= Correction for benching.

Since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/D_o=1, then for a half bench C_B= 0.15

C_P= Correction for plunging flow.

$$C_P = 1+0.2\left[\frac{h}{D_o}\right]*\left[\frac{(h-d)}{D_o}\right]$$

For this location, the connector pipes inverts are within the diameter of the outflow pipe, therefore C_P= 1.

At Station 102+40;
In summary, $K = K_0\ C_D\ C_d\ C_Q\ C_B\ C_P$

Pipe	K_0	C_D	C_d	C_Q	C_B	C_P	K	Vel. Hd	H_f, ft.
3	0.1	1	0.5	1.116	0.15	1	0.0084	1.14	0.01
“B”	1.4	1	0.5	0.155	0.15	1	0.016	1.14	0.02
“C”	1.4	1	0.5	0.135	0.15	1	0.014	1.14	0.02

At Station 103+70; connector pipe at a “Y”, 45 degrees with a sudden change in pipe size from 42 inch to 48 inch.

Appendix 13.A Example Problem (continued)

At Station 103+70; (continued)

For junction:

$$H_j=\frac{Q_oV_o-Q_iV_i-Q_lV_l\cos(\square)}{0.5(A_o+A_i)g}+\frac{V_i^2-V_o^2}{2g}$$

Inlet Pipe 4, Q=84.32
Outlet Pipe 3, Q=91.29
Connector Pipe “D”, Q=6.97, □ = 135□

$$H=\frac{(91.29*7.26-84.32*6.71-6.97*2.22*\cos(45\square))}{0.5(9.62+12.57)*32.2}+\frac{(6.21^2-7.26^2)}{64.4} = 0.093 \text{ ft.}$$

$K= H/(V_0^2/2g) = 0.093/(7.26^2/32.2)=0.114$

Pipe	Q	Diameter	A	V	$V^2/2g$	Theta	Hj, ft	K
Inlet Pipe 4	84.32	48	12.57	6.71	0.699	180□	0.093	0.114
Pipe “D”	6.97	24	3.14	2.22	0.076	135□		
Outlet Pipe 3	91.29	48	12.57	7.26	0.819	0		

For sudden expansion:

$He = K_e(Vi^2/2g)$

For Do/Di=(48/42)=1.14 and Vi=8.76 ft/sec, K_e = 0.1

$He = 0.1[(8.76)^2/64.4] = 0.119$

Therefore loss at upstream end for 42-inch pipe is 0.093+0.119=0.212 ft,

$K=0.212/(7.26^2/64.4)= 0.26$

At Station 105+00: 3 connector pipes at a manhole

Loss co-efficient, $K= K_0\ C_D\ C_d\ C_Q\ C_B\ C_P$

Where K_0= initial head loss coefficient based on relative access hole size.

C_D= Correction for pipe diameter.
C_d= Correction for flow depth.
C_Q= Correction for relative flow.
C_B= Correction for benching.
C_P= Correction for plunging flow.

Appendix 13.A Example Problem (continued)

At Station 105+00: 3 connector pipes at a manhole (continued)

K_0= initial head loss coefficient

Pipe 5, □=180□

= 0.1(48/42)(1-sin(180□))+1.4(48/42)$^{0.15}$sin(180□)

= 0.1(1.14)(1-(0))+1.4(114)$^{0.15}$(0) = 0.114

Connector Pipe 6, Q=35.39, □=135□

= 0.1(48/42)(1-sin(135□))+1.4(48/42)$^{0.15}$sin(135□)

= 0.1(1.14)(1-(0.707))+1.4(1.14)$^{0.15}$(0.707) = 0.033+1.01 = 1.04

Connector Pipe “E”, Q=20.17, □=90□

= 0.1(48/42)(1-sin(90□))+1.4(48/42)$^{0.15}$sin(90□)

= 0.1(1.14)(1-(1))+1.4(1.14)$^{0.15}$(1) = 1.43

C_D= Correction for pipe diameter.

Since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/Do=1, then C_D= 1.

C_d= Correction for flow depth.

C_d= 0.5(d/Do)$^{0.6}$, since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/Do=1, then C_d = 0.5

C_Q= Correction for relative flow.

C_Q= 1+(1-2*sin(□)*(1-Q_i/Q_o)$^{0.75}$

Inlet Pipe 5, Q=28.76, □=180□
Outlet Pipe 4, Q=84.32
Connector Pipe 6, Q=35.39, □=135□
Connector Pipe “E”, Q=20.17, □=90□

For the inlet pipe 5,
C_Q= 1+(1-2*sin(180□))*(1-(28.76/84.32))$^{0.75}$
C_Q= 1+(1-0)*(1-0.341) $^{0.75}$ = 1+(1)*(0.731) =1.731

For connector pipe 6, Q=35.39 cfs, □=135□
C_Q= 1+(1-2*sin(135□))*(1-(35.39/84.32))$^{0.75}$
C_Q= 1+(1-2(0.707))*(1-0.420) $^{0.75}$ = 1+(-0.414)*(0.58) =0.725

Appendix 13.A Example Problem (continued)

At Station 105+00: 3 connector pipes at a manhole (continued)

For connector pipe “E”, Q=20.17 cfs, □=90□
C_Q= 1+(1-2*sin(90□))*(1-(20.17/84.32))$^{0.75}$
C_Q= 1+(1-2)*(1-0.239) $^{0.75}$ = 1+(-1)*(0.815) =0.185

Pipe	Q	Diameter	A	V	$V^2/2g$	Theta	C_Q
Inlet Pipe 5	28.76	36	7.07	4.07	0.257	180□	1.731
Inlet Pipe 6	35.39	30	4.91	7.21	0.807	135□	0.725
Pipe “E”	20.17	30	4.91	4.11	0.262	90□	0.185
Outlet Pipe 4	84.32	42	9.62	8.76	1.193	0□	

C_B= Correction for benching.

Since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/Do=1, then for half bench C_B= 0.15

C_P= Correction for plunging flow.

$$C_P = 1+0.2\left[\frac{h}{D_o}\right]*\left[\frac{(h-d)}{D_o}\right]$$

For this location, the connector pipes inverts are within the diameter of the outflow pipe, therefore C_P= 1.

At Station 105+00:
In summary, $K= K_0\ C_D\ C_d\ C_Q\ C_B\ C_P$

Pipe	K_0	C_D	C_d	C_Q	C_B	C_P	K	Vel. Hd	H_f, ft.
5	0.114	1	0.5	1.731	0.15	1	0.015	1.19	0.02
6	1.04	1	0.5	0.725	0.15	1	0.0565	1.19	0.07
“E”	1.43	1	0.5	0.185	0.15	1	0.016	1.19	0.02

At Station 107+75; Manhole with 180 degree connection.

K_0= initial head loss coefficient

Pipe “5”, Q=28.76, □=180□
= 0.1(48/36)(1-sin(180□))+1.4(48/36)$^{0.15}$sin(180□)
= 0.1(1.33)(1-(0))+1.4(1.33)$^{0.15}$(0) = 0.133

Appendix 13.A Example Problem (continued)

At Station 107+75; Manhole with 180 degree connection. (continued)

$C_D = 1$
$C_d = 0.5$
$C_Q = 1$
$C_B = 0.15$
$C_P = 1$

In summary, $K= K_0\ C_D\ C_d\ C_Q\ C_B\ C_P$

$$K= 0.133*(1)*(0.5)*(1)*(0.15)*(1) = 0.01$$

At Station 108+05; Open pipe inlet.

Ke=0.2, concrete pipe with headwall.

At Station B106+25; Manhole with connector pipe.

Inlet Pipe 7, Q=21.82
Outlet Pipe 6, Q=35.39
Connector Pipe “G”, Q=13.57

Loss co-efficient, $K= K_0\ C_D\ C_d\ C_Q\ C_B\ C_P$

Where K_0= initial head loss coefficient based on relative access hole size.
C_D= Correction for pipe diameter.
C_d= Correction for flow depth.
C_Q= Correction for relative flow.
C_B= Correction for benching.
C_P= Correction for plunging flow.

K_0= initial head loss coefficient

b=48, Do=30”

Pipe 7, □=180□
$K_0 = 0.1(48/30)(1-\sin(180□))+1.4(48/30)^{0.15}(\sin(180□))$
$= 0.1(1.6)(1-(0))+1.4(1.6)^{0.15}(0) = 0.16$

Connector Pipe “G”, □ = 90□
$K_0 = 0.1(48/30)(1-\sin(90□))+1.4(48/30)^{0.15}(\sin(90□))$
$= 0.1(1.6)(1-(1))+1.4(1.6)^{0.15}(1) = 1.50$

Appendix 13.A Example Problem (continued)

At Station B106+25; Manhole with connector pipe. (continued)

C_D= Correction for pipe diameter.

Since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/Do=1, then C_D= 1.

C_d= Correction for flow depth.

$C_d = 0.5(d/D_o)^{0.6}$, since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/D_o=1, then C_d = 0.5

C_Q= Correction for relative flow.

$C_Q = 1+(1-2*\sin(\theta))*(1-Q_i/Q_o)^{0.75}$

Inlet Pipe 7, Q=21.82, θ= 180°
Connector Pipe “G”, Q=13.57, θ = 90°
Outlet Pipe 6, Q=35.39,

For the inlet pipe 7,
$C_Q = 1+(1-2*\sin(180^\circ))*(1-(21.82/35.39))^{0.75}$
$C_Q = 1+(1-0)*(1-0.617)^{0.75} = 1+(1)*(0.487) =1.487$

For connector pipe “G”, Q=13.57 cfs, θ=90°
$C_Q = 1+(1-2*\sin(90^\circ))*(1-(13.57/35.39))^{0.75}$
$C_Q = 1+(1-2)*(1-0.383)^{0.75} = 1+(-(1)*(0.696) =0.304$

Pipe	Q	Diameter	A	V	$V^2/2g$	Theta	C_Q
Inlet Pipe 7	21.82	24	3.14	6.95	0.749	180°	1.487
Pipe “F”	13.57	24	3.14	4.32	0.280	90°	0.304
Outlet Pipe 6	35.39	30	4.91	7.21	0.807	0°	

C_B= Correction for benching.

Since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/Do=1, then for half bench C_B= 0.15

C_P= Correction for plunging flow.

$$C_P = 1+0.2\left[\frac{h}{D_o}\right]*\left[\frac{(h-d)}{D_o}\right]$$

For this location, the connector pipes inverts are within the diameter of the outflow pipe, therefore C_P= 1.

Appendix 13.A Example Problem (continued)

Step 3. Determine the junction loss coefficients for the initial system design. (continued)

At Station B106+25;
In summary, $K = K_0\ C_D\ C_d\ C_Q\ C_B\ C_P$

Pipe	K_0	C_D	C_d	C_Q	C_B	C_P	K	Vel. Hd	H_f, ft.
7	0.16	1	0.5	1.487	0.15	1	0.018	0.81	0.01
“F	1.50	1	0.5	0.304	0.15	1	0.004	0.81	0.00

At Station B108+25, connector pipe at 135 degrees.

K_0= initial head loss coefficient

Pipe “7”, Q=21.82, □=135□
= $0.1(48/24)(1-\sin(135□))+1.4(48/24)^{0.15}\sin(135□)$
= $0.1(2.0)(1-(0.707))+1.4(2.0)^{0.15}(0.707)$ = 0.0.594 + 1.0983 = 1.158

At Station B108+25, connector pipe at 135 degrees. (continued)

$C_D = 1$
$C_d = 0.5$
$C_Q = 1$
$C_B = 0.15$
$C_P= 1$

In summary, $K = K_0\ C_D\ C_d\ C_Q\ C_B\ C_P$

$$K= 1.158*(1)*(0.5)*(1)*(0.15)*(1) = 0.087$$

At Station B108+55; Drop inlet.

Ke=0.2, concrete pipe with headwall.

In the next section based on the junction losses determined above and the actual velocity heads, the hydraulic grade line will be computed

Appendix 13.A Example Problem (continued)

Step 4. Determine the hydraulic grade line for the initial system design.

Hydraulic Grade Line Computation

Flow at outfall is free flow, set HGL at crown of pipe.

Pipe 1. Station 100+00 to Station 101+00
Downstream invert = 101.00.
Upstream invert = 101.50

Downstream Crown=101.00+4.0=105.00

Set HGL at pipe crown, HGL_1=105.00

Pipe	Length	Q	Diameter	A	Vel	H_{vel}	S_f, %	H_f
1	100	117.2	4.0	12.57	9.33	1.35	0.6761	0.68

EGL_1 = HGL_1+ H_{vel}
= 105.00+1.35 = 106.35

For upstream end of pipe, check if flow is open channel or pressure.
EGL_2 =EGL_1+H_f
= 106.35+0.68 =
= 107.03

HGL_2 at U.S. =EGL_2 - H_{vel}
HGL_2 = 107.03-1.35 = 105.68
Crown at U.S.= 105.50, therefore pressure flow.

Pipe 2: Station 101+00 to Station 102+40
Downstream invert = 101.5
Upstream invert = 102.20

At D.S. of pipe 2,
At junction, station 101+00, K=0.155
Junction loss = 0.155*1.35=0.21 ft.

EGL_1= EGL_2 from pipe 1+ junction loss= 107.03+0.21 = 107.24

HGL_1=107.24 – 1.14= 106.10
Crown at D.S.= 101.5+4.0 = 105.50, therefore pressure flow.

Pipe	Length	Q	Diameter	A	Vel	H_{vel}	S_f, %	H_f
2	140	107.7	4.0	12.57	8.57	1.14	0.5645	0.79

Appendix 13.A Example Problem (continued)

Pipe 2: Station 101+00 to Station 102+40 (continued)

For upstream end of pipe, check if flow is open channel or pressure.

$EGL_2 = EGL_1 + H_f$
$= 107.24 + 0.79$
$= 108.03$

HGL_2 at U.S. $= EGL_2 - H_{vel}$
$= 108.03 - 1.14 = 106.89$

Crown at U.S.= 106.2, therefore pressure flow.

Pipe 3: Station 102+40 to Station 103+70

Downstream invert = 102.20
Upstream invert = 103.11

At D.S. of pipe 3.

At junction, station 102+40,
For pipe 3, K=0.0084
Junction loss = 0.01 ft.

$EGL_1 = EGL_2$ from pipe 1+ junction loss= 108.03+0.01 = 108.04

$HGL_1 = 108.04 – 0.82 = 107.22$
Downstream Crown=106.20, pressure flow

Pipe	Length	Q	Diameter	A	Vel	H_{vel}	S_f, %	H_f
3	130	91.29	4.0	12.57	7.26	0.82	0.4056	0.53

For upstream end of pipe, check if flow is open channel or pressure.

$EGL_2 = EGL_1 + H_f$
$= 108.04 + 0.53$
$= 108.57$

HGL_2 at U.S. $= EGL_2 - H_{vel}$
$= 108.57 - 0.82 = 107.75$

Crown at U.S.= 107.11, therefore pressure flow.

Appendix 13.A Example Problem (continued)

Step 4. Determine the hydraulic grade line for the initial system design. (continued)

Pipe 4: Station 103+70 to 105+00
Downstream invert = 103.61
Upstream invert = 105.43

At D.S. of pipe 4.
At junction, station 103+70
For pipe 4, K=0.26
Junction loss = 0.26*0.82 = 0.21 ft.

EGL_1= EGL_2 from pipe 3+ junction loss= 108.57+0.21 = 108.78

HGL_1=108.78 – 1.19= 107.59
Downstream Crown=103.61+3.50 = 107.11, therefore pressure flow.

Pipe	Length	Q	Diameter	A	Vel	H_{vel}	S_f, %	H_f
4	130	84.32	3.5	9.62	8.76	1.19	0.7052	0.92

For upstream end of pipe, check if flow is open channel or pressure.
EGL_2 = EGL_1+H_f
= 108.78+ 0.92
= 109.70

HGL_2 at U.S. =EGL_2 - H_{vel}
= 109.70-1.19 = 108.51

Crown at U.S.= Invert + Dia.= 105.43 + 3.50 = 108.93, therefore open channel flow.

At a pipe slope of 0.01 flow is supercritical. From critical depth chart for circular pipe in chapter 9, critical depth=2.86 ft.
D/D=2.86/3.5= 0.817
From Table 13-B.2 for a flow depth/Diameter ratio of 0.817, C=0.687
A=0.687(3.5^2)=8.42 sq.ft., Vel=Q/A = 84.32/8.42 = 10.01 ft/sec.

Check with Figure 13-3, for a depth of 2.86 ft, d/D=0.82, V/V_{full}= 1.15.
V_c=1.15*8.76=10.07 ft/sec.

Use critical depth of 2.86 ft., V_n=10.01 ft/sec, Vel.Hd=1.56 ft.

HGL_2 = Invert+d_c = 105.43+2.86 = 108.29 > 107.42 at D.S.,
EGL_2=HGL_2 + H_{vel} = 108.29 + 1.56 = 109.85

Appendix 13.A Example Problem (continued)

Step 4. Determine the hydraulic grade line for the initial system design. (continued)

Pipe 5: Station 105+00 to Station 107+80
Downstream invert = 105.93
Upstream invert = 109.99

At D.S. of pipe 5

At junction, station 105+00
Pipe 5 K=0.015
Junction loss = 0.015*1.56 = 0.02 ft.

EGL_1= EGL_2 from pipe 4+ junction loss= 109.85+0.02 = 109.87
For full flow, Hv = 0.26 ft.

HGL_1=109.87 – 0.26= 109.61

Crown = 105.93 + 3.0 = 108.93< 109.61, therefore pressure flow.

Pipe	Length	Q	Diameter	A	Vel	H_{vel}	S_f, %	H_f
5	280	28.76	3.0	7.07	4.07	0.26	0.1885	0.53

For upstream end of pipe, check if flow is open channel or pressure.
EGL_2 = EGL_1+H_f
= 109.87+ 0.53
= 110.40

HGL_2 at U.S. =EGL_2 - H_{vel}
= 110.40-0.26 = 110.14

At U.S. of pipe 5
Invert = 109.99
Crown = 109.99 + 3.0 = 112.99, therefore open channel flow.

For a 36-inch pipe at a slope of 0.0145 and a discharge of 28.76 cfs, flow is supercritical.
At upper end of pipe, flow depth is critical depth.
d_c=1.73 ft and d_n = 1.24 ft.

From critical depth chart for circular pipe in chapter 9, critical depth=1.73 ft.
From Table 13-B.2, for a depth of 1.73 ft., d/D=0.577, C=0.469
A=0.469*(3.0^2) = 4.22
V = 28.76/4.22 = 6.82 ft/sec.

Appendix 13.A Example Problem (continued)

Step 4. Determine the hydraulic grade line for the initial system design. (continued)

Pipe 5: Station 105+00 to Station 107+80 (continued)

H_{vel} = 0.72 ft.

HGL_2 = Invert+d_c = 109.99 + 1.73 = 111.72 > 109.61 at D.S.,

EGL_2=HGL_2 + H_{vel} = 111.72 + 0.72 = 112.44

Pipe 13: Station 107+80 to Station 107+90

Downstream invert = 109.99
Upstream invert = 110.09

At D.S. of pipe 13

At junction, station 107+75
Pipe 13 K=0.01
Junction loss = 0.01*0.72 = 0.01 ft.

EGL_1= EGL_2 from pipe 5+ junction loss= 112.44+0.01 = 112.45

Crown = 109.99 + 3.0 = 112.99>112.45. Therefore open channel flow.

For slope of 0.01, flow is supercritical.
For a 36-inch pipe at a slope of 0.01 and a discharge of 28.76 cfs, d_c=1.73 ft and d_n = 1.24 ft.

$$HGL_1 = \frac{(dc+D)}{2} + \text{invert}$$

$$= \frac{(1.73+3.0)}{2} + 109.99 = 112.36 > 111.79,$$ downstream water surface does not control.

d = 112.36-109.99 =2.37 > 1.73 depth greater than critical depth.
d/D = 2.37/3.0 = 0.79
From table 13-B.2, C= 0.666
A=C*D^2 = 0.666*(3^2) = 6.000
V = 28.76/6.000 = 4.79 ft/sec., H_{vel} = 0.36

EGL_1=HGL_1 + H_{vel} = 112.36 + 0.36 = 112.72

Pipe	Length	Slope, ft/ft	Q	Diameter	A	Vel, full
13	10	0.01	28.76	3.0	7.07	4.07

Check flow depth at upstream end of pipe.

At U.S. of pipe 13
Invert = 110.09
Crown = 110.09 + 3.0 = 113.09

Appendix 13.A Example Problem (continued)

Pipe 13: Station 107+80 to Station 107+90 (continued)
From critical depth chart for circular pipe in chapter 9, critical depth=1.73 ft.
HGL_2 = Invert+d_c = 110.09+1.73 = 111.82 < 112.36 at D.S.
Therefore depth at entrance = 112.36-110.09 = 2.27 ft.
From Table 13-B.2, for a depth of 2.27 ft., d/D=0.757, C= 0.638

$A=0.638*(3.0^2) = 5.742$
V = 28.76/5.742 = 5.01 ft/sec.
H_{vel} = 0.39 ft.

$EGL_2=HGL_2 + H_{vel} = 112.36 + 0.39 = 112.75$

This is a culvert headwall with a supercritical slope, check using culvert analysis.
Results are a headwater at entrance equal to 112.69. This is in outlet control, influenced by "tailwater" condition.

Pipe 6: Station 105+00 to Station B106+25
Downstream invert = 106.43
Upstream invert = 107.68

At D.S. of pipe 6
Crown = 106.43 + 2.5 = 108.93

At junction, station 105+00
Pipe 6 K=0.014
Junction loss = 0.014*0.30 = 0.004 ft., say 0.00

EGL_1= EGL_2 from pipe 4+ junction loss= 109.92+0.00 = 109.92
For full flow, Hv = 0.81 ft.

HGL_1=109.92 – 0.81= 109.11

Crown = 106.43 + 2.50 = 108.90 < 109.11, therefore pressure flow.

Pipe	Length	Q	Diameter	A	Vel	H_{vel}	S_f, %	H_f
6	125	35.39	2.5	4.91	7.21	0.81	0.6652	0.83

Appendix 13.A Example Problem (continued)

Pipe 6: Station 105+00 to Station B106+25 (continued)

For upstream end of pipe, check if flow is open channel or pressure.
For pressure flow,

$EGL_2 = EGL_1 + H_f$
$= 109.92 + 0.83$
$= 110.75$

HGL_2 at U.S. $= EGL_2 - H_{vel}$
$= 110.68 - 0.81 = 109.87$

At U.S. of pipe 6
Invert = 107.68
Crown = 107.68 + 2.5 = 110.18 > 109.87, therefore open channel flow.

At a friction slope of 0.006652 and a pipe slope of 0.01, the flow becomes open channel at

$$L = \frac{(109.11-108.93)}{(0.0100 - 0.006652)} = \frac{0.18}{0.003348} = 53.8 \text{ ft.}$$

For slope of 0.010, flow is supercritical.

For a 30-inch pipe at a slope of 0.01 and a discharge of 35.39 cfs, d_c=2.02 ft and d_n = 1.79 ft.
At upstream end, invert + dc = 107.68 + 2.02 = 109.70 > 109.11, therefore depth is controlled by critical depth at upstream end.

From Table 13-B.2, for a depth of 2.02 ft., d/D=0.808, C=0.680

$A = 0.680 * (2.5^2) = 4.25$ sq.ft.
$V = 35.39/4.25 = 8.33$ ft/sec.
$H_{vel} = 1.08$ ft.

$HGL_2 = \text{Invert} + d_c = 107.68 + 2.02 = 109.70 > 109.11$ at D.S.,
$EGL_2 = HGL_2 + H_{vel} = 109.70 + 1.08 = 110.78$

Pipe 7: Station B106+25 to Station B108+75

Downstream invert = 108.25
Upstream invert = 111.75

At D.S. of pipe 7
Invert = 108.18
Crown = 108.18+ 2.0 = 110.18

Appendix 13.A Example Problem (continued)

Pipe 7: Station B106+25 to Station B108+75 (continued)
At junction, station B106+25
Pipe 7 K=0.02
Junction loss = 0.02*1.07 = 0.02 ft.

Check downstream control,
HGL_1=109.70+0.02=109.72 < 110.18, therefore open channel flow.

Pipe	Length	Slope, ft/ft	Q	Diameter	A	Vel, full
7	250	0.0140	21.82	2.0	3.14	6.95

For slope of 0.014, flow is supercritical.
For a 24-inch pipe at a slope of 0.014 and a discharge of 21.82 cfs, d_c=1.67 ft and d_n = 1.37 ft.

d = 109.72 - 108.18 = 1.54 ft. < d_c.
d/D = 1.54/2.0 = 0.77
From table 13-B.2, C= 0.649. A=C*D^2 = 0.649*(2^2) = 2.596
V = 21.82/2.596 = 8.41 ft/sec.

Check using Figure 13-3, A/A_f=0.84, V/V_f=1.15
A = 0.83*3.14 = 2.61, V=21.82/2.61 = 8.36 ft/sec.
Check V/V_f=1.15*6.95 = 7.99 ft/sec

Use velocity = 8.41 ft/sec, H_{vel} = 1.10
EGL_1=HGL_1 + H_{vel} = 109.72 + 1.10 = 110.82

At U.S. of pipe 7, station B108+75
Crown = 111.68 + 2.00 = 113.68
For slope of 0.015, flow is supercritical.
At upper end of pipe, flow depth is critical depth, d_c=1.67 ft.

From Table 13-B.2, for a depth of 1.67 ft., d/D=0.835, C=0.700
A=0.700*(2.0^2) = 2.80
V = 21.82/2.80 = 7.79 ft/sec.
H_{vel} = 0.94 ft.
HGL_2 = Invert+d_c = 111.68+1.67 = 113.35 > 109.72 at D.S.,
EGL_2=HGL_2 + H_{vel} = 113.35 + 0.94 = 114.29

Appendix 13.A Example Problem (continued)

Step 4. Determine the hydraulic grade line for the initial system design. (continued)

Pipe 15: Station B108+75 to Station B109+05
Downstream invert = 111.68
Upstream invert = 112.13

At D.S. of pipe 15
Pipe is at 135
At junction, station B108+75, K=0.09
Junction loss = 0.09*0.94 = 0.08 ft.

HGL_1=113.35+0.08=113.43
Crown = 111.68+2.0 = 113.68 > 113.43. Therefore open channel flow.

Pipe	Length	Slope, ft/ft	Q	Diameter	A	Vel, full
15	30	0.015	21.82	2.0	3.14	6.95

Depth of flow, 113.43-111.68 = 1.75 ft.

For a 24-inch pipe at a slope of 0.015 and a discharge of 21.82 cfs, d_c=1.67 ft and d_n = 1.34 ft.

D/D = 1.75/2.0 = 0.875, from Table 13-B.2 C= 0.729
A= 0.729 (2^2)= 2.92 sq. ft.
V = 21.82/2.92 = 7.47 ft/sec., H_{vel} = 0.87
EGL_1=HGL_1 + H_{vel} = 113.43+ 0.87 = 114.30

upstream end check depth of flow: 1) HGL is Invert + d_c.
2.) HGL at D/S end

HGL_2= 112.13 + 1.67 = 113.80 > downstream water surface of 113.43, use water depth of 1.67 ft.
For a depth of 1.67 ft, d/D = 0.835, C= 0.7005
A= 0.7005 (2^2)=2.80
V = 21.82/2.80 = 7.79 ft/sec., H_{vel} = 0.94
EGL_2=HGL_2 + H_{vel} = 113.80+ 0.94 = 114.74

This is a drop inlet with a supercritical slope, check using culvert analysis.
Results are a headwater at entrance equal to 115.03. This pipe is in inlet control. If Gutter elevation is 116.2, then “freeboard” is 1.10 ft, greater than 0.5 ft. Outlet pipe does not affect inflow through inlet.

Appendix 13.A Example Problem (continued)

Hydraulic Grade Line Computation Form.

Station	TW HGL_o	D_0	Q_o	L_o	V_o	$V_o^2/2g$	H_o	S_{fo}	H_f	K_o	C_D	C_d	C_Q	C_p	C_B	K	$K(V_o^2/2g)$	EGL_0 2+7	EGL_i 10+18 +19	HGL_i EGL_i-7	Crown Elev
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
100+00 Pipe 1	105.00	4.0	117.2	100.0	9.33	1.35	1.35	0.6761	0.68	-	-	-	-	-	-	0.155	0.21	106.35	107.03	105.68	105.50
101+00 Pipe 2	106.10	4	107.7	140.0	8.57	1.14	0.21	0.5645	0.79	0.1	1	0.5	1.116	0.15	1	0.0084	0.01	107.24	108.03	106.89	106.20
102+40 Pipe 3	107.22	4	91.29	130.0	7.26	0.82	0.01	0.4056	0.53	0.114	-	-	-	-	-	0.114	0.21	108.04	108.57	107.75	107.11
103+70 Pipe 4	107.59	3.5	84.32	130.0	8.76	1.19	0.093	0.7052	0.92	0.26	1	0.5	-	0.15	1	0.26	0.02	108.78	109.85*	108.29*	108.93
105+00 Pipe 5	109.61	3	28.76	280.0	4.07	0.26		0.1885	0.53	0.1	1	0.5	1.731	0.15	1	0.0087	0.01	109.87	112.44*	111.72*	112.99
107+80 Pipe 13	112.36	3.0	28.76	10.0	4.79	0.36		0.01		0.133	1	0.5	1	1	0.15	0.01	0.01	112.72*	112.75*	112.36*	113.09
B105+00 Pipe 6	109.11	2.5	35.39	125.0	7.21	0.81		0.6652	0.83	1.04	1	0.5	0.725	0.15	1	0.14	0.004	109.92	110.78*	109.70*	110.18
B106+25 Pipe 7	109.72	2.0	21.82	250.0	8.41	1.10				0.16	1	0.5	1.487	0.15	1	0.018	0.01	110.82	114.29*	113.35*	113.75
B108+75 Pipe 15	113.42	3.0	21.82	30	7.47	0.87										0.09	0.08	114.30	114.74*	113.80*	114.13

Appendix 13.B Geometric Properties of Circular Channels

Table 13-B.1

Geometric Properties of circular open channels

Area(A)	$\frac{(\text{depth})^2(\square-\sin(\square))}{8}$
Wetted Perimeter (P)	$(\text{depth})(\square/2)$
Hydraulic Radius (R)	$\frac{(\text{depth})*(1-\sin(\square)/(\square))}{4}$
Top Width (T)	$(\text{depth})\sin(\square/2)$
Hydraulic Depth (D_h)	$\frac{(\text{depth})*(\square-\sin(\square))}{8*(\sin(\square/2))}$

□ = angle of subtended arc, in radians.

Table 13-B.2

Coefficient for Area of flow in circular conduit flowing part full.

depth/D	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0013	0.0037	0.0069	0.0105	0.0147	0.0192	0.0242	0.0294	0.0350
0.1	0.0409	0.0470	0.0534	0.0600	0.0668	0.0739	0.0811	0.0885	0.961	0.1039
0.2	0.1118	0.1199	0.1281	0.1365	0.1449	0.1535	0.1623	0.1711	0.1800	0.1890
0.3	0.1982	0.2074	0.2167	0.2260	0.2355	0.2450	0.2546	0.2642	0.2739	0.2836
0.4	0.2934	0.3032	0.3130	0.3229	0.3328	0.3428	0.3527	0.3627	0.3727	0.3827
0.5	0.393	0.403	0.413	0.423	0.433	0.443	0.453	0.462	0.472	0.482
0.6	0.492	0.502	0.512	0.521	0.531	0.540	0.550	0.559	0.569	0.578
0.7	0.587	0.596	0.605	0.614	0.623	0.632	0.640	0.649	0.657	0.666
0.8	0.674	0.681	0.689	0.697	0.704	0.712	0.719	0.725	0.732	0.738
0.9	0.745	0.750	0.756	0.761	0.766	0.771	0.775	0.779	0.782	0.784

Appendix 13.B Geometric Properties of Circular Channels

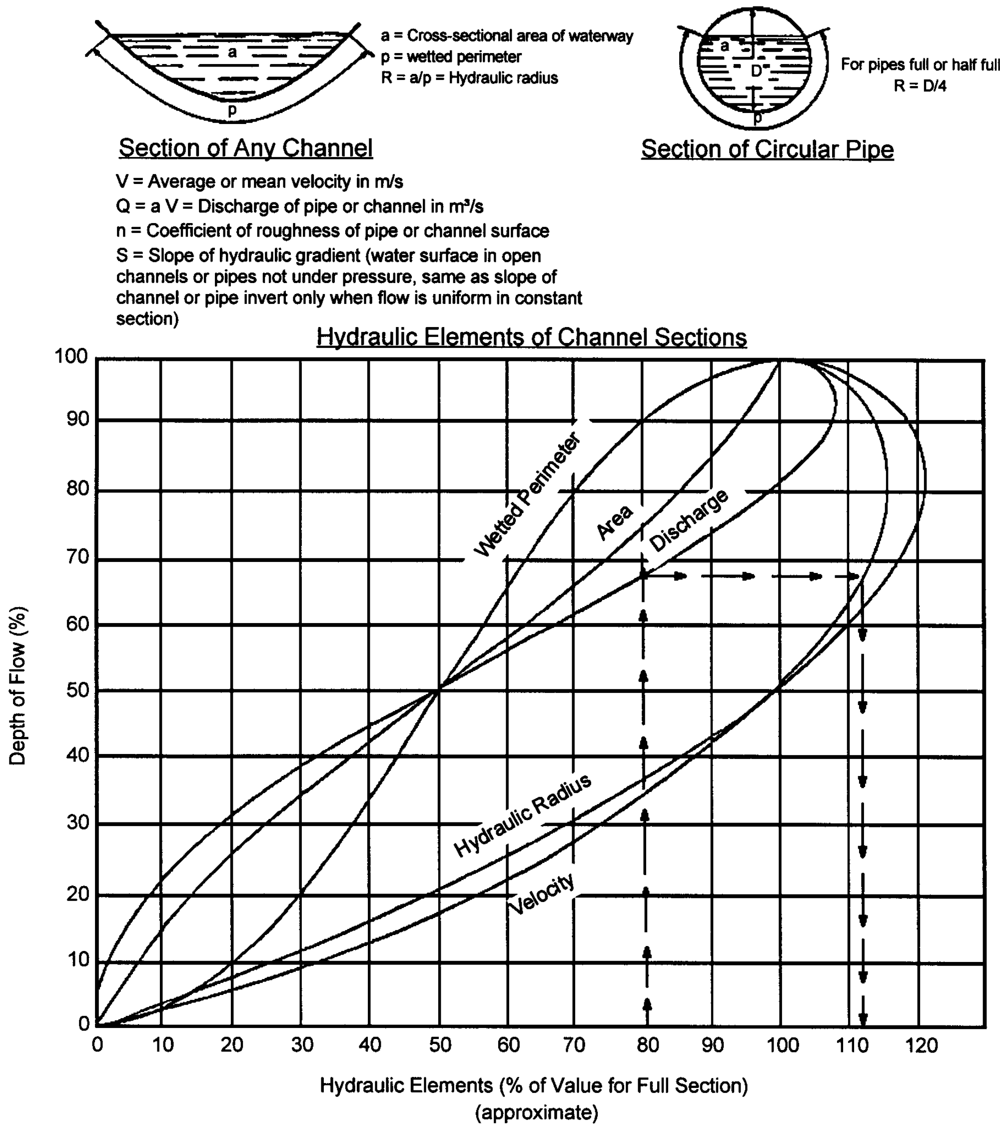


Chart of Hydraulic properties related to flow depth ratio.

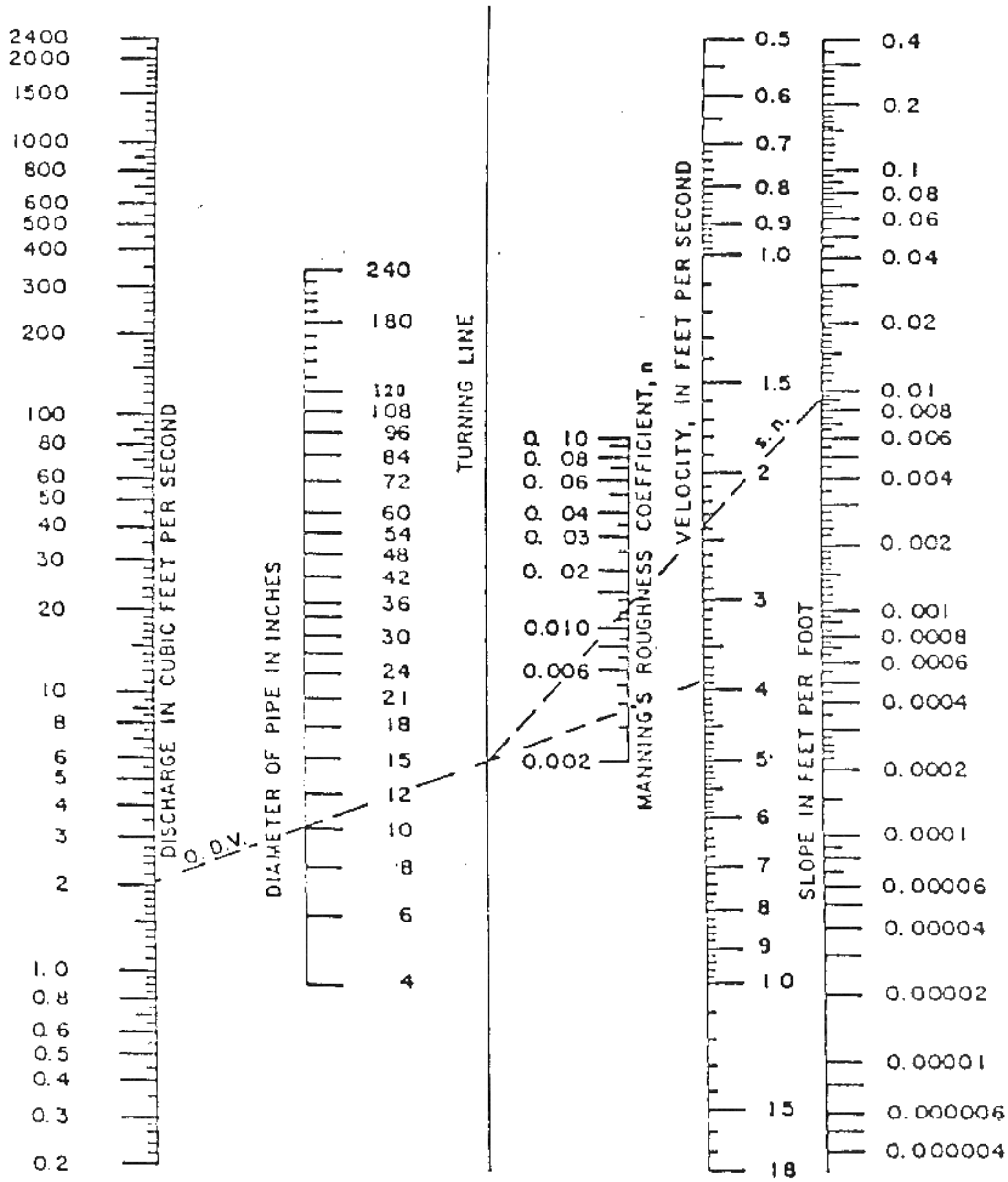
Appendix 13.B Geometric Properties of Circular Channels

TABLE 13-B.3
Pipe Reference Data
Minimum Slope is for a velocity of 3 ft/sec.

$$S = (0.004369)/D^{1.33}$$

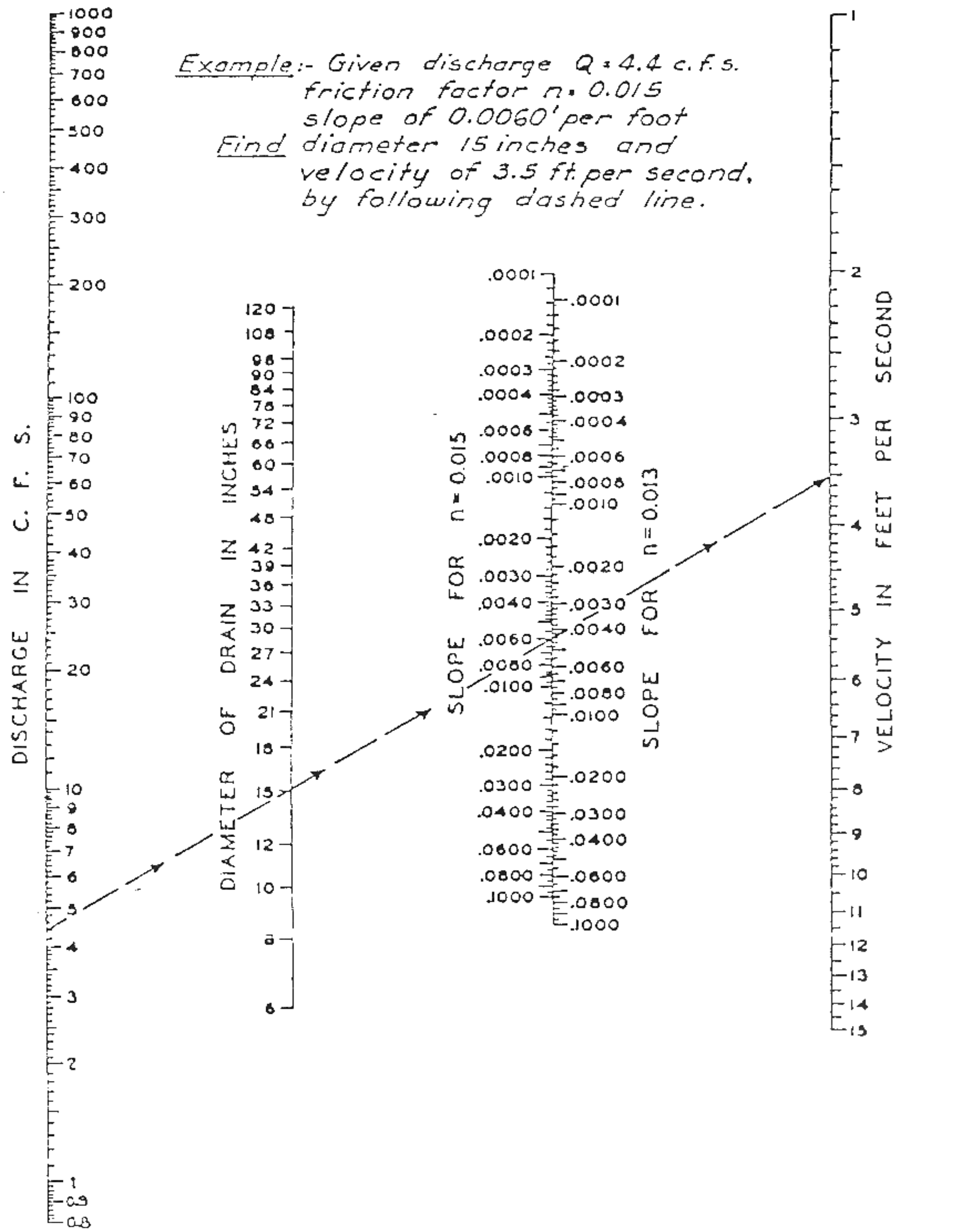
Diameter, Inch	Area, Sq. Ft.	Minimum Slope	Full Flow Discharge, cfs
18	1.767	0.0026	5.30
24	3.142	0.0017	9.43
30	4.909	0.0013	14.73
36	7.069	0.0010	21.21
42	9.621	0.00082	28.86
48	12.566	0.00069	37.70
54	15.904	0.00059	47.71
60	19.635	0.00051	58.90
66	23.758	0.00045	71.27
72	28.274	0.00040	84.22
84	38.484	0.00033	115.4
96	50.266	0.00027	150.8

Appendix 13.C - Storm Drain Design Data and Flow Charts



Flow in Storm Drain

Appendix 13.C - Storm Drain Design Data and Flow Charts



Pipe Flow Nomograph

Appendix 13.C - Storm Drain Design Data and Flow Charts

K for single pipe connection at an access structure.

For a single pipe connection, i.e. one pipe in and one pipe out,

$$K = K_o C_D C_d C_Q C_p C_B \quad \textbf{(13.19)}$$

For a full flow pipe the above values are multiplied

C_D = correction factor for pipe diameter (pressure flow only) =1
C_d = correction factor for flow depth (non-pressure flow only) =0.5
C_Q = correction factor for relative flow= 1
C_B = correction factor for benching =
C_p = correction factor for plunging flow = 1

$$K_o = 0.1(b/ D_o)(1-\sin(\square)) + 1.4(b/ D_o)^{0.15} \sin(\square) \quad \textbf{(13.20)}$$

Therefore K= 0.5 K_o

For b=48 inches

$$K_o = 0.1(48/ D_o)(1-\sin(\square)) + 1.4(48/ D_o)^{0.15} \sin(\square) \quad \textbf{(13.20)}$$

Table 13-C.1
K for pipe in a manhole.

D	K_o =	□ = 180□ sin (□) = 0.0	□ = 157.5□ sin (□) = 0.383	□ = 135□ sin (□)=0.707	□ = 90□ sin (□)= 1.0
24	0.20(1-sin (□)) +1.55 sin (□)	0.1	0.36	0.575	0.775
30	0.16(1-sin (□)) +1.50 sin (□)	0.08	0.335	0.555	0.75
36	0.13(1-sin (□)) +1.46 sin (□)	0.065	0.32	0.535	0.73
42	0.11(1-sin (□)) +1.43 sin (□)	0.055	0.31	0.52	0.715
48	0.10(1-sin (□)) +1.40 sin (□)	0.05	0.30	0.51	0.70
54	0.088(1-sin (□)) +1.375 sin (□)	0.045	0.29	0.50	0.69
60	0.080(1-sin (□)) +1.354 sin (□)	0.04	0.285	0.49	0.675

Appendix 13.D - Storm Drain Design Forms

Storm Drain Computation Form

Storm Drain Computation Sheet

Computed ______ Date ______

Checked ______ Date ______

Route ______

Section ______

County ______

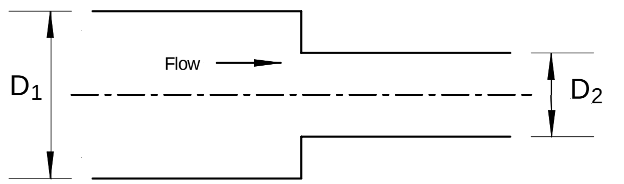
Station		Length (m)	Drainage Area A (ha)		Runoff Coefficient C	A x C		Flow Time (min)		Rainfall Intensity I (mm/h)	Total Runoff 0.0278 CIA = Q (m^3/s)	Diameter Pipe (mm)	Capacity Full (m^3/s)	Velocity (m/s)		Inert Elev.		Manhole Invert Drop	Slope of Drain (m/m)
From	To		Increment	Total		Increment	Total	To Upper End	In Section					Flowing Full	Design Flow	Upper End	Lower End		

Appendix 13.D - Storm Drain Design Forms

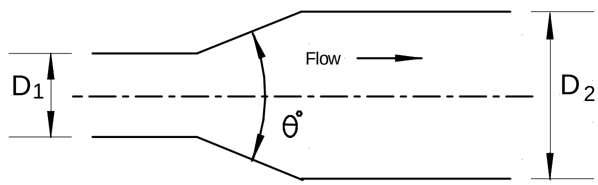
Hydraulic Grade Line Computation Form.

Station	TW HGL_o	D_0	Q_o	L_o	V_o	$V_o^2/2g$	H_o	S_{fo}	H_f	K_o	C_D	C_d	C_Q	C_p	C_B	K	$K(V_o^2/2g)$	EGL_0 2+7	EGL_i 10+18 +19	HGL_i EGL_i-7	Crown Elev
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)

Appendix E Transition Losses (Expansion and Contraction)



Sudden Contraction



Gradual Expansion

Energy Loss Coefficients
Expansion or Contraction
Figure E-1 Contraction and Expansion ,
Sudden and Gradual

Open Channel Flow

Expansion

$$h = \frac{k_e(V_1^2 - V_2^2)}{2g}$$

Contraction

$$h = \frac{k_c*(V_2^2 - V_1^2)}{2g}$$

Where

h = headloss due to expansion or contraction, ft.
k = coefficient for expansion or contraction
V_1= velocity at upstream end.
V_2= velocity at downstream end.
g = acceleration due to gravity, 32.2 ft/sec^2

Pressure Flow

$$h = \frac{k*(V_2^2)}{2g}$$

Where

h = headloss due to expansion or contraction, ft.
k = coefficient for expansion or contraction
V= velocity of flow in the smaller diameter pipe.
g = acceleration due to gravity, 32.2 ft/sec^2

Appendix E Transition Losses (Expansion and Contraction) (continued)

Table E-1
Energy Loss Coefficient, k_e
Open Channel Flow
Gradual Expansion

D_2/D_1	Angle of Cone, Degrees						
	10	**20**	**45**	**60**	**90**	**120**	**180**
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3.0	0.17	0.40	0.86	1.02	1.06	1.04	1.00
D_2/D_1 = Ratio of diameter of large pipe to diameter of smaller pipe. Angle of cone in degrees between the sides of the tapering section.							

For gradual contractions $k_c = 0.5*k_e$.

Table E-2
Energy Loss Coefficient, k_c
Open Channel Flow
Sudden Contraction

D_2/D_1	k_c
0	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1.0	0.0
D_2/D_1 = Ratio of diameter of large pipe to diameter of smaller pipe.	

Appendix E Transition Losses (Expansion and Contraction) (continued)

Table E-3
Coefficient k_e
Pressure Flow

D_2/D_1	Angle of Cone, degrees													
	2	**4**	**6**	**8**	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**60**
1.1	0.01	0.01	0.01	0.02	0.03	0.05	0.10	0.13	0.16	0.18	0.19	0.20	0.21	0.23
1.2	0.02	0.02	0.02	0.03	0.04	0.09	0.16	0.21	0.25	0.29	0.31	0.33	0.35	0.37
1.4	0.02	0.03	0.03	0.04	0.06	0.12	0.23	0.30	0.36	0.41	0.44	0.47	0.50	0.53
1.6	0.03	0.03	0.04	0.05	0.07	0.14	0.26	0.35	0.42	0.47	0.51	0.54	0.57	0.61
1.8	0.03	0.04	0.04	0.05	0.07	0.15	0.28	0.37	0.44	0.50	0.54	0.58	0.61	0.65
2.0	0.03	0.04	0.04	0.05	0.08	0.16	0.30	0.39	0.48	0.54	0.58	0.62	0.65	0.70
2.5	0.03	0.04	0.04	0.05	0.08	0.16	0.30	0.39	0.48	0.55	0.59	0.63	0.66	0.71
3.0	0.03	0.04	0.04	0.05	0.08	0.16	0.31	0.40	0.48	0.55	0.59	0.63	0.66	0.71
	0.03	0.04	0.05	0.06	0.08	0.16	0.31	0.40	0.49	0.56	0.60	0.64	0.67	0.72

D_2/D_1 = Ratio of diameter of large pipe to diameter of smaller pipe.
Angle of cone in degrees between the sides of the tapering section.

Gradual Enlargement

Table E-4
Coefficient k_e
Pressure Flow
Sudden Enlargement

D_2/D_1	Velocity, V_1, ft/sec												
	2	**3**	**4**	**5**	**6**	**7**	**8**	**10**	**12**	**15**	**20**	**30**	**40**
1.2	0.11	0.10	0.10	0.10	0.10	0.10	0.10	0.09	0.09	0.09	0.09	0.09	0.08
1.4	0.26	0.26	0.25	0.24	0.24	0.24	0.24	0.23	0.23	0.22	0.22	0.21	0.20
1.6	0.40	0.39	0.38	0.37	0.37	0.36	0.36	0.35	0.35	0.34	0.33	0.32	0.32
1.8	0.51	0.49	0.48	0.47	0.47	0.46	0.46	0.45	0.44	0.43	0.42	0.41	0.40
2.0	0.60	0.58	0.56	0.55	0.55	0.54	0.53	0.52	0.52	0.51	0.50	0.48	0.47
2.5	0.74	0.72	0.70	0.69	0.68	0.67	0.66	0.65	0.64	0.63	0.62	0.60	0.58
2.5	0.74	0.72	0.70	0.69	0.68	0.67	0.66	0.65	0.64	0.63	0.62	0.60	0.58
3.0	0.83	0.80	0.78	0.77	0.76	0.75	0.74	0.73	0.72	0.70	0.69	0.67	0.65
4.0	0.92	0.89	0.87	0.85	0.84	0.83	0.82	0.80	0.79	0.78	0.76	0.74	0.72
5.0	0.96	0.93	0.91	0.89	0.88	0.87	0.86	0.84	0.83	0.82	0.80	0.77	0.75
10.0	1.00	0.99	0.96	0.95	0.93	0.92	0.91	0.89	0.88	0.86	0.84	0.82	0.80
	1.00	1.00	0.98	0.96	0.95	0.94	0.93	0.91	0.90	0.88	0.86	0.83	0.81

D_2/D_1 = Ratio of diameter of large pipe to diameter of smaller pipe.
V_1 = Velocity in smaller pipe.

Appendix E Transition Losses (Expansion and Contraction) (continued)

Table E-5
Coefficient k_c
Pressure Flow
Sudden Contraction

D_1/D_2	Velocity, V_1, ft/sec												
	2	**3**	**4**	**5**	**6**	**7**	**8**	**10**	**12**	**15**	**20**	**30**	**40**
1.1	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.06
1.2	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.09	0.10	0.11
1.4	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.18	0.18	0.18	0.18	0.19	0.50
1.6	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.25	0.25	0.25	0.24
1.8	0.34	0.34	0.34	0.34	0.34	0.34	0.33	0.33	0.32	0.32	0.31	0.29	0.27
2.0	0.38	0.8	0.37	0.37	0.37	0.37	0.36	0.36	0.35	0.34	0.33	0.31	0.29
2.2	0.40	0.40	0.40	0.39	0.39	0.39	0.39	0.38	0.37	0.37	0.35	0.33	0.30
2.5	0.42	0.42	0.42	0.41	0.41	0.41	0.40	0.40	0.39	0.38	0.37	0.34	031
3.0	0.44	0.44	0.44	0.43	0.43	0.43	0.42	0.42	0.41	0.40	0.39	0.36	0.33
4.0	0.47	0.46	0.46	0.46	0.45	0.45	0.45	0.44	0.43	0.42	0.41	0.37	0.34
5.0	0.48	0.48	0.47	0.47	0.47	0.46	0.46	0.45	0.45	0.44	0.42	0.38	0.35
10.0	0.49	0.48	0.48	0.48	0.48	0.47	0.47	0.46	0.46	0.45	0.43	0.40	0.36
	0.49	0.49	0.48	0.48	0.48	0.47	0.47	0.47	0.46	0.45	0.44	0.41	0.38

D_2/D_1 = Ratio of diameter of large pipe to diameter of smaller pipe.
V_1 = Velocity in smaller pipe.

CHAPTER 14

PUMP STATIONS

Chapter 14 - Pump Stations
Table Of Contents

14.1 Overview

14.1.1 Introduction

Stormwater pump stations are used to remove stormwater for highway sections that cannot be drained by gravity. Because of their high construction and operational cost and potential cost of malfunctions, their use should be limited to where no other system is feasible. Stormwater pump stations generally have high short-term capacity requirements and infrequent use, making them very costly on a per-use basis. The cost of a gravity system must be compared with the life-cycle costs (construction, maintenance and operation) of the pumping station. A pumping station should only be used when the life-cycle cost of the pump station is demonstratively less than the alternative gravity system.

The hydraulic analysis of a pump station involves the interrelationship of 3 components:
- the inflow hydrograph,
- the storage capacity of the wet well and the outside storage, and
- the discharge rate of the pumping system.

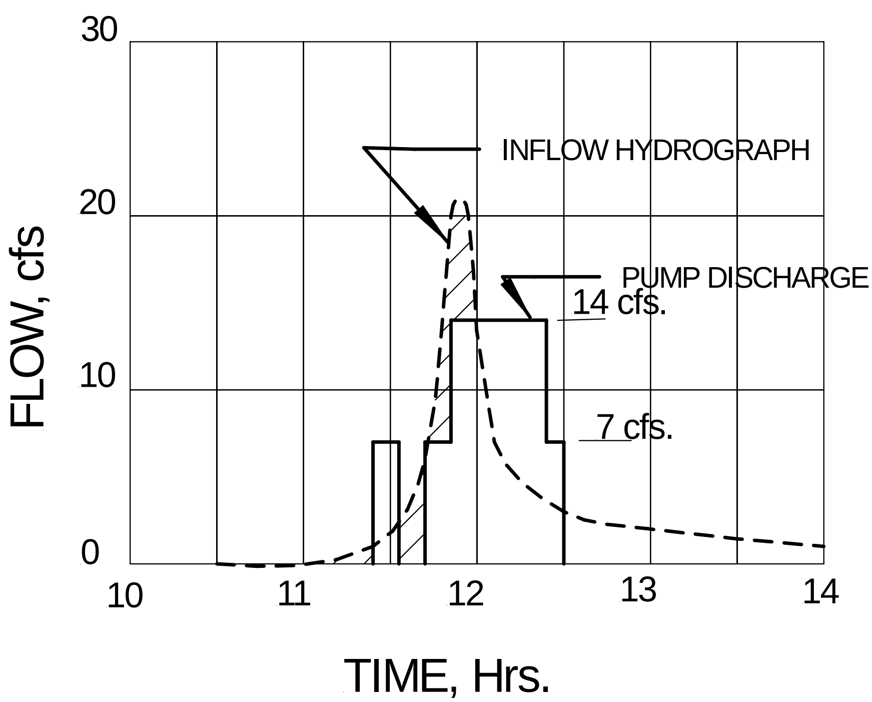


Fig 14-1 Pump Station Operation Hydrographs

The inflow hydrograph is determined by the physical factors of the watershed and regional climatological factors. The discharge rate of the pump station is often controlled by outfall capacity. Therefore, the main objective in pump station design is to store enough inflow (volume of water under the inflow hydrograph) to allow station discharge to meet specified limits while not exceeding the freeboard requirements at the inlets. A minimum volume of storage is required to prevent excessive pump cycling. Even where there are no limitations to pump station discharge, a design that balances storage and pumping capacity provides the most economical design since storage permits use of smaller and/or fewer pumps.

14.1 Overview (continued)

14.1.2 Design Choices

There are many choices to be made regarding pump stations for management of storm water along highways; some are driven by topography and the interaction of collection and discharge systems, others are related to construction and operating costs. Presented below are some of the areas to be considered.

Location/Building Design

- Site Access
- Site and Building Lighting
- Site and Building Security
- Equipment Access
    - Roof Hatches
    - Monorails
- Safe Workspace Environment
    - Potable water supply
    - Ventilation
    - Confined Space Hazard sensors

Operational/Mechanical Design

- Type: Wet pit vs. Dry Pit.

- Pumps:
    - Power & Back-up Systems
    - Type
    - Drive
    - Capacity and Number
    - Operational Size and Volume Requirements
        - Submergence Depth
        - Side Clearances
    - Cycling Plan
        - Hours, Number of Starts
    - Solids Handling Capacity

- Operation Plan/Storage

- Operating Monitoring Controls
    - Automatic and Manual Operation
    - Backup Systems
    - Non-operation Signals
    - Water-level Sensors
    - Hour meters
    - Number-of-starts meters

14.1 Overview (continued)

Operational/Mechanical Design (continued)

- Intake System
  - Trash Racks
  - Inlet type

- Discharge: Force Main vs. Gravity
  - Flap Gates
  - Valves

Many of the decisions regarding the above are currently based on engineering judgment and experience. To assure cost-effectiveness, the designer should assess each choice and develop economic comparisons of alternatives on the basis of annual cost. However, some general recommendations can be made which will help minimize the design effort and the cost of these expensive drainage facilities. These are discussed in the following pages of this chapter.

In the development of the Valley Freeway System in the Phoenix area, ADOT has adopted/standardized a design approach applicable to urban freeway pump stations. The station is based on a peak pump capacity of 200 cfs using 4-50 cfs (4-22,500 gpm) pumps. The elements of this standardized design will be presented as appropriate. Discussions with ADOT maintenance personnel should be had early in the design process.

For further information on the design and use of pump stations, see *Highway Storm Water Pump Station Design Manual*, FHWA-HEC-24 and the other references given at the end of this chapter. The Hydraulic Institute, 9 Sylvan Way, Parsippany, New Jersey, 07054-3802 has developed standards for pumps. Pump station design should be consistent with these standards.

14.2 Symbols And Definitions

To provide consistency within this chapter as well as throughout this manual the following symbols will be used. These symbols were selected because of their wide use in the technical publications.

Table 14-1 Symbols And Definitions

Symbol	Definition	Units
A	Minimum distance from back wall to trash rack	ft
B	Maximum distance between a pump and the backwall	ft
C	Average distance from floor to pump intake	ft
D	Pump diameter	in.
DHW	Design high water elevation	ft
H_f	Friction head	ft
H_l	Losses through fittings, valves, etc.	ft
H_s	Maximum static head	ft
H_t	Storage depth	ft
H_v	Velocity head	ft
H_x	Depth for storage volume	ft
INV	Inlet invert elevation	ft
L	Length of wet well	ft
N	Number of equal size pumps	-
NPSH	Net Positive Suction Head	ft
Q_i	Inflow	ft^3/sec
Q_p	Total capacity of all pumps Peak Discharge Rate	ft^3/sec
S	Minimum submergence at the intake	ft
t_c	Minimum allowable cycle time	sec
TDH	Total dynamic head	ft
V_t	Total cycling storage volume	ft^3
V_x	Individual pump cycling volumes	ft^3
W	Minimum required distance between pumps	ft
Y	Minimum level floor distance upstream of pump	ft

1 Ft^3 = 7.481 gallons
1 cfs = 448 gpm

14.3 Design Considerations

14.3.1 Location

Economic and design considerations dictate that the pump station be located relatively near the low point of the highway. Hopefully a frontage road or overpass is available for easy access to the station. The station and access road should be located on high ground so that access can be obtained if the highway becomes flooded. Soil borings should be made during the selection of the site to determine the allowable bearing capacity of the soil and to identify any potential problems

Decisions regarding location and building architecture need to be made during the concept phase so station design can proceed in an efficient manner. Following are some items that are to be considered for locating the pump station site:

- Building and site access is safe and monitorable.
- Pump stations shall be situated so that the facility is accessible during storm events up to the 100-year event.
- Ample parking and working areas should be provided adjacent to the station for maintenance and repair vehicles.
- The station facade and grounds are aesthetically compatible with the surrounding area.

14.3.2 Design Capacity

Frequency

The design capacity of a pump station is determined by its location within the drainage system. It is ADOT practice that pump stations and appurtenant storage system for draining roadway sumps that are considered "depressed" shall accommodate the inflow for a 50-year storm event in a manner that meets the design spread criteria. It is desirable to evaluate the total drainage system for the 100-year storm event to determine the extent of flooding and the associated risk.

Contributing Area

Hydrologic design should be based on the ultimate development of the area that must drain to the station. Every consideration should be made to keep the contributing drainage area as small as possible. Water that originates outside of the depressed areas should not be allowed to enter the depressed areas because of the need to pump all of this water. Only flows that cannot be by-passed or passed through should be collected. The contributing drainage area should be isolated to prevent off-site flows being diverted to the pump station.

Storage

Storage, in addition to that which exists in the wet well, should be evaluated at all pump station sites. For most highway pump stations, the high flows of the inflow hydrograph will occur over a relatively short time. Additional storage will greatly reduce the peak pumping rate required. An economic analysis can be used to determine the optimum combination of storage and pumping capacity. Since most highway-related pump stations are associated with a localized low point, it is not reasonable to consider above ground storage. The simplest form of storage for these depressed situations is either the enlargement of the collection system or the construction of an underground storage facility or a combination of the two. These can typically be constructed under the roadway area and will not require additional right-of-way.

14.3 Design Considerations (continued)

14.3.2 Design Capacity (continued)

When storage is used to reduce peak flow rates, a routing procedure must be used to design the system. To determine the discharge rate, the routing procedure integrates three independent elements: the inflow hydrograph, the stage-storage relationship, and the stage-discharge relationship.

14.3.3 Pit Types

Basically, there are two types of stations: wet-pit and dry-pit.

Wet-Pit Stations - In the wet-pit station, the pumps are submerged in a wet well or sump with the motors and the controls located overhead. With this design, the stormwater is pumped vertically through a riser pipe. The motor is commonly connected to the pump by a long drive shaft located in the center of the riser pipe. See Figure 14-2 for typical layout.

Dry-Pit Stations - Dry-pit stations consist of two separate elements: the storage box or wet well and the dry well. Stormwater is stored in the wet well which is connected to the dry well by horizontal suction piping. Centrifugal pumps are usually used. Power is provided by either close-coupled motors in the dry well or long drive shafts with the motors located overhead. The main advantage of the dry-pit station for stormwater is the availability of a dry area for personnel to perform routine and emergency pump and pipe maintenance. See Figure 14-3 for typical layout.

Roof Hatches And Monorails

It will be necessary to remove motors and pumps from the station for periodic maintenance and repair. Removable roof hatches located over the equipment is a cost-effective way of providing this capability. Mobile cranes can simply lift the equipment directly from the station onto maintenance trucks. Monorails are usually more cost-effective for larger stations.

Wet Pit Size

In the wet pit, the plan area must accommodate the distance between the pumps, the clearance to the sidewalls, the flow length approaching the pump, and the minimum distance from floor to the underside of the bell. The station depth should be minimum. No more depth than that required for pump submergence and clearance below the inlet invert is necessary, unless foundation conditions dictate otherwise. For the design event, typically the highway operational frequency, the top water surface shall be no higher than that which will result in a hydraulic grade line at the critical inlets with a freeboard of 0.5 foot to the gutter elevation.

14.3 Design Considerations (continued)

14.3.3 Pit Types (continued)

Method of Construction
The method of construction has a major impact on the construction cost of pump station. For stormwater pump station, yearly operating costs are usually insignificant compared to construction costs. Therefore, the type of construction should be chosen carefully, between open-pit construction or braced construction. Soil conditions are the primary factor in selecting the most cost-effective alternative.

Recommendation for Design
Since dry-pit stations are as much as 60% more expensive than wet-pit stations, wet-pit stations are recommended.

14.3.3.1 ADOT Freeway Design Practice

Use a wet well design. Wet well sizing is based on guidelines from the **Hydraulic Institute Standards**. The wet-well must be sealed from the engine room and control room. The wet well area will include explosion proof lighting. A gas tight seal shall be provided between the wet well and the engine area. Access to the wet well will be by a separate enclosed stairway that has a push/pull intake and exhaust system.

14.3.4 Pump Types

14.3.4.1 Flow Types

The most common types of stormwater pumps are axial flow (propeller), radial flow (impeller) and mixed flow (combination of the previous two). Each type of pump has its particular merits. It is difficult to have a totally objective selection procedure. Cost, reliability, operating and maintenance requirements are all important considerations when making the selection. First costs are usually of more concern than operating costs in stormwater pump stations since the operating periods during the year are relatively short. Ordinarily, first costs are minimized by providing as much storage as possible, with two or three small pumps.

Axial Flow Pumps - Axial flow pumps lift the water up a vertical riser pipe; flow is parallel to the pump axis and drive shaft. They are commonly used for low head, high discharge applications. Axial flow pumps do not handle debris particularly well because the propellers will bend or possibly break if they strike a relatively large, hard object. Also, fibrous material will wrap itself around the propellers.

Radial Flow Pumps - Radial flow pumps utilize centrifugal force to move water up the riser pipe. They will handle any range of head and discharge, but are the best choice for high head applications. Radial flow pumps generally handle debris quite well. A single vane, non-clog impeller handles debris the best because it provides the largest impeller opening. The debris handling capability decreases with an increase in the number of vanes since the size of the openings decrease.

14.3 Design Considerations (continued)

14.3.3.1 Pit Type (continued)

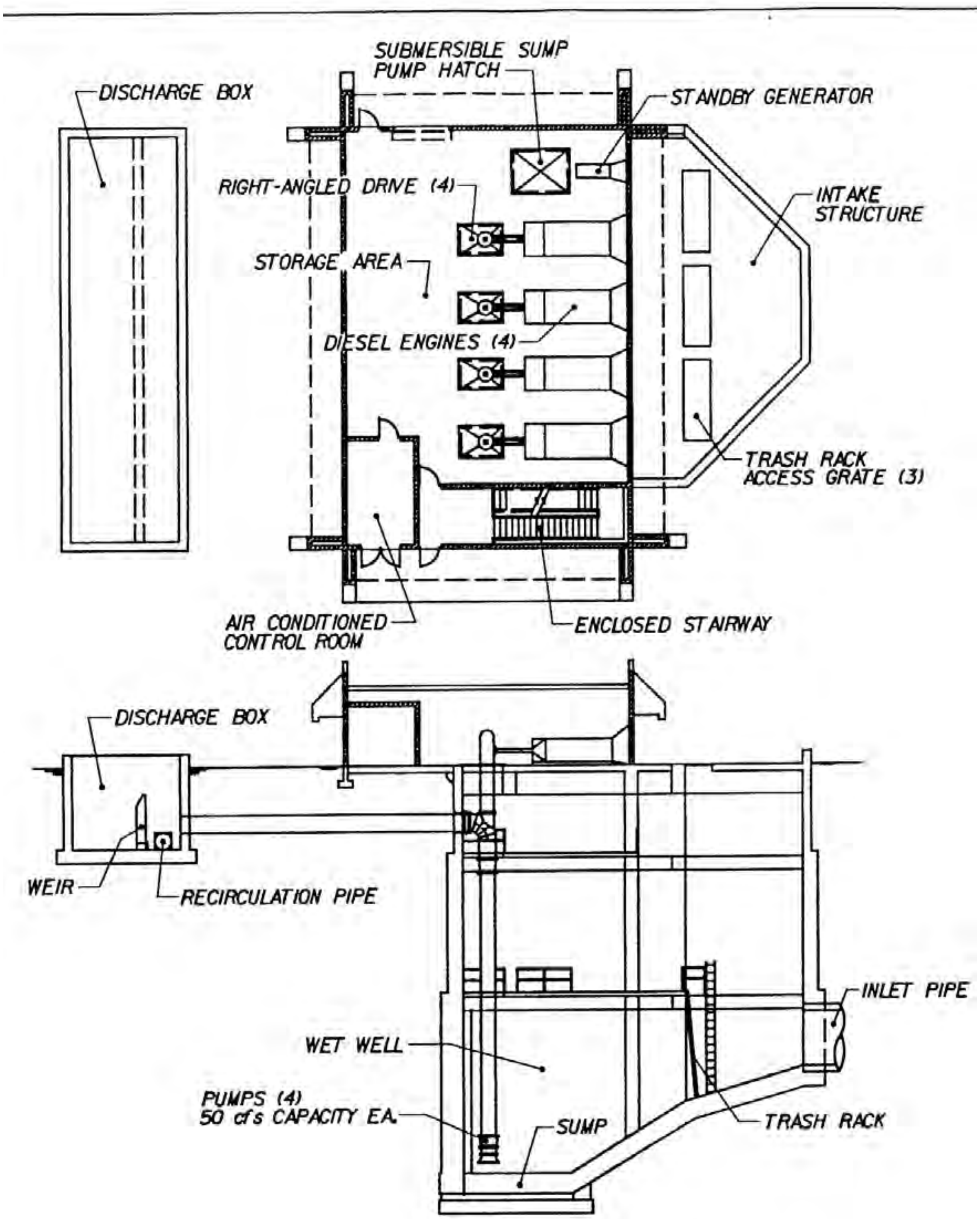


Figure 14-2 Typical Wet-Pit Station

14.3 Design Considerations (continued)

14.3.3.1 Pit Type (continued)

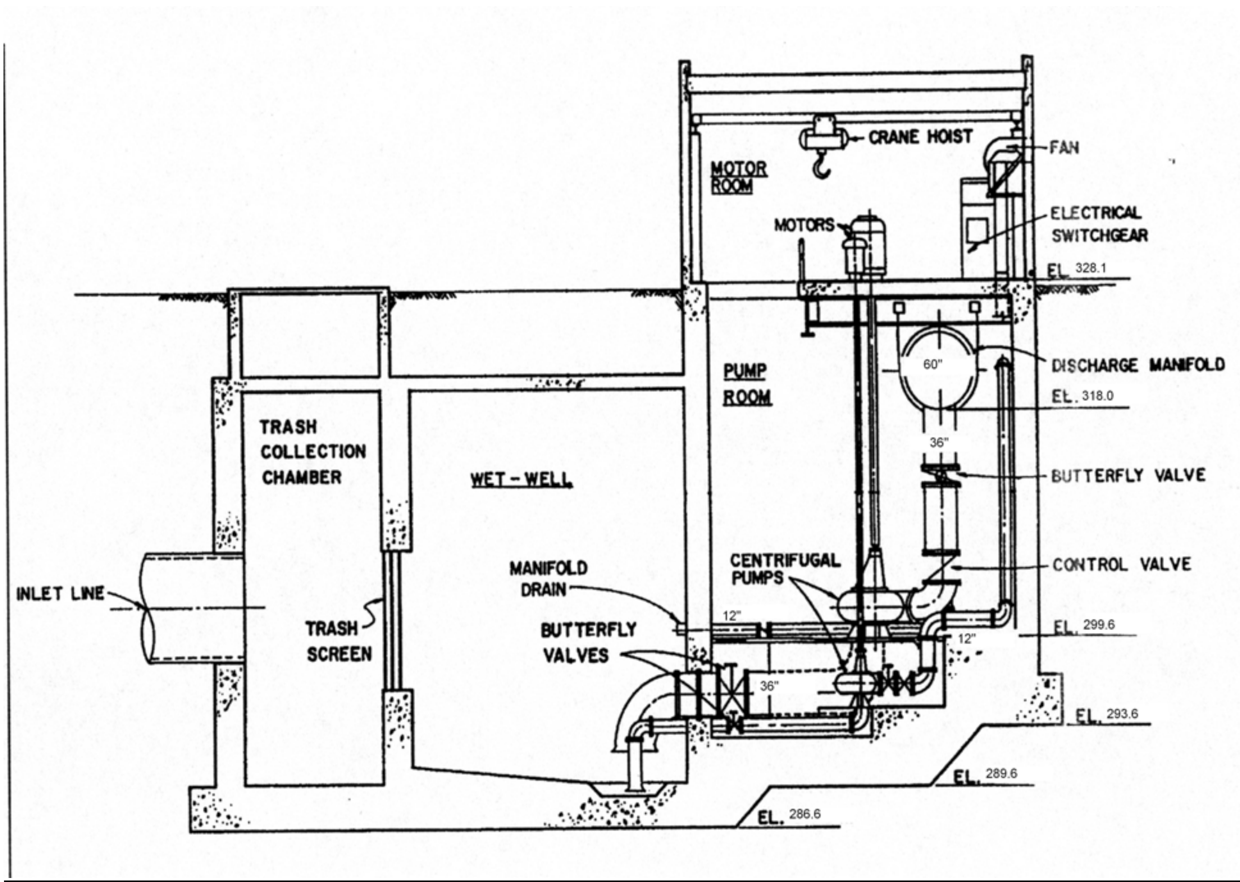


Figure 14-3 Typical Dry-Pit Station

Source: FHWA IP-82-17

14.3 Design Considerations (continued)

14.3.4.1 Flow Types (continued)

Mixed Flow Pumps - Mixed flow pumps are very similar to axial flow except they create head by a combination of lift and centrifugal action. An obvious physical difference is the presence of the impeller "bowl" just above the pump inlet. They are used for intermediate head and discharge applications and handle debris slightly better than propellers. These pumps can be driven by motors or engines housed overhead or in a dry well or by submersible motors located in a wet well. Submersible pumps frequently provide special advantages in simplifying the design, construction, maintenance and, therefore, cost of the pumping station. Use of anything other than a constant speed, single stage, single suction pump would be rare. Pumps shall be capable of handling solids 3” or smaller.

14.3.4.2 Submergence

Submergence is the depth of water above the pump inlet necessary to prevent cavitation and vortexing. It varies significantly with pump type, speed, inlet bell diameter and atmospheric pressure. This dimension is provided by the pump manufacturer and is determined by laboratory testing. A very important part of submergence is the required net positive suction head (NPSH) because it governs cavitation. The available NPSH should be calculated and compared to the manufacturer's requirement. Additional submergence may be required at higher elevations. As a general rule, radial flow pumps require the least submergence while axial flow pumps require the most.

One popular method of reducing the submergence requirement (and therefore the station depth) for axial and mixed flow pumps, when cavitation is not a concern, is to attach a suction umbrella. A suction umbrella is a dish-shaped steel plate attached to the pump inlet that improves the entrance conditions by reducing the intake velocities. For umbrella velocities of 2 ft/sec or less, a submergence to pump bell diameter ratio of 0.80 can be used. Required submergence criteria developed by the Hydraulic Institute is shown in Figure 14-4 as presented in “American National Standard for Pump Intake Design”.

14.3 Design Considerations (continued)

14.3.4.2 Submergence (continued)

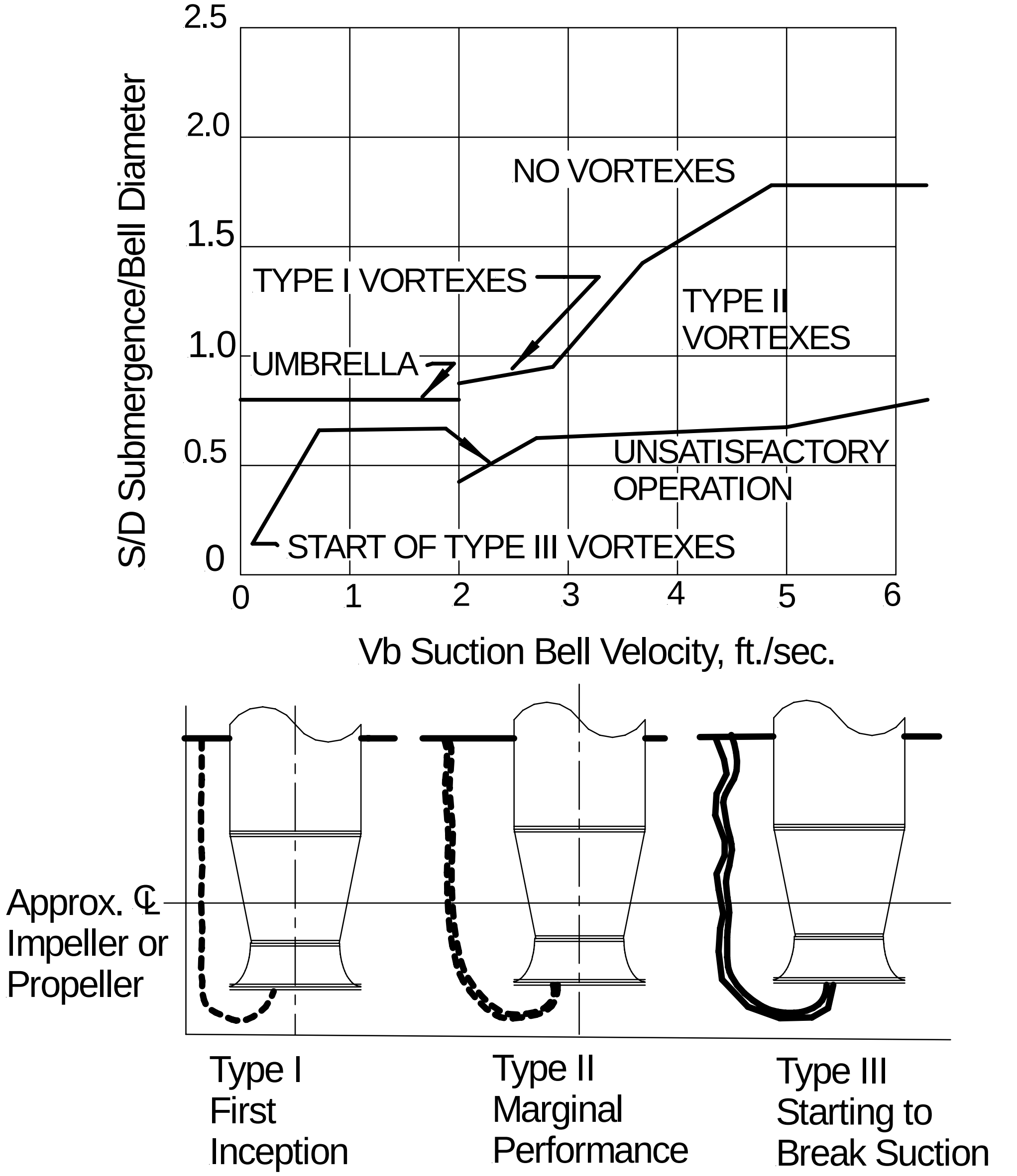


Figure 14-4 Vortex Description And Submergence Requirements

14.3 Design Considerations (continued)

14.3.4.3 Standby/Spare Pumps

Considering the short duration of high inflows, the relative infrequency of the design storm event, the odds of a malfunction and the typical consequences of a malfunction, spare or standby pumps are not warranted in stormwater applications. If the consequences of a malfunction are particularly critical, it is more appropriate to add another main pump and reduce their size accordingly.

14.3.4.4 Sump Pumps

It is ADOT preference for freeway installations to use a station electric “trash” sump pump that has approximately one-half the capacity of the primary pumps. The pump shall be capable of handling four-inch solids.

14.3.4.5 ADOT Freeway Design Practice

Use a long shaft vertical turbine. The pump assembly consists of a bowl assembly with impeller, a long shaft, and a pipe column with an attached elbow. The long shaft will require lubrication at each bearing. This is to be provided by an automatic system that provides lubricant at each start.

14.3.5 Power

14.3.5.1 General

Several types of power may be available for a pump station. Examples are electric motors, and diesel or natural gas engines. The maintenance engineer should be contacted for input in the selection process. The designer should select the type of power that best meets the needs of the project based on an estimate of future energy considerations and overall station reliability. A comparative cost analysis of alternatives is helpful in making this decision. The need for backup power is dependent upon the consequences of failure. The decision to provide it should be based on economics and safety.

Electric Motors - Back-up/Standby Power

For electric motors, two independent electrical feeds from the electric utility with an automatic transfer switch may be the cost-effective choice when backup power is required. A standby generator is generally less cost-effective because of its initial costs. Also, standby generators require considerable maintenance and testing to ensure operation in times of need.

For extensive depressed freeway systems involving a number of electric motor-driven stations, a mobile generator may be the cost-effective choice for backup power. A trailer-mounted generator can be stored at any one of the pump stations. If a power outage occurs, maintenance forces can move the generator to the affected station to provide temporary power. If a mobile generator is used as the source of backup power, it may be necessary to add additional storage to compensate for the time lag that results in moving the generator from site to site. This lag can typically be 1.0 to 1.5 hours from the time the maintenance forces are notified.

14.3 Design Considerations (continued)

14.3.5.1 General (continued)

Consideration should be given to whether the pump station is to have standby power (SBP). If it is preferred that stations have a SBP receptacle, manual transfer switch and a portable engine/generator set, then the practical power limit of the pumps becomes 55 kW (75 HP) since this is the limit of the power generating capabilities of most portable generator units.

14.3.5.2 ADOT Freeway Design Practice

Diesel or natural gas engines driving an electric generator are preferred. The standard engine is non-turbocharged, water-cooled, operating at 1200 rpm. Engine exhaust must be equipped with mufflers and be directed in a way to reduce noise impacts to residences and/or businesses. A minimum clearance of 4 feet around all pumps, engines, and other mechanical equipment shall be provided in the station design.

14.3.6 Monitoring and Control

14.3.6.1 General

Pump stations are vulnerable to a wide range of operational problems from malfunction of the equipment to loss of power. Monitoring systems such as on-site warning lights and remote alarms can help minimize such failures and their consequences.

Telemetering is an option that should be considered for monitoring critical pump stations. Operating functions may be telemetered from the station to a central control unit. This allows the central control unit to initiate corrective actions immediately if a malfunction occurs. Such functions as power, pump operations, unauthorized entry, explosive fumes and high water levels can be monitored effectively in this manner. Perhaps the best overall procedure to assure the proper functioning of a pump station is the implementation of a regular schedule of maintenance conducted by trained, experienced personnel.

Instruments such as hour meters and number-of-starts meters should be used on each pump to help schedule maintenance. Input from maintenance forces should be a continuous process so that each new generation of stations will be an improvement.

Water-Level Sensors

The water-level sensors activate the pumps and, therefore, are a vital component of the control system. There are a number of different types of sensors that can be used. Types include the float switch, electronic probes, ultrasonic devices, mercury switch and air pressure switch. The location or setting of these sensors control the starting and stopping of the pump motors. Their function is critical because the pump motors or engines must not start more frequently than an allowable number of times per hour (i.e., the minimum cycle time) to avoid damage. To prolong the life of the motors, sufficient volume must be provided between the pump start and stop elevations to meet the minimum cycle time requirement. The on-off setting for the first pump is particularly important because it defines the most frequently used cycle.

14.3 Design Considerations (continued)

14.3.6.2 ADOT Freeway Design Practice

Control System
A control room separate from the wet well or engine/pump room shall contain the control equipment. The control room is completely isolated from both the wet well and the engine room. It is separately air-conditioned. The room shall be temperature controlled. Should the station electric power fail, the stand-by generator will be activated, and the generator will continue to drive the control room air-conditioner, as a well as the other ventilation systems.

The sump pump will always be the first pump on and the last pump off. The four primary pumps will be consecutively lagged as the water level rises in the wet well. The order of lagging shall be by manual selection.

Uninterruptible Power Supply (UPS)
UPS shall be provided to furnish power for controlling and monitoring the pump station. Batteries sized for one-half hour outage at full capacity will supply power to the UPS. A signal from the UPS is provided to start a stand-by engine generator set when there is a power outage. An adjustable time will delay start of the standby generator. The UPS will be equipped with a static transfer switch, a manual maintenance switch, metering, and alarms and status lights.

Emergency/Stand-by Power Supply
A diesel engine-generator set is provided to provide all electrical station power and air-conditioning in the event of commercial power outage. The stand-by generator also is used to recharge the UPS system. The stand-by engine/generator set will start on a signal from the UPS after a set delay. Power is provided to the UPS via an automatic transfer switch.

Sensors and Monitors
For each engine driven pump and the sump pump a level-sensing probe located in the wet well will provide signals to start and stop. Pilot lights on a control panel will indicate the status of each pump. Any pump running will also actuate external signals to signify the station is operating. Sensors are attached at each of the discharge line flap gates to determine if the pump station is discharging. A sensor shall be provided in the discharge box to measure the head at the weir. The head shall be converted to discharge. Means will be provided to transmit the pump on/off and discharge status to a central location.

Probes will be provided for low water and high water conditions. Either of these conditions will activate local and remote indicators. These two alarms will only be capable of being reset by manual means at the pump station control panel.

14.3 Design Considerations (continued)

14.3.6.2 ADOT Freeway Design Practice (continued)

Communications Link

A reliable means of communicating the status of the following conditions is to be provided:

1.) Pump station discharge
2.) High level in wet well
3.) Pump failure
4.) Pump status (on/off)
5.) Sump pump (overload, on/off)
6.) Engine failure
7.) Engine status (on/off)
8.) Fuel level in tanks (level and low-level alarm)
9.) Control room power failure
10.) Utility power failure
11.) Uninterruptible Power Supply (failure, low battery)
12.) Stand-by generator (malfunction, on/off)
13.) Engine room and wet well high temperature alarm
14.) Fans in wet well, stairway, engine room (overload, on/off)
15.) Combustible gas (wet well and/or engine room)
16.) Fire
17.) Low foam liquid level (indicates fire suppression system activated)
18.) Security violated

14.3 Design Considerations (continued)

14.3.7 Collection System

14.3.7.1 General

Storm drains leading to the pumping station are usually designed on a flat grade to minimize depth and cost. A minimum grade that produces a velocity of 3 ft/sec in the pipe while flowing at a depth of one-quarter (1/4) full is suggested to avoid siltation problems in the collection system. Minimum cover or local head requirements will usually govern the depth of the uppermost inlets. Storm drainage systems tributary to the pump station can be quite extensive and costly. Linear or intermediate storage along the storm drain may be used to reduce peak flows and pipe sizes. For some pump stations, the storage available in the collection system may be significant. However, it is often necessary to provide additional storage near the pump station. This may be done by oversizing the collection system or designing an underground vault.

The collector lines should preferably terminate at a forebay or storage box. However, they may discharge directly into the station. Under the latter condition, the capacity of the collectors and the storage within them is critical to providing adequate cycling time for the pumps and must be carefully calculated. The inlet pipe should enter the station perpendicular to the line of pumps. It should be aligned with the centerline of the wet well and should have a straight run of at least 100 feet before entering the station. The inflow should distribute itself equally to all pumps. Baffles may be required to ensure that this is achieved.

Grate inlets and/or slotted drain inlets are recommended for lines that connect to pump stations. They will act as screens to prevent large objects from entering the system and possibly damaging the pumps. This approach has an additional advantage of simplifying debris removal since debris can be more easily removed from the roadway than the wet well.

14.3.7.2 Trash Racks And Grit Chambers

Trash racks should be provided at the entrance to the wet well if large debris is anticipated. For stormwater pumping stations, simple steel bar screens are adequate. Usually, the bar screens are inclined with bar spacing approximately 1.5 in. Constructing the screens in modules facilitate removal for maintenance. An emergency overflow should be provided to protect against clogging and subsequent surcharging of the collection system.

14.3 Design Considerations (continued)

14.3.8 Discharge System

14.3.8.1 General

The discharge piping should be kept as simple as possible. Pumping systems that lift the stormwater vertically and discharge it through individual lines to a gravity storm drain as quickly as possible are preferred. The pump location with respect to the outfall chamber should be set to provide as short a distance as possible. Individual pump discharge lines are the most cost-effective system for short outfall lengths. Individual lines may exit the pumping station either above or below grade. Damaging pump reversal could occur with very long force mains. Check valves should be installed. The effect of stormwater returning to the sump after pumping stops should be considered.

Gate valves should be provided in each pump discharge line to provide for continued operation during periods of repair, etc. A cost analysis should be performed to determine what length and type of discharge piping justifies a manifold. Number of valves required shall be kept to a minimum to reduce cost, maintenance and headloss through the system.

It may be necessary to pump to a higher elevation using long discharge lines. This may dictate that the individual lines be combined into a force main via a manifold. For such cases, check valves must be provided on the individual lines to keep stormwater from running back into the wet well and restarting the pumps or prolonging their operation time. Check valves should preferably be located on horizontal layouts rather than vertical, to prevent sedimentation on the downstream side after the valve closing.

The discharge line is typically either steel or cast iron and must be sized to be at least the size of the pump discharge diameter. The need for a larger pipe required to limit the maximum velocity to 10 ft/sec should be checked by the following equation.

$$D= 1.128(Q/V)^{\mathbf{0.5}}$$

14.3.8.2 Flap Gates And Valving

Flap Gates - The purpose of a flap gate is to restrict water from flowing back into the discharge pipe and to discourage entry into the outfall line. Flap gates are usually not watertight so the elevation of the discharge pipe should be set above the normal water levels in the receiving channel. If flap gates are used, it may not be necessary to provide for check valves.

Check Values - Check valves are watertight and are required to prevent backflow on force mains that contain sufficient water to restart the pumps. They also effectively stop backflow from reversing the direction of pump and motor rotation. They must be used on manifolds to prevent return flow from perpetuating pump operation. Check valves should be "non-slam" to prevent water hammer. Types include: swing, ball, dashpot and electric.

Gate Valves – A gate valve is used as a shut-off device on force mains to allow for pump or valve removal. These valves should not be used to throttle flow. They should be either totally open or totally closed.

14.3 Design Considerations (continued)

14.3.8.2 Flap Gates And Valving (continued)

Air/Vacuum Valves - Air/Vacuum valves are used to allow air to escape the discharge piping when pumping begins and to prevent vacuum damage to the discharge piping when pumping stops. They are especially important with large diameter pipe. If the pump discharge is open to the atmosphere, an air-vacuum release valve is not necessary. Combination air release valves are used at high points in force mains to evacuate trapped air.

14.3.8.3 ADOT Freeway Design Practice

The pump discharge line is connected to a discharge box. The box drains to the discharge outfall or storm drain. The discharge box is divided by a low wall that performs as a weir. The wall also creates a chamber that provides storage for pump recirculation operation. The chamber for re-circulation is sized for testing and exercising one pump/engine set or the sump pump. The wall is beveled to form a sharp crested weir.

Two manually operated sluice gates are provided; one between the recirculation chamber and the outlet chamber and one at the recirculation pipe to prevent backflow during normal operations. A flap gate, usually of cast iron, is provided for each pump discharge line.

14.3.9 Safety

14.3.9.1 General

All elements of the pump station should be carefully reviewed for safety of operation and maintenance. Ladders, stairwells and other access points should facilitate use by maintenance personnel. Adequate space should be provided for the operation and maintenance of all equipment. Particular attention should be given to guarding moving components such as drive shafts and providing proper and reliable lighting. It may also be prudent to provide air-testing equipment in the station so maintenance personnel can be assured of clean air before entering.

Pump stations may be classified as a confined space in which case access requirements along with any safety equipment are all defined by code. Pump stations should be designed to be secure from entry by unauthorized personnel and as few windows as possible should be provided.

The designer should be aware of established safety procedures for working in pump stations. The designs shall meet the established safety criteria and enhance the ease of compliance

14.3.9.2 Internal Environment

Ventilation

Ventilation of dry and wet wells is necessary to ensure a safe working environment for maintenance personnel. The ventilation system can be activated by a combined light/ventilation switch at the entrance to the station. Maintenance procedures normally require personnel to wait several minutes after ventilation has started before entering the well. The testing of the air in the wet well prior to allowing entry may be

14.3 Design Considerations (continued)

14.3.9.2 Internal Environment (continued)

required. If mechanical ventilation is required to prevent buildup of potentially explosive gasses, the pump motors or any spark producing equipment should be rated explosion proof or the fans run continuously.

Heating and dehumidifying requirements are variable. Their use is primarily dependent upon equipment and station type, environmental conditions and station use.

14.3.9.3 Hazardous Spills

The possibility of hazardous spills is always present under highway conditions. In particular, this has reference to gasoline, and the vulnerability of pump stations and pumping equipment to fire damage. There is a history of such incidents having occurred and also of spills of oils, corrosive chemicals, pesticides and the like having been flushed into stations, with undesirable results. The usual design practice has been to provide a closed conduit system leading directly from the highway to the pump station without any open forebay to intercept hazardous fluids, or vent off volatile gases. With a closed system, there must be a gas-tight seal between the pump pit and the motor room in the pump station. Preferably, the pump station should be isolated from the main collection system and the effect of hazardous spills by a properly designed storage facility upstream of the station. This may be an open forebay or a closed box below the highway pavement or adjacent to it. The closed box must be ventilated by sufficient grating area at each end.

14.3.9.4 ADOT Freeway Design Practice

Combustible Gas Detection
Combustible gas detection systems shall be provided for the wet well and engine room. The systems shall include self-contained gas monitors and remote detectors. The systems shall be powered by the UPS. The systems shall be capable of responding to at least two levels of the presence of combustible gases. The lower level shall send signals to the communication system, actuate local signals, and activate the ventilation system. The high level sensor shall shut-off any engines that are running and activate the foam fire suppression system.

Fire Suppression System
The foam fire suppression system shall be capable of three 10-minute applications.

Ventilation System
The station exhaust fan ventilation system is composed of three separate areas. One each is provided for the wet well, engine room and stairway. The wet well system has a push/pull intake and exhaust system shall provide for 12 air changes per hour. As noted above, this system is activated by either manually or by gas detectors in the wet well.

The engine room has two exhaust fans that are thermostatically controlled. They are set to turn on when the room temperature exceeds 90 and 95 degrees Fahrenheit and the room temperature is higher than the outside ambient temperature. As noted above, this system is activated by either manually or by gas detectors in the wet well or engine room.

14.3 Design Considerations (continued)

14.3.9.4 ADOT Freeway Design Practice (continued)

The stairway has push/pull intake and exhaust systems that exhaust air from four levels. The stairwell light switch activates the stairwell ventilation system. The stairwell is equipped with fire doors at the top and bottom entrances. Each exhaust duct is equipped with a fire damper that closes automatically when smoke is detected in the wet well or engine room.

14.3.10 Building Architecture

14.3.10.1 General

The appearance of the pump station should be compatible with the surrounding area. The site layout should provide a secure, easily monitored structure. Applicable building codes shall be followed. The internal areas of the pump station are divided into the wet well, the engine/pump room, the control room, and an enclosed stairwell that provides access to the wet well. Items of particular concern regarding building architecture are the fire protection, security, and air-conditioning and ventilation systems.

14.3.10.2 Building Access

Separate access to the building will be provided for personnel into the engine/pump room, the control room, and the stairwell. In addition to the personnel access doors, access for pick-up or other light to medium sized vehicles shall be provided by an 8 ft by 8 ft. roll up door into the engine/pump room and through double door into the control room. A roof access hatch over each engine/pump shall be provided.

14.3.11 Site Design

Site location and layout is important to achieve a station that is secure and compatible with the surrounding area.

14.3.11.1 Access

The site should be accessed from a frontage road or a local street. It should not require access from a limited access highway.

14.3.11.2 Security

The area is to be well lighted. An eight-foot high wall shall surround the station with access only through a 24-foot locked gate. The access gate shall be situated so as to allow maximum visibility of as many accessible entrances as possible. If this is not possible, provide an open-weave fence section in the wall to allow observation of doors not visible from the access gate. All accessible exterior doors shall be equipped with proximity sensors that will activate local and remote alarms.

14.3 Design Considerations (continued)

14.3.11.3 Fuel Storage

Generally the fuel storage tanks are located along the block wall. The location must be accessible for refueling and maintenance, but out of the way for other operations. Diesel fuel storage tanks are to have 24-hour capacity. Storage tanks may be in-ground tanks located in a vault or above ground steel-lined concrete. Vaults are to be provided with a roof deck to block solar radiation and water intrusion. The vault will provide for containment of 110 percent of the volume of the diesel tanks. The vault will not have a drain; any accumulation of liquid must be pumped out with a portable pump. Above ground concrete tanks must be fire rated and ballistic proof.

14.3.11.4 Utilities

Required utilities are electricity and water. Sanitary sewers are not required. The minimum electric service is 480 volt, 3-phase, four wire. The minimum size of water main is 6-inch diameter. A fire hydrant should be provided near the pump station and a minimum 4-inch service connection with back- flow preventor should be provided to the pump station building.

14.3.11.5 Drainage

Adequate provisions for site drainage shall be provided. Local drainage can be connected to the discharge box or outlet pipe. On-site drainage should NOT be allowed to enter the pump station and require pumping.

14.4 Design Concepts and Criteria

The following discussion is made with the objective of minimizing the construction, operation and maintenance costs of highway stormwater pump stations while remaining consistent with the practical limitations of all aspects.

14.4.1 Discharge Head And System Curve

14.4.1.1 Headloss

Stormwater pumps are extremely sensitive to changes in head requiring the head demand on the pumps be calculated as accurately as possible. All valve and bend losses are to be considered in the computations. In selecting the size of discharge piping, consideration should be given to the manufactured pump outlet size vs. the head loss produced by smaller piping. This approach should identify a reasonable compromise in balancing cost. Once the head losses have been calculated for the range of discharges expected, the system curve (Q vs. TDH) can be plotted. This curve defines the energy required to pump any flow through the discharge system. It is especially critical for the analysis of a discharge system with a forcemain. When overlaid with pump performance curves (provided by manufacturer), it will yield the pump operating points (see Figure 14-5).

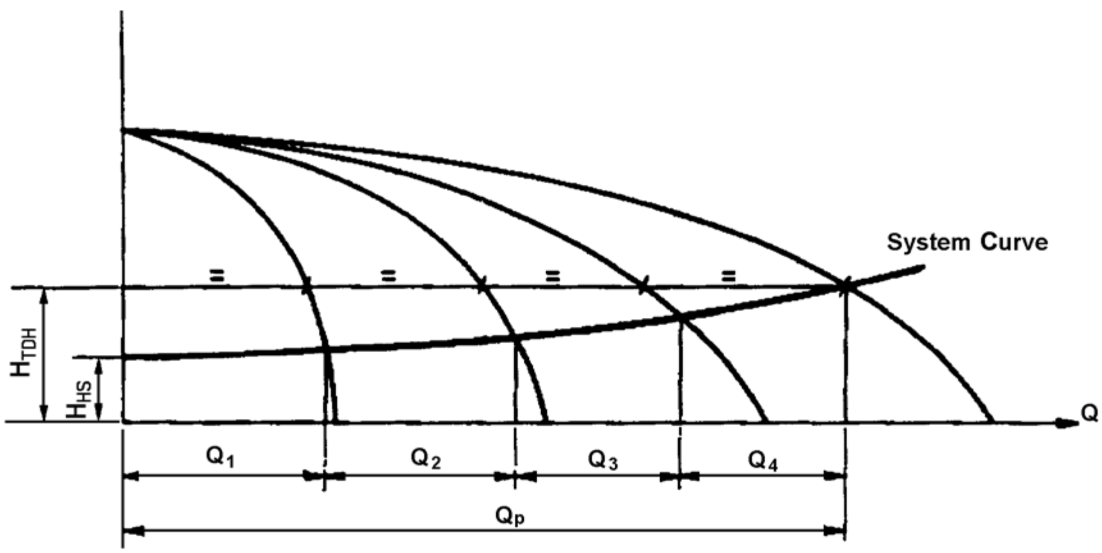


Figure 14-5 Systems Curve

The combination of static head, velocity head and various head losses in the discharge system due to friction is called total dynamic head. These head losses are minimized by the selection of correctly sized discharge lines and other components.

Losses in pipe system:	K
Elbow, 90 degree	0.20
Miscellaneous couplings and fittings	0.05
Flap Gate	0.25
Outlet	1.00

14.4 Design Concepts and Criteria (continued)

14.4.1.2 Total Dynamic Head

Pumps for a given station are selected to all operate together to deliver the Design Q at a Total Dynamic Head computed to correspond with the Design Water Level. Because pumps must operate over a range of water levels, the quantity delivered will vary significantly between the low level of the range and the high level.

A curve of total dynamic head versus pump capacity is available for each pump from the manufacturer. When running, the pump will respond to the total dynamic head prevailing and the quantity of discharge will be in accordance with the curve. The designer must study the pump performance curves for various pumps in order to develop an understanding of the pumping conditions (head, discharge, efficiency, kilowatt, etc.) throughout the full range of head that the pump will operate under. The system specified must operate properly under the full range of specified head.

When the pump is raising the water from the lowest level, the static head will be greatest and the discharge will be the least. When operating at the highest level, the static head will be the least and the discharge will be the greatest. Pump capabilities must always be expressed in both quantity of discharge and the total dynamic head at a given level. Typically, these conditions are specified for three points on the performance curve. One point will be near maximum TDH, the next will be the design point, and the third will be at about minimum TDH.

A pump is selected to operate with the best possible efficiency as its Design Point, corresponding to the Design Water Level of the station, and its performance is expressed as the required discharge at the resulting total dynamic head. The efficiency of a stormwater pump at its design point may be 75 or 80% or more, but this will depend on the type of pump. When the static lift is greatest (low water in sump), the energy required (horsepower) maybe the greatest even though the quantity of water raised is less. This is because the pump efficiency may also be much less. The pump selection should be made so that maximum efficiency is at the design point.

The total dynamic head (TDH) should be determined for a sufficient number of points to draw the system head curve that will be discussed later in this chapter. The TDH is computed as follows:

$$\mathbf{TDH = H_s + H_f + H_v + H_l} \qquad \mathbf{(14.1)}$$

where: TDH = total dynamic head, ft
H_s = max. static head (at lowest pump-off elevation), ft
H_f = friction head, ft (i.e., friction loss)
H_v = velocity head, ft ($V^2/2g$)
H_l = losses through fittings, valves, etc., ft

Adjustments may have to be made to these curves to account for losses within the pumping unit provided by the manufacturer.

14.4 Design Concepts and Criteria (continued)

14.4.2 Pumps

14.4.2.1 Main Pumps

Number And Capacity

The number of pumps needed are determined by following a systematic process defined in section 14.6. However, two to three pumps have been judged to be the recommended minimum. If the total discharge to be pumped is small and the area draining to the station has little chance of increasing substantially, the use of two pumps in one station is preferred. Consideration may be given to over sizing the pumps to compensate, in part, for a pump failure. A two-pump system could have pumps designed to pump 66 - 100% of the required discharge and the three pump system could be designed so that each pump will pump 50% of the design flow. The possible impacts caused by the loss of one pump can be used as a basis for deciding the size and numbers of the pumps.

Economic limitations on power unit size as well as practical limitations governing operation and maintenance should be used to determine the upper limit of pump size. The minimum number of pumps used may increase due to these limitations.

Equal-size pumps are to be used. Identical size and type enables all pumps to be freely alternated into service. This equalizes wear and reduces needed cycling storage. It also simplifies scheduling maintenance and allows pump parts to be interchangeable. Hour meters and start meters are to be provided, these aid in scheduling needed maintenance.

Final Selection

For the typical highway application, any of the three pump types described earlier will usually suffice. If not, manufacturers' information will likely dictate the type required. However, knowing the operating RPMs, a computation for specific speed can be made to check the appropriateness of the pump type (see Figure 14-6 to determine the ranges where specific impeller types should be used). Suction specific speed may be defined as that speed in revolutions per minute at which a given impeller would operate if reduced proportionally in size so as to deliver a capacity of 3.3 gpm against TDH of 3 ft. This is an index number descriptive of the suction characteristics of a given pump. Higher numerical values are associated with better NPSH capabilities. This number should be checked for dry pit applications and systems with suction lifts. Once the pump type and capacity have been determined, the final selection of the pump can be made.

14.4 Design Concepts and Criteria (continued)

14.4.2.1 Main Pumps (continued)

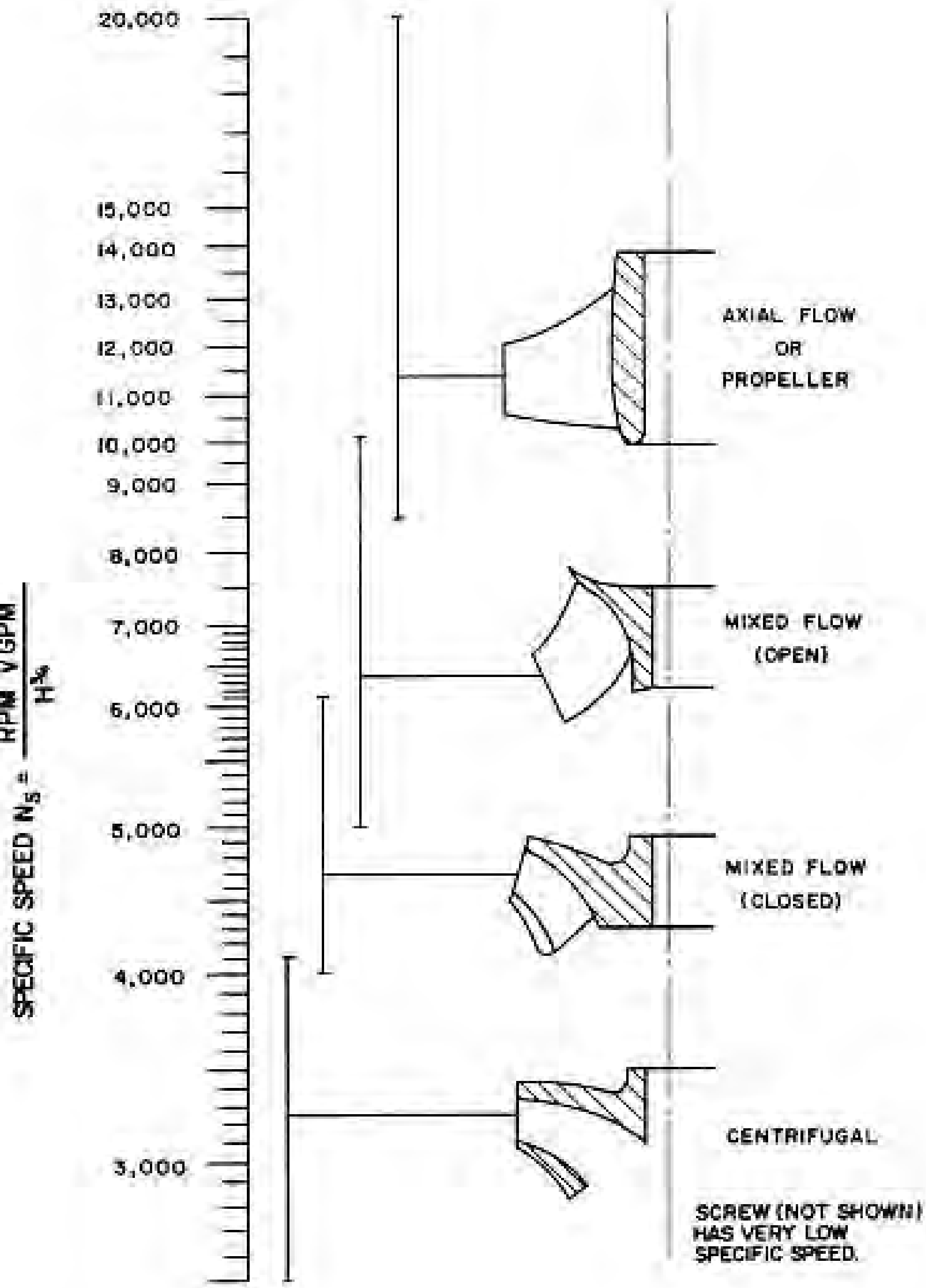


Figure 14-6 Specific Pump Speed vs. Impeller Types

14.4 Design Concepts and Criteria (continued)

14.4.3 Wet Well Design

The primary criteria for sizing the wet well involves the number of pumps and pump bell diameter. This criteria includes floor clearance, minimum distance between an inlet bell and the wall, the minimum clearance between adjacent inlet bells, width of partition wall between pumps, and the submergence required for the pump bell diameter. The specific criteria for both circular and rectangular wet well can be found in "American National Standard for Pump Station Intake Design" by the Hydraulic Institute as well as from pump manufacturers.

The wet well size and shape are important factors for both their contribution to available storage and for providing room fro proper sizing and layout of pumps. However, the final number of pumps is not normally known until the final design phase. Therefore, it is necessary to estimate wet well dimensions based on a trial number and size of pumps. It may be necessary to increase dimensions to provide additional storage or to accommodate additional pumps.

14.4.3.1 Cycling Sequence And Volumes

Cycling is the starting and stopping of pumps, the frequency of which must be limited to prevent damage and possible malfunction. Sufficient volume must be provided either in the wet well or outside the wet well for safe cycling. The volume required to satisfy the minimum cycle time is dependent upon the characteristics of the power unit, the number and capacity of pumps, the sequential order in which the pumps operate and whether or not the pumps are alternated during operation. However, to keep sediment in suspension, the wet well volume should not be oversized

Before discussing pump cycling considerations, operation of a pump station will be described. Initially, the water level in the storage basin will rise at a rate depending on the rate of the inflow and physical geometry of the storage basin. When the water level reaches the stage designated as the first pump start elevation, the pump will be activated and discharge water from storage at its designated pumping rate. For pumps that are engine driven, there will be a delay between the initial start signal and the commencement of pumping. This delay may be on the order of 90 seconds. The volume of inflow that will occur during this delay must be taken into consideration in the design of the storage volume. If the pumping rate exceeds the rate of inflow, the water level will drop until it reaches the first pump stop elevation. With the pump stopped, the basin begins to refill and the cycle is repeated. This scenario illustrates that the cycling time will be lengthened by increasing the amount of storage between pump on and off elevations. This volume of storage between first pump on and off elevations is termed usable volume. In theory, the minimum cycle time allowable to reduce wear on the pumps will occur when the inflow to the usable storage volume is one-half the pump capacity.

There are two basic cycling sequences. One is referred to as the "common off elevation." In this sequence, the pumps start at successively higher elevations as required; however, they all stop at the same off elevation. This is advantageous when large amounts of sediment are anticipated. The other sequence uses a "successive start/stop" arrangement in which the start elevation for one pump is also the stop elevation for the subsequent pump, i.e., the start elevation for pump 1 is the stop elevation for pump 2, the start elevation for pump 2 is the stop elevation for pump 3, etc. (see Figure 14-7). There are countless variations between these two sequences.

14.4 Design Concepts and Criteria (continued)

14.4.3.1 Cycling Sequence And Volumes (continued)

There are also different pump start alternation techniques that reduce the cycling volume requirement and equalize wear on the pumps. They range from simply alternating the first pump to start, to continuously alternating all pumps during operation, a technique referred to as "cyclical running alternation". Using this technique, each pump is stopped in the same order in which it starts, i.e., the first pump to start will be the first pump to stop, etc. (see Figure 14-8).

Alternating the first pump to start is sufficient for stormwater pump stations where more than one pump on will be rare and of short duration. This alternation technique coupled with the successive start/stop cycling sequence requires the smallest total cycling volume possible (see Figure 14-9). This total volume is computed as follows:

$$\mathbf{V_t = Q_p\, t_c/4N} \qquad \textbf{(14.2)}$$

where: V_t = total cycling storage volume, ft^3
Q_p = total capacity of all pumps, ft^3/sec
t_c = minimum allowable cycle time, sec (= 3600/max. starts per hour)
N = total number of equal-size pumps

The proof is as follows:
t = Time between starts
t = Time to Empty + Time to Fill usable storage volume V_t

$$\mathbf{t = V_t/(Q_p\text{-}Q_i) + V_t/Q_i} \qquad \textbf{When } \mathbf{Q_i = Q_p/2,\ ft^3/sec,}$$

$$\mathbf{t = 4V_t/Q_p,\ s} \qquad \textbf{(14.4)}$$

or for t in minutes $t = 4V_t/60Q_p = V_t/15Q_p$

Pump Alteration Sequence Effect

Let us take an example when four pumps are installed in the same station with pumps starting in sequence and stopping in reverse order.

- By designing the control system for pump alternation, sump volumes will be reduced as well as distribute the pump operating time more evenly between the four pumps.
- If the inflow is less than the capacity of one pump, pump number one would, without alternation, do all the work.
- With alternation pump number one starts and draws down. Next start would call pump number two, etc.
- This means that with four pumps of the same size and operating in an alternating sequence each pump is called on to pump down sump volume V_1 every fourth time. The cycle time of each pump will be four times longer than the cycle time of filling and emptying of V_1.

14.4 Design Concepts and Criteria (continued)

14.4.3.1 Cycling Sequence And Volumes (continued)

- The volume required for each pump will vary, depending upon the characteristics of the discharge system. It should be noted that with these volumes, the minimum allowable cycle time will only be experienced when the proportionate inflow to each pump is exactly one-half the capacity of that pump. All other inflows will produce a cycle time longer than the minimum.
- This system works for any number of pumps in a station.

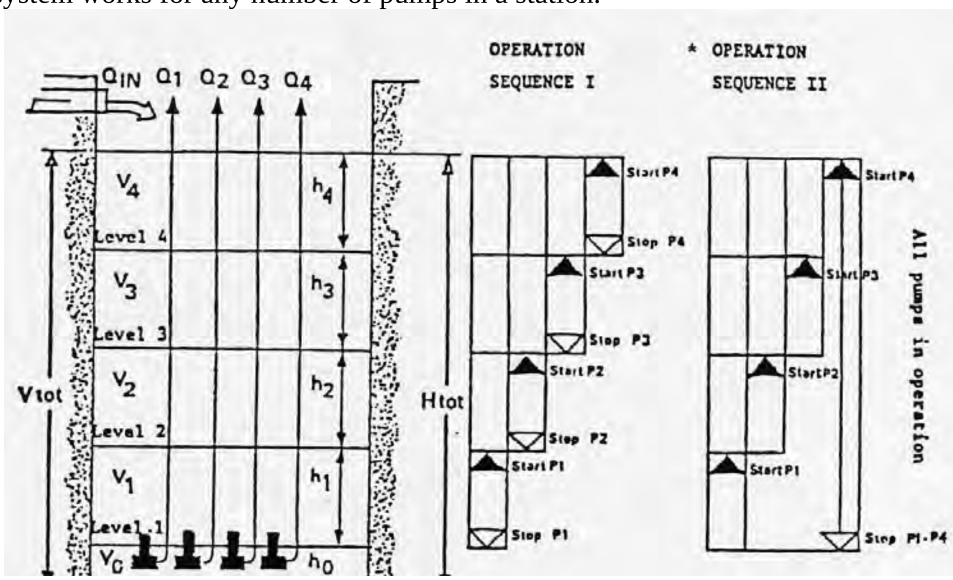


Figure 14-7 Pump Sequences

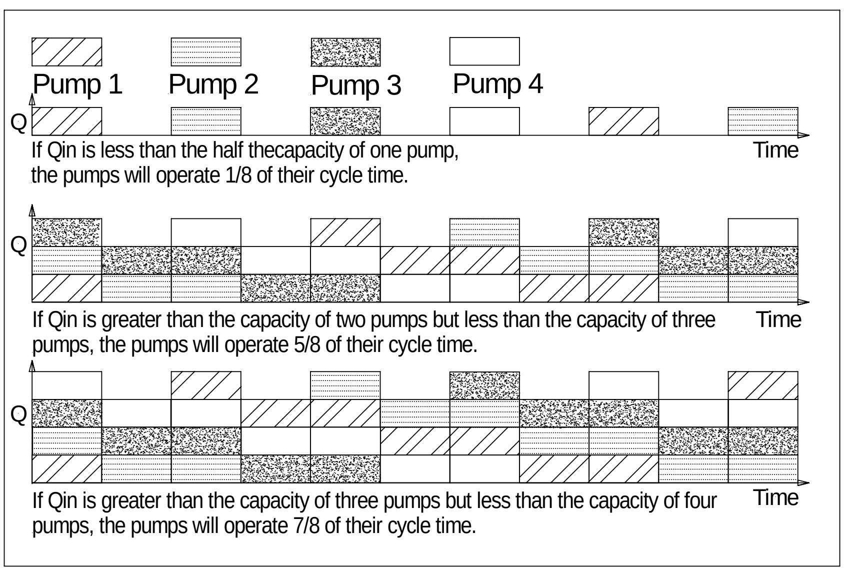
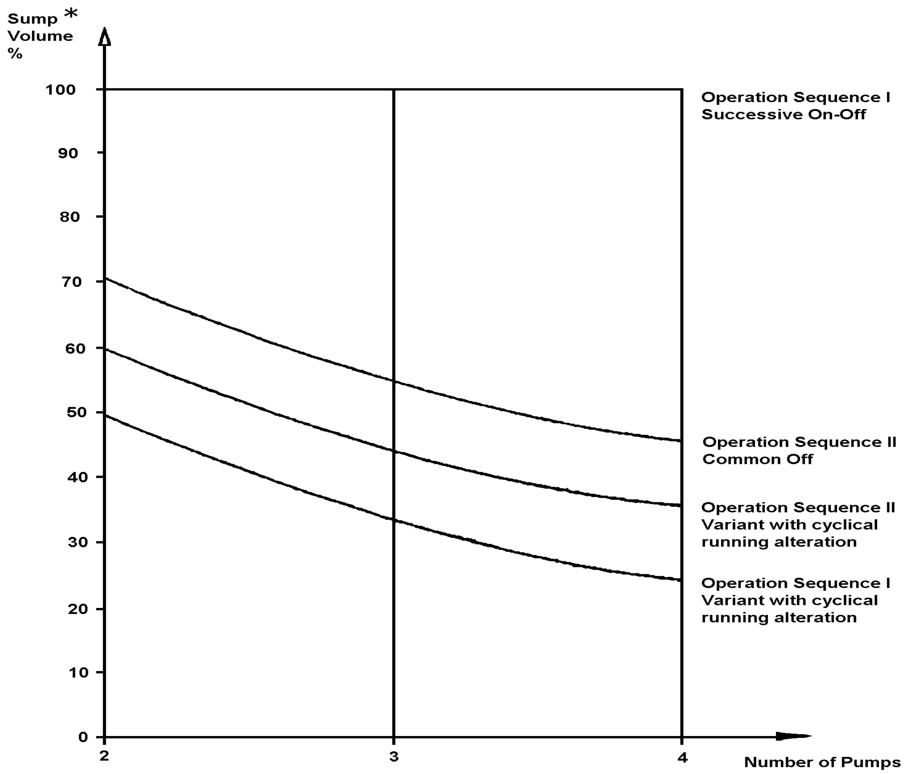


Figure 14-8 Schematic of Pump Sequences at Different Inflow Rates Pumps With Cyclical Running Alternation -- A Variant Of Operation Sequence I in Figure 14-7

14.4 Design Concepts and Criteria (continued)

14.4.3.1 Cycling Sequence And Volumes (continued)



100%

corresponds to the volume that is derived from the formula $V_t = (Q_p T_{min})/4N$

Figure 14-9 Comparison Of The Pump Volume with and without Cyclical Running Alternation

Lowest Pump "Off" Elevation

It is recommended that the lowest pump "off" elevation be located at or within 1 foot below the inlet invert elevation unless plan dimension constraints dictate that the station floor be lowered to obtain the necessary cycling volume. This recommendation is based on the fact that it is usually less expensive to expand a station's plan dimensions than to increase its depth. This elevation represents the static pumping head to be used for pumping selection

Pump "On and Off" Elevations

These should be set at the elevations, which satisfy the individual pump cycling volumes (V_x). Starting the pumps as soon as possible by incrementing these volumes successively above the lowest pump off elevation will maximize what storage is available within the wet well and the collection system. The depth required for each volume is computed as follows:

$$\mathbf{H_x = V_x/plan\ area} \qquad \mathbf{(14.3)}$$

14.4 Design Concepts and Criteria (continued)

14.4.3.1 Cycling Sequence And Volumes (continued)

Pumping Range

It is recommended that the minimum distance between the "on" and "off" elevation of an individual pump be 6 inches. If it is less than 6 inches, then reduce the plan area. Though rare, this could require a reduction in the number of pumps, an increase in station depth, or both.

Allowable High Water Elevation

The allowable high water (AHW) elevation in the station should be set such that the water surface elevation at the lowest inlet in the collection system provides 0.5 feet of freeboard below the roadway grate.

Clearances

Pump to pump, pump to back wall, and pump to sidewall clearances should be the minimum possible to minimize the potential for sedimentation problems. Consult manufacturer literature or a dimensioning guide. The pump inlet to floor clearance plus the pump submergence requirement constitutes the distance from the lowest pump "off" elevation to the wet well floor. The final elevation may have to be adjusted as the type of pump to be installed is finalized.

Intake System Design

The primary function of the intake structure is to supply an even distribution of flow to the pumps. An uneven distribution may cause strong local currents resulting in reduced pump efficiency and undesirable operational characteristics. The ideal approach is a straight channel coming directly into the pump or suction pipe. Turns and obstructions are detrimental, since they may cause eddy currents and tend to initiate deep-cored vortices. The inflow should be perpendicular to a line of pumps and water should not flow past one pump to get to another. Unusual circumstances will require a unique design of the intake structure to provide proper flow to the pumps.

ADOT Freeway Design Practice

Pump on/off elevations for the primary pumps are determined during final design of the station considering the pump characteristic minimum cycle time, and total storage available (both underground plus wet well). The recommended minimum pump cycle time is 20 minutes.

The sump pump is set to turn on two feet below the level of the intake pipe invert and turn off at the wet well floor elevation. The first main pump should turn on at a slightly higher level than the sump pump and both pumps will be on for a portion of the total range. When the wet well level backs into the intake/storage pipes, the volume of storage in the sloping pipe is calculated as described in Appendix A for an "ungula".

14.4 Design Concepts and Criteria (continued)

14.4.4 Inflow-Outflow Routing

14.4.4.1 Storage

The development of the wet-well design as discussed in Section 14.4.3 has general application when it is anticipated that most of the peak flow will be pumped. In that case, pump run time and cycling sequences are of great importance. In the case of many of the highway storm drain situations, it has been the practice to store substantial parts of the flow in order to minimize pumping requirements as well as outflow piping. The demand on the pumping system is different and thus additional considerations need to be made.

The total storage capacity that can or should be provided is an important initial consideration in pump station design. The designer should recognize that a balance should be reached between pump rate and storage volume. This will require a trial and error procedure used in conjunction with an economic analysis. Using the hydrograph and pump-system curves, various levels of pump capacity can be tried and the corresponding required total storage can be determined. The basic principle is that the volume of water as represented by the shaded area of the hydrograph in Figure 14-10 is beyond the capacity of the pumps and must be stored. If a larger part or most of the design storm is allowed to collect in a storage facility, a much smaller pump station can be utilized, with anticipated cost benefits. The principles discussed for minimum run time, pump cycling, etc. in the design of wet wells should also be considered in the case of larger storage volume development. However, it will be noted that differences exist as the volume of storage become larger. Typically, the concern for meeting minimum run times and cycling time will be reduced because the volume of storage is sufficient to prevent these conditions from controlling the pump operation. The start and stop elevations will be of different magnitudes because of the volume represented by each increment of storage depth.

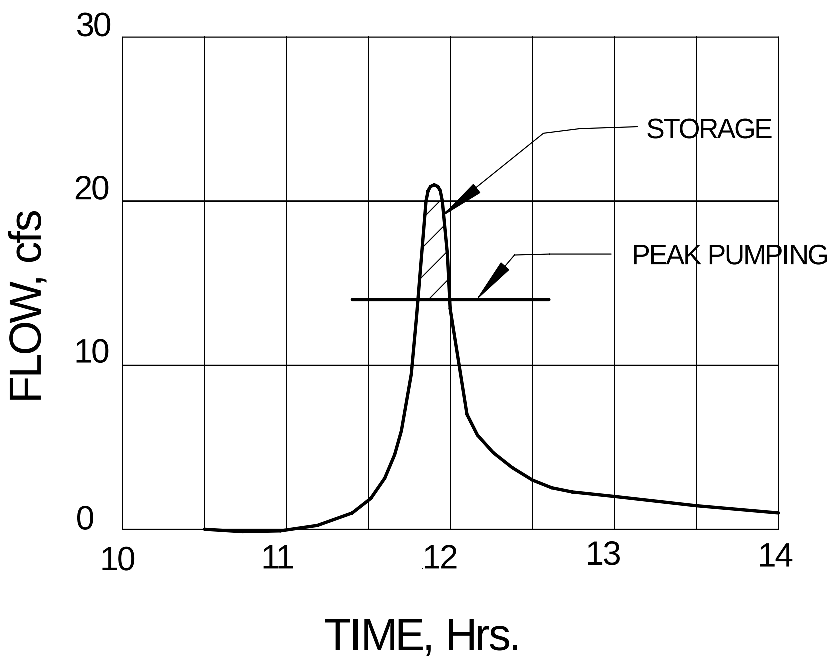


Figure 14-10 Estimating Required Storage

14.4 Design Concepts and Criteria (continued)

14.4.4 Inflow-Outflow Routing (continued)

14.4.4.2 Mass Curve Diagram

The approach used for the design of the pump station is that associated with the development of an inflow mass curve, figure 14-11. The mass inflow curve procedure is commonly used when significant storage is provided outside of the wet well. The plotting of the performance curve on the mass inflow diagram gives the designer a good graphical tool for determining storage requirements. The procedure also makes it easy to visualize pump start/stop and run times. In the event that a pump failure should occur, the designer can also evaluate the storage requirement and thus the flooding or inundation that could occur. The graphical procedure described below is approximate and labor intensive. Computer solutions of the method should be used for the analysis of pumping systems.

In this process, the designer uses an inflow hydrograph and a developed stage-storage relationship. Trial pumping systems will be applied to the inflow mass curve to develop a mass curve routing diagram. The inflow hydrograph is a fixed design component while the storage and pumping discharge rates are variable. The designer will select a design pumping discharge rate, this may be based on downstream capacity considerations. With the inflow mass curve and an assigned pumping rate, the required storage volume can be determined by various trials of the routing procedure.

As the stormwater flows into the storage basin, it will accumulate until the first pump start elevation is reached. The first pump is activated and if the inflow rate is greater than the pump rate, the stormwater will continue to accumulate until the second pump start elevation is reached. As the inflow rate decreases, the pumps will shut off at their respective pump stop elevations. These conditions are modeled in the mass curve diagram by establishing the point at which the cumulative flow curve has reached the storage volume associated with the first pump-start elevation. This storage volume is represented by the vertical distance between the cumulative flow curve and the base line. A vertical storage line is drawn at this point since it establishes the time at which the pump first starts. The pump discharge line is drawn from the intersection of the vertical storage line and the base line upwards toward the right; the slope of this line is equal to the discharge rate of the pump. The pump discharge curve represents the cumulative discharge from the storage basin, while the vertical distance between the inflow mass curve and the pump discharge curve represents the amount of stormwater stored in the basin.

If the rate of inflow is greater than the pump capacity, the inflow mass curve and the pump discharge curve will continue to diverge until the volume of water in storage is equal to the storage associated with the second pump-start elevation. At this point the second pump starts, and the slope of the pump discharge line is increased to equal the combined pumping rates. The procedure continues until peak storage conditions are reached. At some point on the inflow mass curve, the inflow rate will decrease, and the slope of the inflow mass curve will flatten. To determine the maximum amount of storage required, a line is drawn parallel to the pump discharge curve and tangent to the inflow mass curve. The vertical distance between the lines represents the maximum amount of storage required.

The routing procedure continues until the pump discharge curve intersects the inflow mass curve. At this point the storage basin has been completely emptied, and a pumping cycle has been completed. As the storm recedes, the pumps will cycle to discharge the remaining runoff.

14.4 Design Concepts and Criteria (continued)

14.4.4.2 Mass Curve Diagram (continued)

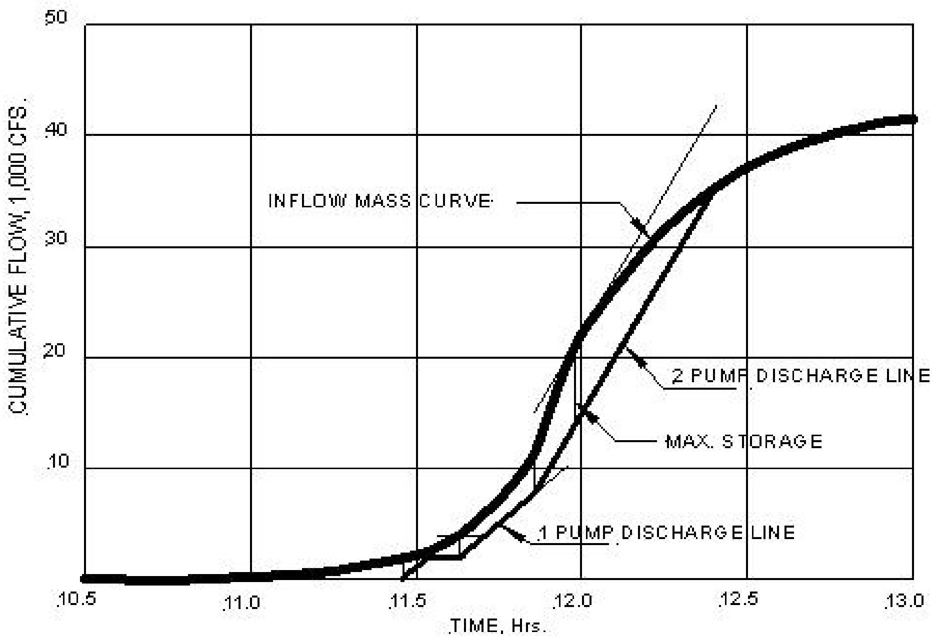


Figure 14-11 Mass Curve Diagram

In developing the pump discharge curve, the designer should remember that the pump's performance curve is quite sensitive to changes in head and that the static head will fluctuate as the water level in the storage basin fluctuates. The designer should also recognize that the pump discharge rate represents an average pumping rate.

14.5 Design Procedure

14.5.1 Introduction

The following is a systematic procedure that integrates the hydraulic design variables involved in sump design. It incorporates the above recommended design criteria and yields the required number and capacity of pumps as well as the wet well and storage dimensions. The final dimensions can be adjusted as required to accommodate non-hydraulic considerations such as maintenance.

Theoretically an infinite number of designs are possible for a given site. Therefore, to initiate design, constraints must be evaluated and a trial design formulated to meet these constraints. Then by routing the inflow hydrograph through the trial pump station its adequacy can be evaluated.

Design Procedure:

1.) Develop Inflow Hydrograph
2.) Estimate Pumping Rate, Volume of Storage, and Number of Pumps
3.) Determine High Water Level
4.) Determine Pump Pit dimensions,
5.) Pump Cycling and Usable Storage
6.) Estimate Volume of Storage
7.) Stage-Storage Relationship
8.) Determine Total Dynamic Head, Net Positive Head, and Head Capacity Curves
9.) Pump Design Point
10.) Power Requirements
11.) Mass Curve Routing
12.) Documentation

Design Checklist

Initial Data

Contributing Drainage Area
Locating of Outfall
Capacity of Outfall
HYDROLOGY
Environmental Considerations

Possible Components

Building Architectural

Location
Capacity
Storage
Pit Type
Equipment Access
Hoisting Equipment
Safety
Ventilation Equipment
Potable Water Supply
Hazardous Materiel Containment

14.5 Design Procedure (continued)

Design Checklist (continued)

Possible Components

Mechanical System
Pump Type
Power Source
Monitor & Control Systems
Sediment Handling

Collection System
- Inlets
- Trash Rack
- Grit Chamber

Discharge System
Piping
Valving

Hydraulic Analysis
Pump Characteristics
Pipe Losses
Miscellaneous Losses
Mass Curve Routing

14.5 Design Procedure (continued)

14.5.2 Pump Station Design

The procedure for pump station design is illustrated in the following 11 steps.

Step 1 Inflow to Pump Station

Develop inflow hydrograph to the pump station using the procedures presented in the ADOT Hydrology Manual. For highway pump stations where the inflow is of short duration and high intensity, a hydrograph that correctly depicts the time/area/discharge relationship should be used, even for small drainage areas. For small basins, HEC-1 can be used to develop the hydrograph in a two stage process. See Appendix B in Chapter 15.

Step 2 Estimate Maximum Discharge, Pumping Rate and Number of Pumps

Because of the complex relationship between the variables of pumping rates, storage and pump on-off settings, a trial and error approach is usually necessary for estimating the pumping rates and storage required for a balanced design. A wide range of combinations will produce an adequate design. The goal is to develop an economic balance between volume and pumping capacity.

Determine the required type, size, and capacity of pumps and power unit characteristics. Usually the minimum number is 3, however this may be 2 for small pump stations or more than 3 if each pump is limited by power unit size. Develop the system curve for the proposed discharge system and superimpose it on the pump performance curves.

Determine the maximum number of starts/hour (minimum cycle time) for the type of power unit proposed.

Step 3 Allowable High Water Level

The highest permissible water level must be set as 0.5 to 2 feet below the finished pavement surface at the lowest pavement inlet.

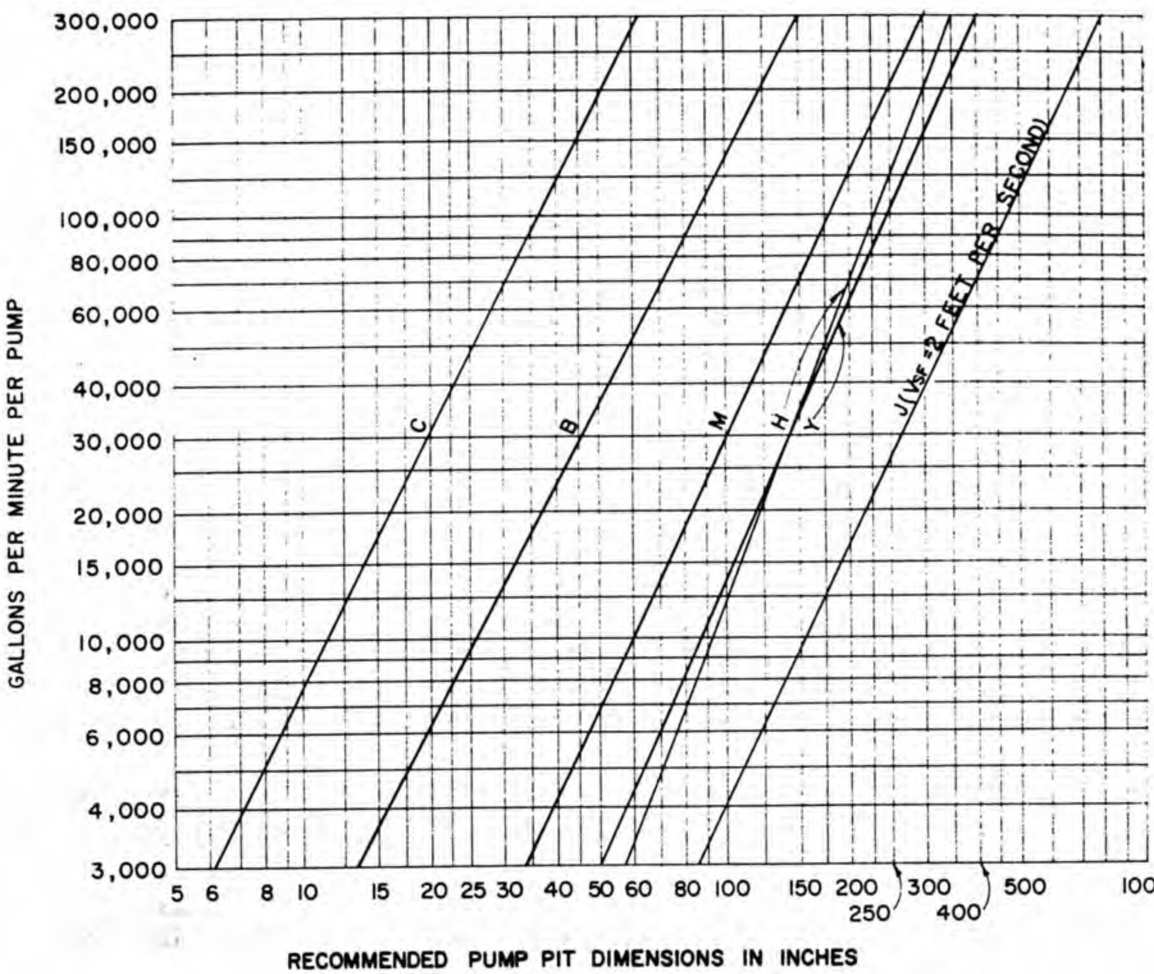
At the design inflow, some head loss will occur through the pipes and appurtenances leading to the pump station. Therefore a hydraulic gradient will be established and the maximum permissible water elevation at the station will be the elevation of the hydraulic gradient. This gradient will be very flat for most wet well designs with exterior storage because of the unrestricted flow into the wet well. Determine the inflow invert elevation

Step 4 Determine Pump Pit Dimensions

Determine the minimum required plan dimensions (LxW) for the pump station from manufacturer's literature or from dimensioning guides such as those provided by the Hydraulic Institute, see Figures 14-12 and 14-13. The dimensions are usually determined by locating the selected number of pumps on a floor plan keeping in mind the guidance given in Section 14.4.3 for clearances and intake system design. Keep in mind the need for clearances around electrical panels and other associated equipment that will be housed in the pump station building.

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)



J = Minimum distance from trash rack to backwall (length of Pump Pit).
V_{SF}= Maximum Velocity of stream flow. (0.5 ft./sec. recommended)
B = Maximum distance from pump centerline to backwall.
C = Average distance from underside of bell to bottom of pit
H = Minimum distance from minimum water level to bottom of pit.
M = Minimum center-to-center of pumps.
Y = Minimum distance to pump centerline from downstream end of any obstruction in sump. (Obstruction must be streamlined.)

Figure 14-12 Recommended Rectangular Pump Pit Dimensions

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

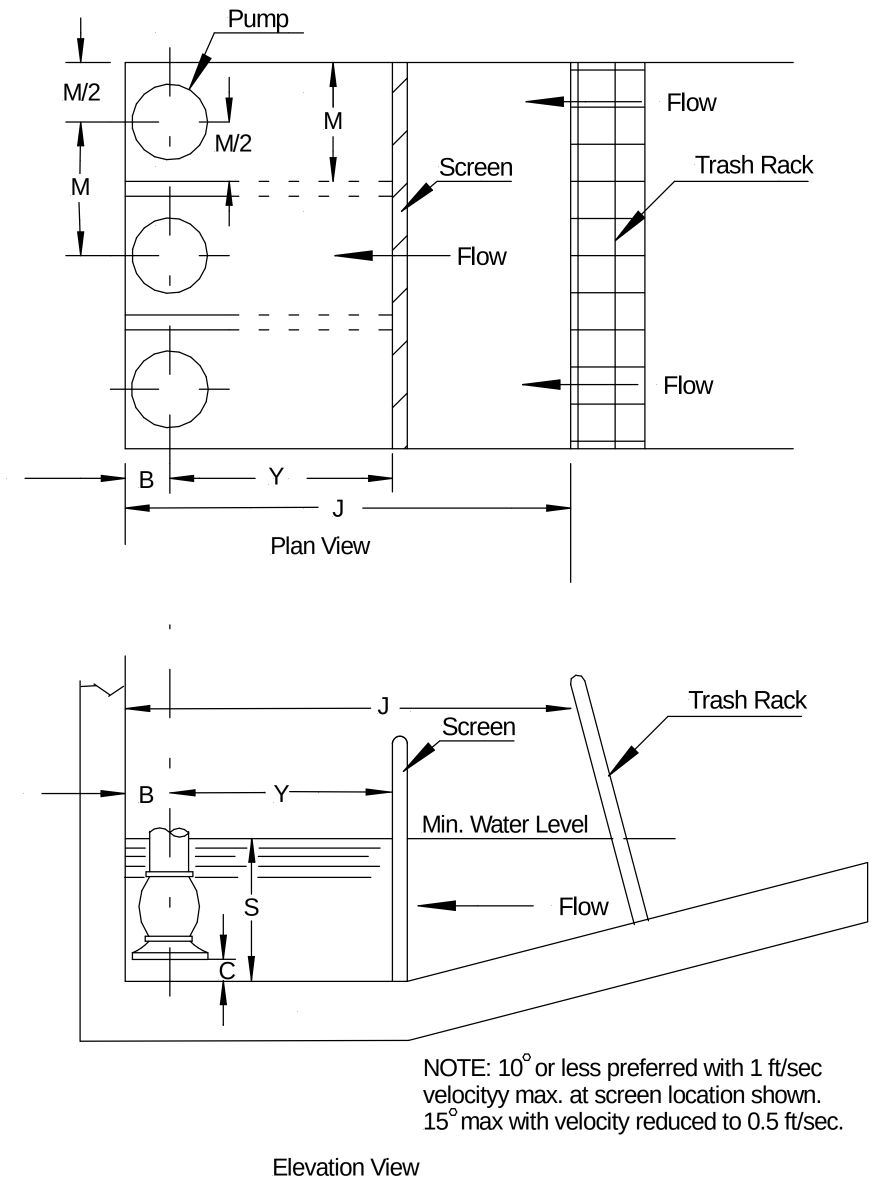


Figure 14-13 Sump Dimension, Wet Pit Type Pumps

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

Step 5 Pump Cycling and Usable Storage

One of the basic parameters addressed initially was that the proper number of pumps must be selected to deliver the design Q. Also, the correct elevations must be chosen to turn each pump on and off. Otherwise, rapid cycling (frequent starting and stopping of pumps) may occur causing undue wear and possible damage to the pumps. The volume of storage between first pump on and off elevations is termed usable volume. For a given pump with a capacity Q_p, cycling will be a maximum (least time between starts) when the inflow Q_i to the usable storage is one-half the pump capacity. Assuming this condition, usable volume can be related to cycling time

Generally, the minimum allowable cycling time, t, for electrically driven pumps is designated by the pump manufacturer based on electric motor size. In general, the larger the motor, the larger is the starting current required, the larger the damaging heating effect and the greater the cycling time required. The pump manufacturer should always be consulted for allowable cycling time during the final design phase of project development.

However, the following limits may be used for estimating allowable cycle time during preliminary design:

Motor kW	Cycling Time (t), min
0 - 11	5.0
15 - 22	6.5
26 - 45	8.0
48 - 75	10.0
112 - 149	13.0

Knowing the pumping rate and minimum cycling time, the minimum necessary allowable storage, V, to achieve this time can be calculated by:

$$\mathbf{V = 15\ Q_p t} \qquad \textbf{(14.5)}$$

Where:

Q_p = pump capacity, cfs.
t = time between starts, in minutes.

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

Step 5 Pump Cycling and Usable Storage (continued)

When larger volumes of storage are available, the initial pump start elevations can be selected from the stage-storage curve. Since the first pump turned on should typically have the ability to empty the storage facility, its turn off elevation would be the bottom of the storage basin. The elevation associated with the minimum allowable storage volume in the stage-storage curve is the lowest turn-on elevation that should be allowed for the starting point of the first pump. The minimum allowable storage is calculated by the equation $V=15\ Q_p t$.

Having selected the trial wet-pit dimensions, the pumping range, Δh, can then be calculated. The pumping range represents the vertical height between pump start and pump stop elevations. Usually, the first pump stop elevation is controlled by the minimum recommended bell submergence criteria specified by the pump manufacturer or the minimum water level, H, specified in the design. The first pump start elevation will be a distance, Δh, above H. When the only storage provided is in the wet pit, the pumping range can be calculated by dividing the required storage volume by the wet pit area.

$$\Delta h = V/\text{wet pit area} \quad (14.6)$$

The minimum plan area of the wet pit based on the pump layout is checked by comparison of the required pumping range with the available pumping range. Determine the design high water elevations (DHW) by adding the storage depth, Δh, to inlet invert elevation. DHW=INV+ Δh. If DHW is less than or equal to the allowable high water elevation (AHW), the dimensions are satisfactory. If the DHW is greater than the AHW, compute

$$X=Vt/[L(AHW-INV)] - W$$

Where L= length of wet well, dimension perpendicular to the line of pumps)
W= width of wet well

If X is less than the width required between pumps compute the new wet well length as

$$L=Vt/[(W(AHW-DHW)],$$ The wet well width should not be increased.

If X is greater than the width required between pumps, and one pump and return to step 2 (i.e., determine the new pump/motor characteristics, wet well dimensions, and cycle time storage volumes).

The second and subsequent pump start elevations will be determined by plotting the pump performance on the mass inflow curve. This distance between pump starts may be in the range of 1 to 3 feet for stations with a small amount of storage and 0.25 to 0.5 feet for larger storage situations.

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

Step 6 Estimate Volume of Storage

Some approximation of storage in addition to the pumping rate is necessary to produce the first trial design. Using the approach presented in section 14.4.4.1 the peak pumping rate is assigned and a horizontal line representing the peak rate is drawn across the top of the hydrograph. The shaded area above the peak pumping rate represents an estimated volume of storage required above the last pump turn on point. This area is measured to give an estimated starting size for the storage facility. Once an estimated storage volume is determined, a storage facility can be estimated. The shape, size, depth, etc., can be established to match the site and a stage-storage relationship can be developed.

Step 7 Stage-Storage Relationship

Routing procedures require that a stage-storage relationship be developed. This is accomplished by calculating the available volume of water for storage at uniform vertical intervals.

Having roughly estimated the volume of storage required and trial pumping rate by the approximate methods described in the preceding sections, the configuration and elevations of the storage chamber can be initially set. Knowing this geometry, the volume of water stored can be calculated for its respective depth. In addition to the wet-pit, storage will also be provided by the inflow pipes and exterior storage if the elevation of water in the wet-pit is above the inflow invert. If the storage pipe is circular, the volume can be calculated using the ungula of a cone formula as discussed in Appendix A, Figure 14-A1. Figures 14-A1, 14-A2 and 14-A3 give examples of the calculation and plotting of the storage in a circular pipe and a circular wet pit. A similar procedure would be followed for other storage configurations. Volume in a storage chamber can be calculated below various elevations by formulas depending on the shape of the chamber. A storage vs. elevation curve can then be plotted and storage below any elevation can readily be obtained.

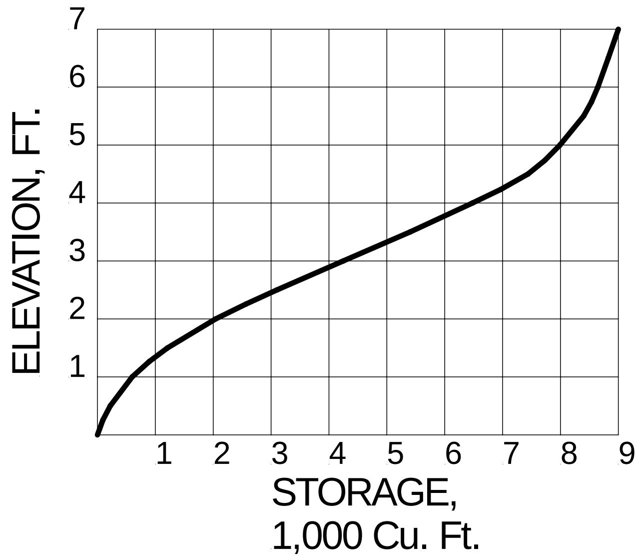


Figure 14-14 Stage-Storage Curve

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

Step 8 Total Dynamic Head

Total Dynamic Head is the sum of the static head, velocity head and various head losses in the pump discharge system due to friction. Knowing the range of water levels in the storage pit and having a trial pump pit design with discharge pipe lengths and diameters and appurtenances such as elbows and valves designated, total dynamic head for the discharge system can be calculated.

The designer must select a specific pump in order to establish the size of the discharge piping that will be needed. This is done by using information either previously developed or established. Though the designer will not typically specify the manufacturer or a specific pump, study of various manufacturers' literature will assist in establishing reasonable relationships between total dynamic head, discharge, efficiency and energy requirements. This study will also give the designer a good indication of discharge piping needed since pumps that produce the desired results will have a specific discharge pipe size.

To summarize the Total Dynamic Head (TDH) is equal to: $TDH = H_s + H_f + H_v + H_p$

Where: H_s = static head or height through which the water must be raised, ft
H_f = loss due to friction in the pipe, ft
H_v = velocity head, ft
H_p = loss due to friction in water passing through the pump valves, fittings and other items, ft.

The Manning's formula expressed as follows is generally used for discharge lines.

$$\mathbf{H_f = L * S_f = L * [(Q * n)/(1.486 * A * R^{2/3})]^2} \qquad \mathbf{(14.7)}$$

Where: Q = discharge, ft^3/sec
L = length of pipe, ft
n = Manning's roughness value
A = cross sectional area of discharge pipe, ft^2
R = hydraulic radius of discharge pipe, ft (For pipe running full, R = diameter/4)

Friction losses can also be computed by the Darcy Formula. This requires computation of the relative roughness of the pipe, the Reynold's number and the friction factor. The Hydraulic Institute and others have produced line loss tables and charts that make determination of losses quite easy and accurate. The tables and charts have been developed for a variety of pipe materials and are recommended for use in determining line and fitting losses for the discharge side of the pumping system.

Headlosses for other components should be evaluated using the information is the Storm Drainage System Chapter. Standard textbooks and manufacturers' catalogs should also be consulted.

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

Step 9 Pump Design Point

Using methods described in the previous step, the Total Dynamic Head of the outlet system is calculated for a specific static head and various discharges. These TDHs are then plotted vs. discharge. This plot is called a system head curve. A system head curve is a graphical representation of total dynamic head plotted against discharge Q for the entire pumping and discharge system. The required design point of a pump can be established after the pump curve is superimposed to give a visual representation of both system and pump. As usually drawn, the system head curve starts from a low point on the Y-ordinate representing the static head at zero discharge. It then rises to the right as the discharge and the friction losses increase. A design point can be selected on the system head curve and a pump can be selected to match that point. The usual pump curve is the reverse of the system head curve so the point of intersection is clearly identifiable. System head curves are often drawn for several different static heads, representing low, design and maximum water levels in the sump.

One, two or more pump curves can be plotted over the system head curves and conditions examined. If a change of discharge line size is contemplated, a new system head curve for the changed size (and changed head loss) is easily constructed. Figure 14-15 shows the difference in the pump operating range when two pumps are connected to a common discharge line versus separate discharge lines. With a common discharge line the design point will move from A, for the first pump operating alone, to B, with both pumps operating at the minimum static head. Where as, the same two pumps with separate discharge lines operate over a more limited range since the additional loss from a shared pipe is not experienced. In highway design, it is common practice to provide individual discharge lines for each pump. Therefore, the additional loss from a shared pipe is not experienced. It should be noted that the pump will always operate at the intersection of the system curve and the pump curve.

Each pump considered will have a unique performance curve that has been developed by the manufacturer. More precisely, a family of curves is shown for each pump, because any pump can be fitted with various size impellers. These performance curves are the basis for the pump curve plotted in the system head curves discussed above. The designer must study pump performance curves. The designer must have specific information on the pumps available in order to be able to specify pumps needed for the pump station.

Any point on an individual performance curve identifies the performance of a pump for a specific Total Dynamic Head (TDH) that exists in the system. It also identifies the power required and the efficiency of operation of the pump. It can be seen that for either an increase or decrease in TDH, the efficiency is reduced as the performance moves away from the mid-point of the performance curve. It should also be noted that as the TDH increases, the power requirement also increases. The designer must make certain that the motor specified is adequate over the full range of TDHs that will exist. It is desirable that the design point be as close to the mid-point as possible, or else to the left of the mid-point rather than to the right of or above it. The range of the pump performance should not extend into the areas where substantially reduced efficiencies exist.

It is necessary that the designer correlate the design point discussed above with an elevation at about the mid-point of the pumping range. By doing this, the pump will work both above and below the TDH for the design point and will thus operate in the best efficiency range.

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

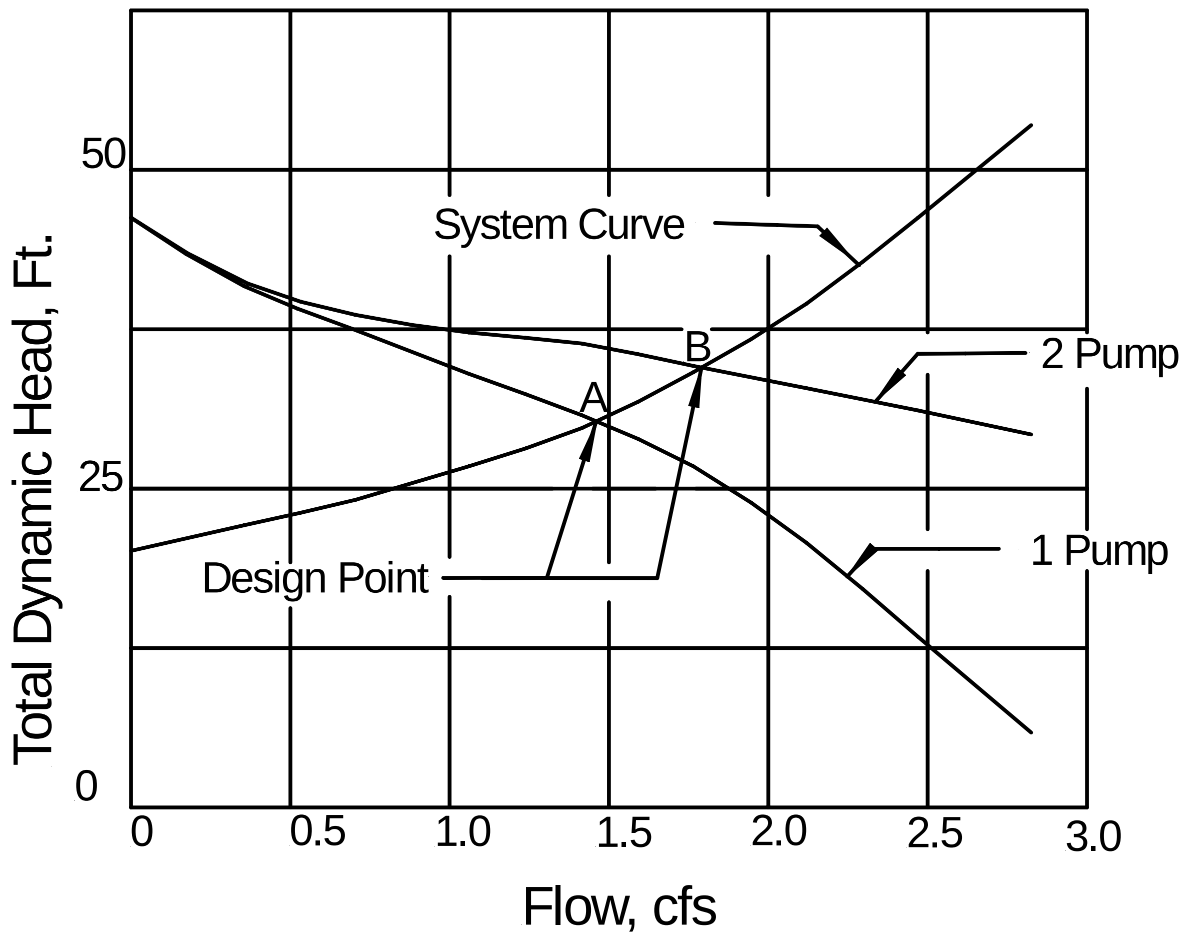


Figure 14-15 System Head Curves

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

Step 10 Power Requirements

To select the proper size of pump motor, compute the energy required to raise the water from its lowest level in the pump pit to its point of discharge. This is best described by analyzing pump efficiency. Pump efficiency is defined as the ratio of pump energy output to the energy input applied to the pump. The energy input to the pump is the same as the driver's output and is called brake kilowatts.

$$e = Q\gamma H/1000 \text{ brake kW} \tag{14.8}$$

Where: e = efficiency = pump output/brake kilowatts
Q = pump capacity, ft^3/sec
γ = specific weight of liquid (62.4 lbs/ft^3 for cold water)
H = head, ft

Efficiency can be broken down into partial efficiencies — hydraulic, mechanical, etc. The efficiency as described above, however, is a gross efficiency used for the comparison of centrifugal pumps. The designer should study pump performance curves from several manufacturers to determine appropriate efficiency ranges. A minimum acceptable efficiency should be specified by the designer for each performance point specified.

To compute the energy required to drive a pump, assume that the pump will operate at 80% efficiency. The above equation can then be solved for brake kilowatts.

Step 11 Mass Curve Routing

The procedures described thus far will provide all the necessary dimensions, cycle times, appurtenances, etc. to design the pump station. A flood event can be simulated by routing the design inflow hydrograph through the pump station by methods described in Section 14.4.4.2. In this way, the performance of the pump station can be observed at each hydrograph time increment and pump station design evaluated. Then, if necessary, the design can be "fine-tuned."

14.6 References

Flygt. Cost Savings In Pumping Stations.

Hydraulic Institute -- Engineering Data Book.

Hydraulic Institute -- American National Standard for Vertical Pumps.

Hydraulic Institute -- American National Standard for Pump Intake Design.

Federal Highway Administration, “Highway Stormwater Pump Station Design Manual”, HEC24, 2000.

Appendix A
Volume in Ungula

Appendix A Volume in Ungula

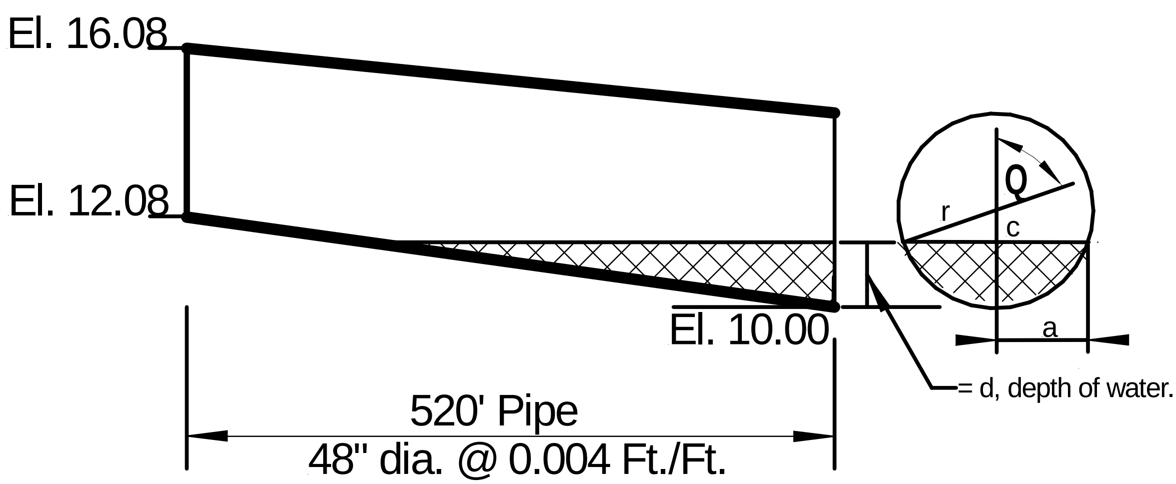


Figure 14-A-1 Storage in Ungula

Source: FHWA IP-82-17, Vol. 1

Ungula Volume: $V = L*(0.67a^3 \pm cB) / (r \pm c)$

Where: L = length of ungula, ft
r = radius of base, ft
B = area of base, ft^2
d = depth of flow, ft
c = distance from water surface to center of circle = r-d = r*sin(Q)
a = ½ width of water surface = $(r^2-c^2)^{0.5}$ = r*cos(Q)

If depth is greater than a semicircle, use + sign. If depth is less than a semicircle, use - sign.

Area of Base, $B = C_aD^2$. See Table A-1, **Area Coefficient** - C_a

Example:
At L = 375 ft, depth = 1.5 ft.; r=2.0 ft;
c = 2.0-1.5 = 0.5; $a = (2.0^2-0.5^2)^{0.5}$ = 1.936
Enter table with d/D = 1.5/4.0 = 0.375. C_a is read from table.
For d/D = 0.375, C_a is interpolated as 0.2691. Therefore B = 0.2691*46 = 4.306 ft^2

The volume for the ungula is $V = L*(0.67a^3 \pm cB) / (r \pm c)$ =
$V = 375*(0.67*(1.936)^3 - 0.5*4.306)/(2.0-0.5) = 375*(4.862-2.153)/(1.5) = 677\ ft^3$

Appendix A Volume in Ungula

Ungula Example

At a depth of 3.5 feet at the entrance, it takes 875 feet to get to zero depth, the depth at the end of the pipe is 3.5 –.004*520 = 1.42.

Therefore volume is the volume for 3.5 feet over 875 feet minus 1.42 feet over 355 feet.

For 3.5 depth, $c = 2.0\text{-}3.5 = -1.5$, $a = (2.0^2-1.5^2)^{0.5} = 1.323$
For $d/D = 3.5/4.0 = 0.875$, C_a is interpolated as 0.729.
Therefore B = 0.729*16= 11.66

The volume for the 3.5' ungula is $V = L*(0.67a^3 \pm cB) / (r \pm c) =$
$V = 875*(0.67*(1.323)^3-(-1.5*11.66)/(2.0-(-1.5)) = 875*(1.551-(-17.49)/(3.5) = 4760\ ft^3$

For 1.42 depth, $c = 2.0\text{-}1.42 = 0.58'$, $a = (2.0^2-0.58^2)^{0.5} = 1.914$
For $d/D = 1.42/4.0 = 0.3575$, C_a is interpolated as 0.2498.
Therefore B = 0.2498*16= 3.997

Volume for the 1.42 ungula over 355 feet is
$V = 355*(0.67*(1.914)^3-(0.58*3.997)/(2.0-(0.58)) = 355*(4.698-2.318)/(1.42) = 595\ ft^3$

Therefore volume of storage = 4760-595 = 4165 ft^3

At a depth of 5.5 feet, the pipe is full for the first 375 feet. At the end, the depth is 3.42'. The volume is equal to the volume of a full pipe minus the volume of a pipe 0.58 feet.
For a depth of 0.58 feet, the volume is as follows:
$c = 2.0\text{-}0.58 = 1.42$, $a = (2.0^2-1.42^2)^{0.5} = 1.408$

Enter table with $d/D = 0.58/4 = 0.145$, C_a is interpolated as 0.0704.
Therefore B = 0.0704*16= 1.126 ft^2

The volume for the unfilled ungula is $V = L*(0.67a^3 \pm cB) / (r \pm c) =$
$V = 145*(0.67*(1.408)^{3-}1.42*1.126)/(2.0-1.42) = 145*(1.870-1.599)/(0.58) = 68\ ft^3$

Therefore volume of storage = 12.57*520 – 68 = 6467 ft^3

Appendix A Volume in Ungula

Table of Area Coefficient based on ratio of depth of flow to Diameter.

Let D/d = (Depth of water)/(Diameter of pipe) and C_a = the tabulated value.
Then Area = $C_a d^2$

Table A-1
Area Coefficient - C_a

D/d	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0013	0.0037	0.0069	0.0105	0.0147	0.0192	0.0242	0.0294	0.0350
0.1	0.0409	0.0470	0.0534	0.0600	0.0668	0.0739	0.0811	0.0885	0.0961	0.1039
0.2	0.1118	0.1199	0.1281	0.1365	0.1449	0.1535	0.1623	0.1711	0.1800	0.1890
0.3	0.1982	0.2074	0.2167	0.2260	0.2355	0.2450	0.2546	0.2642	0.2739	0.2836
0.4	0.2934	0.3032	0.3130	0.3229	0.3328	0.3428	0.3527	0.3627	0.3727	0.3827
0.5	0.393	0.403	0.413	0.423	0.433	0.443	0.453	0.462	0.472	0.482
0.6	0.492	0.502	0.512	0.521	0.531	0.540	0.550	0.559	0.569	0.578
0.7	0.587	0.596	0.605	.0614	0.623	0.632	0.640	0.649	0.657	0.666
0.8	0.647	0.681	0.689	0.697	0.704	0.712	0.719	0.725	0.732	0.738
0.9	0.745	0.750	0.756	0.761	0.766	0.771	0.775	0.779	0.782	0.784

Appendix B
Pump Station Example

Appendix B Pump Station Example

Determine the required storage to reduce the peak flow of 22 cfs to 14 cfs as shown in Figure 14-B-1. Using the assumed storage pipe shown in Figure 14-B-2, the stage-storage curve in Figure 14-B-3, the stage discharge curve in Figure 14-B-4 and the inflow hydrograph in Fig 14-B-1, the storage can be determined.

The inflow mass curve is developed in Figure 14-B-5. Since 14 cfs was to be pumped, it was assumed that two 7 cfs pumps would be used. The pumping conditions are as follows:

	Pump-Start		Pump-Stop	
	Elevation	Volume	Elevation	Volume
Pump No. 1 (7 cfs-3500 gpm)	2.0	2000	0.0	0
Pump No. 2 (7 cfs-3500 gpm)	3.0	4220	1.0	600

The storage volumes (ft^3) are associated with the respective elevations.

To visualize what is happening during the design period, the pump discharge is superimposed on the inflow mass curve as shown in Figure 14-B-6. Note that the first pump is turned on at about hour 11.4 when a storage volume of 2000 ft^3 has accumulated. At about hour 11.5 pump number one has emptied the storage basin and the pump turns off. At about hour 11.7 the storage volume has again reached 2000 ft^3 and a pump is turned on. If an alternating start plan had been developed, this would be the second pump that would turn on at this point. If an alternating start plan had not been designed the first pump would again be started. At about hour 11.8 the volume in storage has increased to 4220 ft^3, which is associated with a turn on elevation of 3.0 ft. Both pumps operate until about hour 12.4 when the volume in the storage basin has been essentially pumped out. The pumps will continue to start and stop until the hydrograph has receded and the inflow stops.

The shaded area between the curves (see Figure 14-B-7) represents stormwater that is going into storage. Pump cycling at the end of the storm has been omitted in order to simplify the illustration. When the stored volume remaining is equal to the volume (600 ft^3) associated with the Pump No. 2 stop elevation (1.0 ft), pump number 2 shuts off. Pump No. 1 shuts off when the storage pipe is emptied at Pump No. 1 stop elevation (0.0).

Appendix B Pump Station Example (Continued)

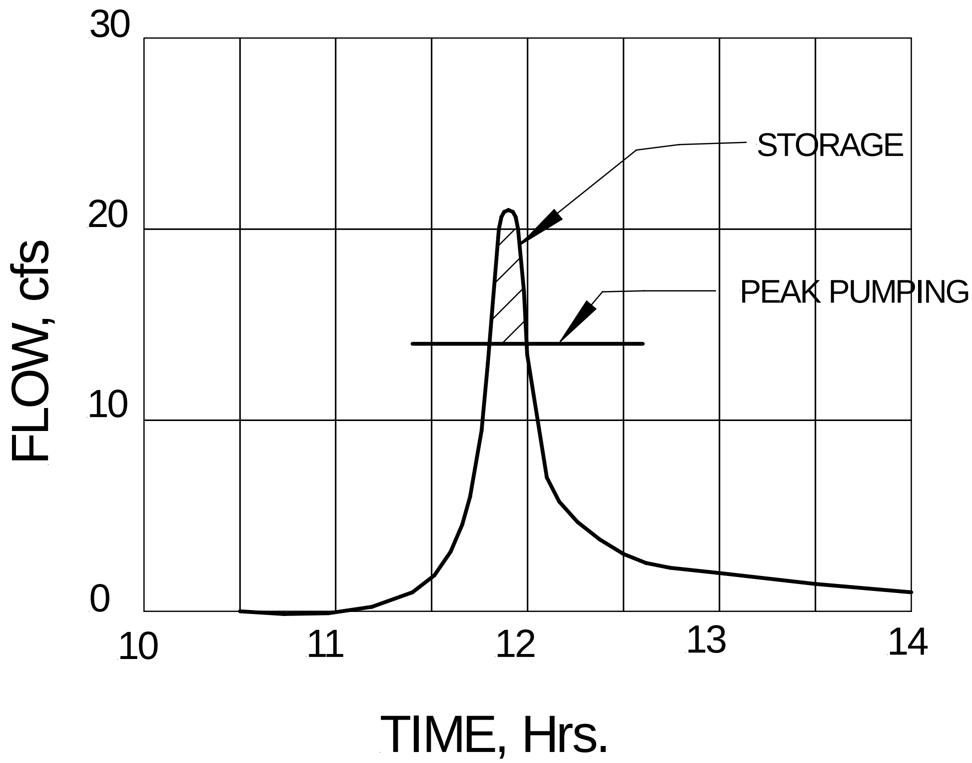


Figure 14-B-1 Example Estimated Required Storage

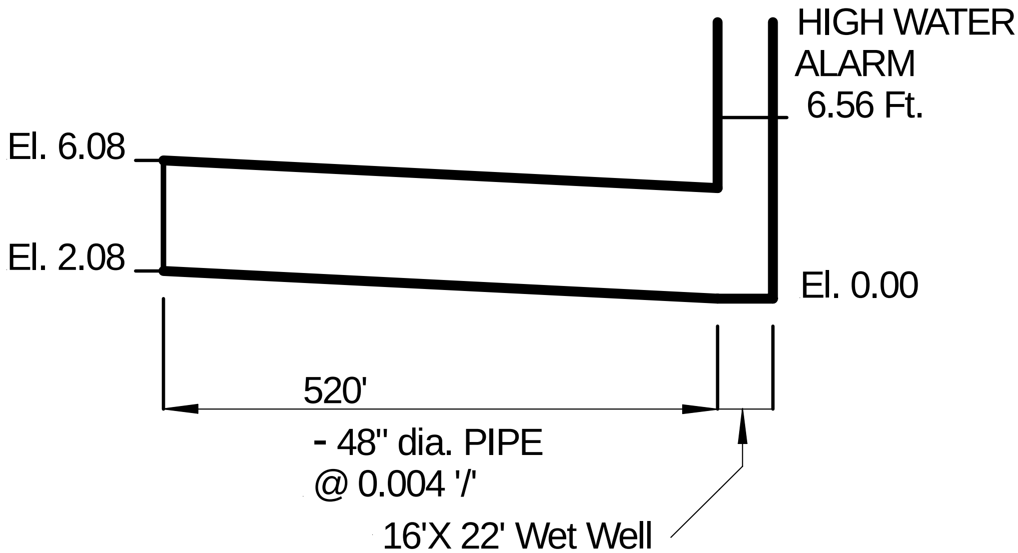


Figure 14-B-2 Storage Pipe Sketch

Appendix B Pump Station Example (Continued)

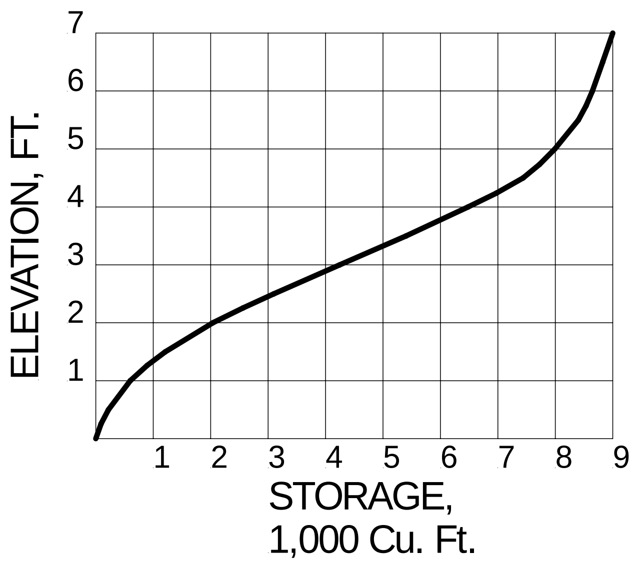


Figure 14-B-3 Stage-Storage Curve

Stage-Storage Tabulation 48 in Pipe at 0.40%, 16’ x 22’ Wet Well

Elevation (Ft)	Pipe (Ft^3)	Wet Well (Ft^3)	Total (Ft^3)
0.00	0.00	0.00	0.00
0.5	46	176	222
1.0	253	352	605
1.5	673	528	1201
2.0	1334	704	2038
2.5	2213	880	3093
3.0	3188	1056	4244
3.5	4169	1232	5401
4.0	5072	1408	6480
4.5	5772	1584	7356
5.0	6230	1760	7990
5.5	6468	1936	8404
6.0	6534	2112	8646
6.5	6534	2288	8822
7.0	6534	2464	8998

Appendix B Pump Station Example (Continued)

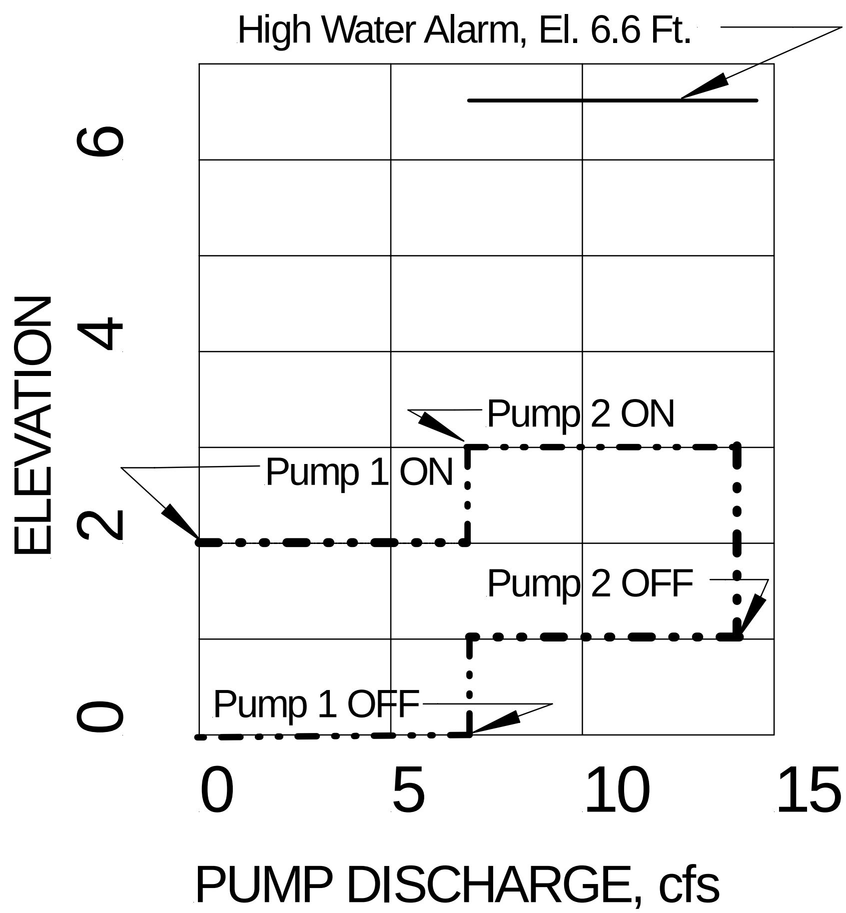


Figure 14-B-4 Pump Stage-Discharge Curve

Appendix B Pump Station Example (Continued)

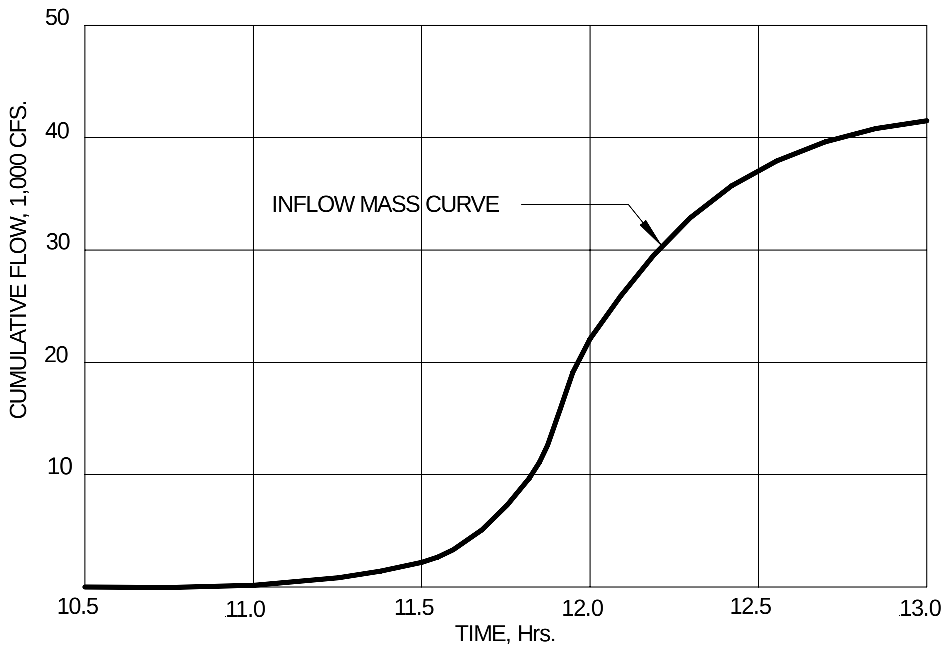


Figure 14-B-5 Development Of Inflow Mass Curve

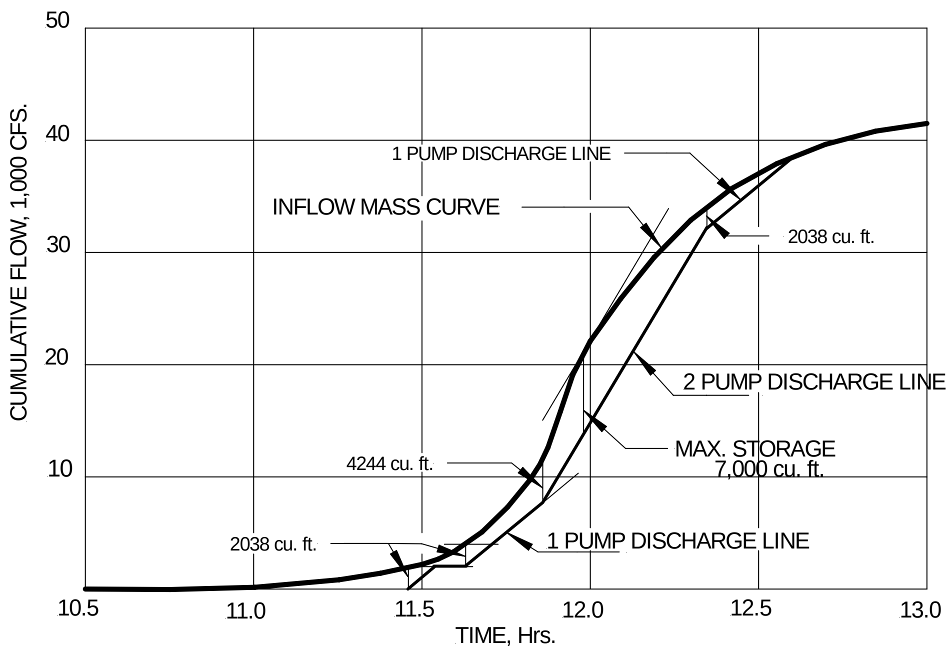


Figure 14-B-6 Mass Curve Diagram

Appendix B Pump Station Example (Continued)

The trial design is now complete. The peak inflow rate of 21 cfs has been reduced to a peak outflow rate of 14 cfs through use of a maximum storage of 8400 ft^3 . A reduction of 33 % has been achieved.

It should be noted that the number of starts per hour can be determined by looking at the plots on the inflow mass curve. Once the mass inflow curve has been developed, it is a relatively easy process to try different pumping rates and different starting elevations until a satisfactory design is developed. This is only one possible design option. Other pumping rates are plotted on the inflow mass curve to determine their performance. Other combinations can be considered. The pumping rate can be reduced by providing more storage. The storage may be reduced by reducing starting elevations.

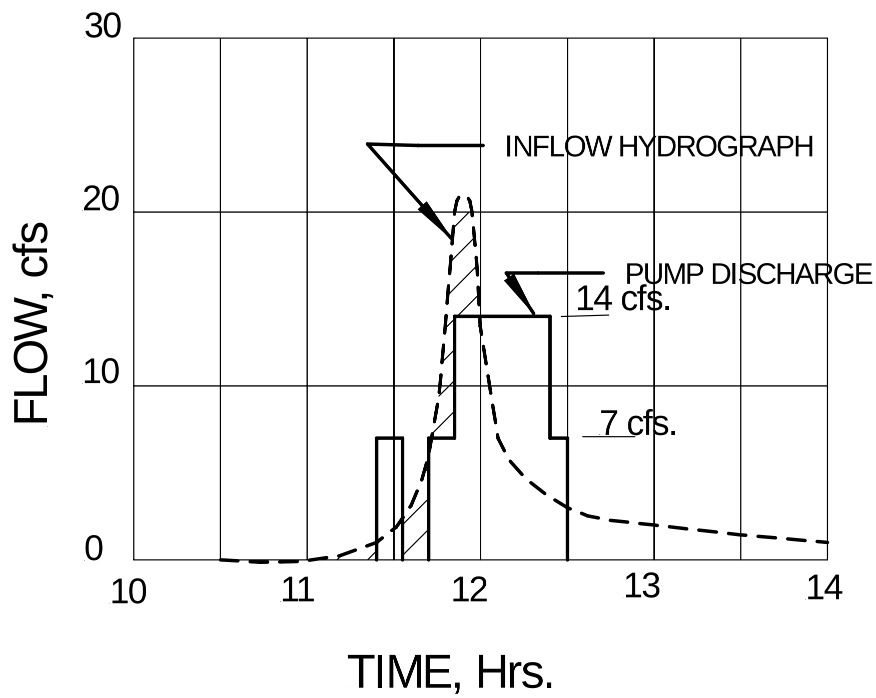


Figure 14-B-7 Pump Operation Hydrographs

CHAPTER 15

STORAGE FACILITIES

Chapter 15 Storage Facilities
Table Of Contents

15.1 Introduction

15.1.1 Overview

This chapter provides general design criteria for detention/retention storage basins as well as procedures for performing preliminary and final sizing and reservoir routing calculations. Storage and flood routing associated with culverts is addressed in the Culvert Chapter (note: criteria presented in this chapter do not necessarily apply to routine culvert design).

15.1.2 Detention And Retention

Stormwater storage facilities are often referred to as either detention or retention facilities. For the purposes of this chapter, detention facilities are those that are designed to detain runoff for some short period of time sufficient to reduce the peak discharge. They are designed using a dynamic stage-storage-discharge relationship. Retention facilities are designed to contain the flows until the storm has passed. They are drained by either gravity or pumping. Recharge basins are a special type of detention basin designed to drain into the groundwater table; these are not addressed in this manual. Ponds with a drainage basin less than 160 acres may be sized based on total retention of the inflow.

Storage facilities are also classified on the basis of whether they are dry or wet between storm events. A dry pond has on outlet positioned at or below the lowest elevation in the pond, such that the pond drains completely between storm events. A wet pond, however, has its lowest outlet at an elevation above the bottom of the pond. Water remains in the pond between storm events and is depleted only by infiltration or evaporation. Only dry ponds will be discussed further.

Since most of the design procedures are the same for detention and retention facilities, the term storage facilities will be used in this chapter to include detention and retention facilities. If special procedures are needed for detention or retention facilities these will be specified.

15.1.3 Regional versus On-Site Detention

Stormwater storage facilities may be designed to handle drainage that is local to the highway system, address off-site flows coming to the highway or be part of a larger regional system. The type of basin and cooperating stakeholders will affect the design criteria and methods.

15.2 Symbols And Definitions

To provide consistency within this chapter as well as throughout this manual, the following symbols will be used. These symbols were selected because of their wide use in technical publications. In some cases the same symbol is used in existing publications for more than one definition. Where this occurs in this chapter, the symbol will be defined where it occurs in the text or equations.

Table 15-1 Symbols And Definitions

Symbol	Definition	Units
A	Cross sectional or surface area	ft^2
C	Weir coefficient	—
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
f	Infiltration rate	ft/hr
g	Acceleration due to gravity	ft/sec^2
H	Head on structure	ft
H_c	Height of weir crest above channel bottom	ft
I	Infiltration rate	mm/hr
I	Inflow rate	ft^3/sec
L	Length	ft
O	Outflow rate	ft^3/sec
Q	Flow	ft^3/sec
S,Vs	Storage volume	ft^3, Ac-ft
t	Routing time period	sec
t_b	Time base on hydrograph	hrs
T_i	Duration of basin inflow	hrs
t_p	Time to peak	hrs
W	Width of basin	ft
z	Side slope factor	—

ADWR is the Arizona Department of Water Resources. This agency has jurisdiction over dams.

Emergency spillway is the outlet location where water will flow when it exceeds the storage capacity of the basin. The location and orientation shall be selected with consideration of the impact of the direction that water will outflow.

Jurisdictional Dam is a structure subject to ADWR jurisdiction, structures that store less than 50 Acre-feet and have an impoundment height of less than 25 feet are not considered jurisdictional. See Appendix A.

Principal spillway is the outlet structure through which it is intended that the design outflow occur without requiring flow through the emergency outlet.

Freeboard is the vertical distance from the maximum water surface to the lowest bank elevation.

15.3 Design Objectives And Concepts

15.3.1 Introduction

Stormwater storage basins are used to reduce the impact of stormwater flows on downstream areas. The impact concerns may be of quantity and/or quality. The most common desired outcome is a lower peak flow. The process involves the directing of stormwater runoff into a basin and controlling the outflow to a controlled lower rate though a properly sized outlet. The outflow hydrograph will have a lower peak discharge and a longer duration of discharge.

15.3.2 Quantity Goals

Quantity goals for storage facilities may be based on:

- prevention or reduction of peak runoff rate increases caused by changes in the watershed,
- mitigation of downstream drainage capacity problems,
- reduction or elimination of the need for downstream outfall improvements, and
- recharging of groundwater resources.

The objectives for stormwater quantity are typically based on limiting the peak discharge rates to match one or more of the following values:

- historic rates for specific design conditions (i.e., post-development peak equals pre-development peak for specified frequency of occurrence.)
- limiting risks based on the hydraulic capacity of the downstream drainage system, or
- a specified value set by jurisdictional regulation or agreement (such as determined by joint project agreements for downstream improvements).

Location Considerations

The utility of providing a storage facility depends on the amount of storage, its location within the system and its operational characteristics. In addition to controlling the peak discharge from the outlet works, storage facilities will change the timing of the entire hydrograph. Thus it is important for the engineer to design storage facilities as a drainage structure that both controls runoff from a defined area and interacts with other drainage structures within the watershed. Multiple storage facilities located in the same watershed will affect the timing of the runoff through the conveyance system that could result in a decrease or increase of flood peaks in different downstream locations. If several storage facilities are located within a particular watershed it is important to determine what effects a particular facility may have on combined hydrographs in down-stream locations. Effective stormwater management must be coordinated on a regional or basin-wide basis.

The evaluation of storage facilities should include comparison of the design flow at a point or points downstream of the proposed storage site with and without the additional storage. This may require channel routing calculations to be carried downstream to a point where the effect of the proposed storage facility hydrograph on the downstream hydrograph can be assessed for detrimental impacts on downstream areas.

For some installations, it may be feasible and desirable to by-pass some portion of the approach flow. This is accommodated for in the design by adjustment of the peak outflow and by-pass flow to meet the desired downstream discharge. This is especially useful if the by-pass flow is high enough so that very common storm events are not diverted into the storage basin.

15.3 Design Concepts & Objectives (continued)

15.3.3 Quality Goals

Quantity goals for storage facilities may be based on:

- control of sediment deposition and
- providing filtration or capture of first flush pollutants.

15.3.4 Basin Design Concept

The basic concept associated with detention basins is the attenuation of flow through the application of a routing analysis for a given pond, outlet configuration and inflow hydrograph. At any given time interval one of the following possibilities is occurring:

- If the average inflow rate is larger than the average outflow rate for the given time interval, the volume of water stored increased, the water surface increased, and the average outflow rate for the next time interval increased.
- If the average inflow rate is equal to the average outflow rate for the given time interval, the volume of storage and the water surface stayed constant.
- If the average inflow rate is less than the average outflow rate for the give time interval, the volume of water stored decreased, the water surface decreased, and the average outflow rate for the next time interval decreased.

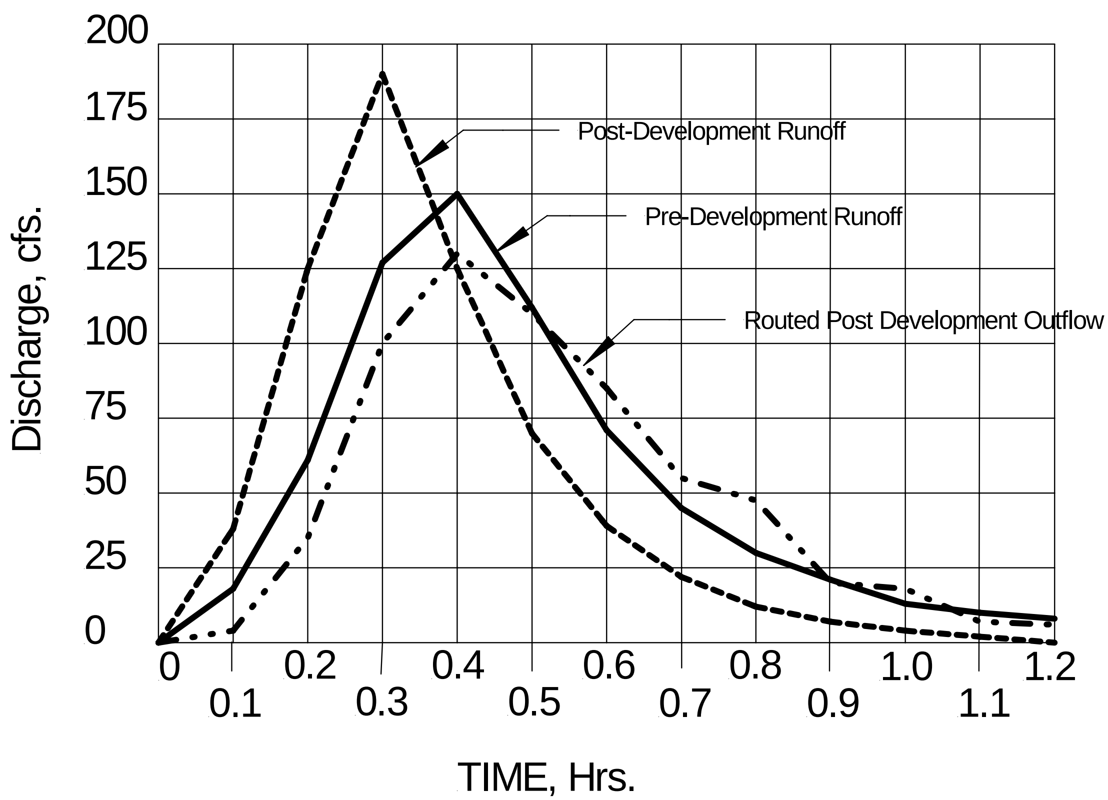


Fig. 15.1 Detention Routing: Inflow-Outflow hydrograph

15.3 Design Concepts & Objectives (continued)

15.3.4 Basin Design Concept (continued)

The volume under the inflow hydrograph is the total volume of runoff entering the basin. The volume under the outflow hydrograph is the total volume of runoff leaving the basin. The change in the value of the peak discharge and the time of peak flow is accomplished by the attenuation of the flow through the basin. The volume above the outflow hydrograph and below the inflow hydrograph is the amount of storage required. The maximum volume occurs when the two hydrographs intersect, which is also when the maximum outflow rate occurs.

For most detention basins the outflow outlet is a culvert or weir structure that is uncontrolled. For a basin with an uncontrolled outlet to a free outfall, the peak storage and the peak outflow will occur at the point where the outflow hydrograph intersects the inflow hydrograph.

A general procedure using the above data in the design of storage facilities is presented below. This procedure can involve a significant number of reservoir routing calculations to obtain the desired results.

Step 1 Compute inflow hydrograph for runoff from the appropriate design storms using the procedures outlined in the ADOT Hydrology Manual. Both pre- and post-development hydrographs may be required.

Step 2 Perform preliminary calculations to estimate detention storage requirements for the hydrographs from Step 1 (see Section 15.7). When looking at a range of storms, if storage requirements are satisfied for runoff from the minimum and maximum design storm events, runoff from intermediate storms is assumed to be controlled. The maximum storage requirement calculated should be used.

Step 3 Determine the physical dimensions necessary to hold the estimated volume, including freeboard.

Step 4 Size the outlet structure. The estimated peak stage will occur for the estimated maximum volume. The outlet structure should be sized to convey the allowable discharge at this peak stage.

Step 5 Perform routing calculations using inflow hydrographs to check the design using the storage routing equations. If the routed outflow peak discharges exceed the desired outflow peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated basin volume and return to step 3.

Step 6 When a satisfactory initial basin size is determined, consider emergency overflow from runoff due to storms larger than the design storm and established freeboard requirements.

Step 7 Evaluate the downstream effects of detention outflow to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility should be routed though the downstream channel system until a confluence point is reached where the drainage area being retained represents 10% of the total drainage area.

Step 8 Evaluate the structure outlet velocity and provide channel and bank stabilization if the velocity would cause erosion problems downstream.

15.4 Design Criteria

15.4.1 General Criteria

In addition to the operational design flow, flows in excess of the design flow that might be expected to pass through the storage facility should be included in the analysis (i.e., 100-year flood), especially for the spillway. The design criteria for storage facilities should include:

- release rate,
- grading and depth requirements,
- outlet connection and location,

These will be used to determine the

- outlet size,
- storage volume, and
- shape/layout of basin.

15.4.2 Release Rate

Release rates shall approximate the desired/required peak runoff rates for the design storm events, with the emergency overflow capable of handling flows in excess of the design storms. In addition, the release rate shall accomplish draining of all detention volume within 36 hours after the cessation of the storm event.

The storage facility may be required to meet release rates for a range of design storms. This may be accomplished using multi-stage control structures (Figure 15-12). If a multi-stage control structure satisfies the upper and lower discharge requirements, discharges from intermediate storm return periods can usually be assumed to be adequately controlled.

The emergency spillway should be capable of passing flows up to 120% of the 100-year storm.

15.4.3 Storage

Storage volume shall be adequate to attenuate the post-development peak discharge rates to the values required as stated above. To achieve these rates, the basin's storage should be equal to the area between the pre- and post-construction hydrographs.

15.4.4 Grading And Depth

15.4.4.1 General

The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Storage facilities situated above ground may become subject to the Arizona Revised Statue (A.R.S.) 45-1201 Dams and Reservoirs as administered by the Arizona Department of Water Resources (ADWR). Structures that have impoundment depths greater than 6 ft and storage volumes greater than 15 Ac-Ft may be come under the jurisdiction of the ADWR unless the facility is excavated below existing ground. See Appendix A for ADWR jurisdiction criteria.

15.4 Design Criteria (continued)

15.4.4.1 General (continued)

It is ADOT's goal not to construct or own any facilities that come under ADWR jurisdiction. ADOT prefers to create storage facilities by excavation below existing ground.

A basin can have multiple levels: one to hold the smaller storms, and a second, which is rarely inundated and can be used for other purposes, to store the larger storms. Other considerations when setting depths include flood elevation requirements, public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, maintenance requirements and required freeboard. Aesthetically pleasing features are also important in urban areas.

A minimum freeboard of 1 foot above the design storm high water elevation shall be provided.

Storage facilities shall be designed to address the following maintenance concerns:

- maintenance of fences and perimeter plantings.
- grass and vegetation maintenance,
- bank deterioration,
- blockage of outlet structures,
- litter accumulation
- weed growth,
- standing water or soggy surfaces,
- sedimentation control,
- mosquito control,

The following sections provide guidelines that should minimize maintenance problems.

15.4.4.2 Grading

Areas above the normal high water elevations of storage facilities should be sloped at a minimum of 5% toward the facilities to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff.

Side slopes are limited to those that are compatible with the landscape treatment; 2.5:1 for desert/rock slopes and 3:1 for grass slopes.

The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A minimum 2% bottom slope is recommended. A low flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows, and prevent standing water conditions. If sediment deposition is expected, an initial sedimentation trap basin with access should be considered. Access ramps shall be provided for maintenance equipment. Generally, 25 feet is provided between the top of the basin and the right-of-way line.

15.4 Design Criteria (continued)

15.4.5 Inlet and Outlet Works

Inlet structures may be necessary where the inflow is being dropped into the basin. Riprap or other energy dissipation measures may be necessary. If by-pass flows are desired, then a diversion structure is necessary in the upstream channel. Diversion structures are of two types. One type diverts a fixed percentage of the approach flow. This type of diversion is accomplished using a flow splitter. The second type diverts the discharge above a base flow. Flows below the base level are not captured into the detention basin. Diversion of flows above a base level is more effective at reducing the required size of storage

The outlet structure allows flows to discharge from the storage basin at a controlled rate. Outlet works selected for storage facilities typically include a principal spillway and an emergency overflow. The principal spillway is intended to convey the design storm without requiring flow to enter an emergency outlet. Outlets can be designed in a wide variety of configurations. Outlet works may be sized to vary the outflow with the varying depth. Outlet works can take the form of combinations of drop inlets, pipes, weirs and orifices at various levels. Slotted riser pipes are discouraged because of clogging problems. If the outlet is a pipe through an embankment, then an anti-seepage collar should be provided to minimize piping by water leakage of the soil particles surrounding the pipe.

For large storage facilities, selecting a flood magnitude for sizing the emergency outlet should be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The minimum flood to be used to size the emergency outlet is the 120% of the 100-year flood. The emergency outlet shall be designed to operate under extreme conditions to prevent failure of the retention structure. The outlet may need to be protected with riprap or paving to prevent excessive damage to the spillway if it would be subjected to high velocity flows. Design information regarding spillway hydraulics is available in *Design of Small Dams*, USBR, 1987.

15.4.6 Access/Protective Treatment

Protective treatment may be required to prevent entry to facilities that present a hazard. Safety considerations include fencing the basin, reducing the maximum depth and/or including ledges and mild slopes to prevent people from falling in and facilitate their escape from the basin. Fences may be required for detention areas where one or more of the following conditions exist:

- Rapid stage increases would make escape practically impossible.
- Water depths exceed 3 feet for more than 24 hours.
- A low-flow watercourse or ditch passing through the detention area has a depth greater than 0.5 ft or a flow velocity greater than 5 ft/s.
- Side slopes equal or steeper than 2:1.

15.4 Design Criteria (continued)

15.4.6 Access/Protective Treatment (continued)

ADOT concurrence shall be obtained when the basin is not to be fenced. Grates or fencing may be appropriate for other conditions, but in all circumstances heavy debris must be transported through the detention area. In some cases, it may be advisable to fence the watercourse or ditch rather than the detention area.

15.5 General Procedure

15.5.1 Basic Concept

The basic concept involved in storm water detention analysis is a routing procedure using the storage-indication method to transform the inflow hydrograph into the outflow hydrograph. Routing calculations needed to analyze storage facilities, although not extremely complex, are time consuming and repetitive. To assist with these calculations there are many available reservoir routing computer programs.

The routing calculations are based on the Puls method. To perform the calculations one must have the following data:

- Inflow hydrograph for all selected design storms. Generally derived by use of HEC-1.
- Stage-storage curve for proposed storage facility (see section 15.5.2). For large storage volumes, use Acre-Feet, otherwise use cubic feet.
- Stage-discharge curve for all outlet control structures (see Section 15.5.3).

Using these data, a routing procedure is used to route the inflow hydrograph through the storage facility with different basin and outlet geometry until the desired outflow hydrograph is achieved (see Section 15.8). The computation begins with inflow. At a given time interval, the inflow is known. Using the storage at the end of the previous time interval, add the inflow volume to get the intra-period Stage, using the intra-period stage get the outflow rate. Use the outflow rate to get the intra-period outflow volume. Use the beginning of period volume plus the intra-period inflow volume minus the intra-period outflow volume to determine the end of period volume. The next time interval is then calculated.

The general procedure described above is presented below. See Section 15.8 for a detailed description of each step.

Step 1 Develop an inflow hydrograph, stage-discharge curve and stage-storage curve for the proposed storage facility.

Step 2 Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph ($\Delta t < T_c/5$).

Step 3 Use the storage-discharge and stage-storage data from Step 1 to develop storage characteristics curves that provide values of $S \pm (O/2)\Delta t$ versus stage. An example tabulation of storage characteristics curve data is shown in Table 15-4.

15.5 General Procedure (continued)

15.5.1 Basic Concept (continued)

Step 4 I_1 and I_2 are known. Given the depth of storage or stage, H_1, at the beginning of that time interval, S_1-$(O_1/2)\Delta t$ can be determined from the appropriate storage characteristics curve (Figure 15-10).

Step 5 Determine the value of $S_2 + (O_2/2)\ \Delta t$ from the following equation

$$S_2 + (O_2/2)\ \Delta t = [S_1 - (O_1/2)\ \Delta t] + [(I_1 + I_2)/2]\Delta t \tag{15.2}$$

Where: S_2 = storage volume at time 2, ft^3
O_2 = outflow rate at time 2, ft^3/sec
Δt = routing time period, sec
S_1 = storage volume at time 1,ft^3
O_1 = outflow rate at time 1, ft^3/sec
I_1 = inflow rate at time 1, ft^3/sec
I_2 = inflow rate at time 2, ft^3/sec
Other consistent units are equally appropriate.

Step 6 Enter the storage characteristics curve at the calculated value of S_2+$(O_2/2)\Delta t$ determined in Step 5 and read off a new depth of water, H_2.

Step 7 Determine the value of O_2, which corresponds to a stage of H_2 determined in Step 6, using the stage-discharge curve.

Step 8 Repeat Steps 1 through 7 by setting new values of I_1, O_1, S_1 and H_1 equal to the previous I_2, O_2, S_2 and H_2, and using a new I_2 value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

15.5.2 Stage-Storage Curve

A stage-storage curve defines the incremental relationship between the depth of water and storage volume in a reservoir, Figure 15-3. Storage basins may be irregular in shape to blend well with the surrounding terrain. The data for this type of curve are usually developed using a topographic map and one of the following formulas: the average-end area, frustum of a pyramid, or prismoidal formulas. The average-end area formula is usually preferred as the method to be used on non-geometric areas. This is usually developed from the stage versus surface area data. The information should extend from the basin invert to the top of the embankment. The precision of the information increases as the interval of stage decreases.

15.5 General Procedure (continued)

15.5.2 Stage-Storage Curve (continued)

The **double-end area** formula is expressed as:

$$V_{1,2} = [(A_1 + A_2)/2]d \qquad (15.3)$$

Where: $V_{1,2}$ = storage volume, ft^3, between elevations 1 and 2
$A_{1,2}$ = surface area at elevations 1 and 2 respectively, ft^2
d = change in elevation between points 1 and 2, ft

The **frustum of a pyramid** is expressed as:

$$V = d/3\ [A_1 + (A_1A_2)^{0.5} + A_2] \qquad (15.4)$$

Where: V = volume of frustum of a pyramid, ft^3
d = change in elevation between points 1 and 2, ft
$A_{1,2}$ = surface area at elevations 1 and 2 respectively, ft^2

The **prismoidal formula** for trapezoidal basins is expressed as:

$$V = LWD + (L + W)\ ZD^2 + (4/3)\ Z^2\ D^3 \qquad (15.5)$$

Where: V = volume of trapezoidal basin, ft^3
L = length of basin at base, ft
W = width of basin at base, ft
D = depth of basin, ft
Z = side slope factor, ratio of vertical to horizontal

15.5.3 Stage-Discharge Curve

A stage-discharge curve defines the relationship between the stage (depth) of water and the discharge or outflow from a storage facility, Figure 15-3. This curve is also called a *rating curve*. A typical storage facility has two spillways: principal and emergency. The stage-discharge curve should take into account the discharge characteristics of both the principal and emergency spillways.

Tailwater influences and structure losses must be considered when developing discharge curves. If a combination of outlet structures is used, backwater effects of one structure may affect the discharge of the combination of structures. Section 15.6 presents methods for calculating the stage-discharge relationship of weirs. The culvert chapter presents methods for calculating the stage-discharge relationship for structures having with a Hw/D ratio greater than 1.2 or affected by tailwater. The precision of the information increases as the interval of stage decreases.

15.5 General Procedure (continued)

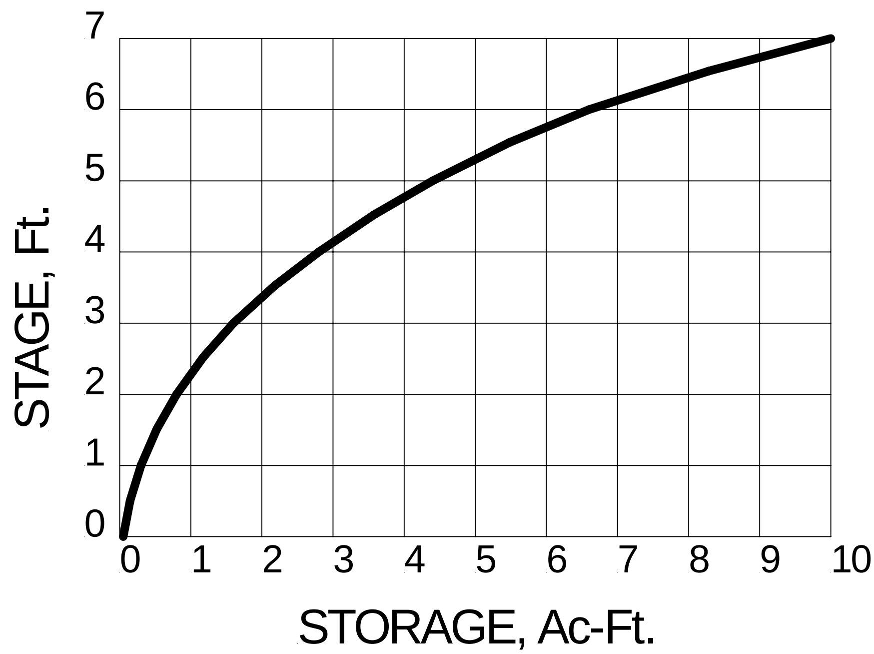


Figure 15-2 Example Stage-Storage Curve

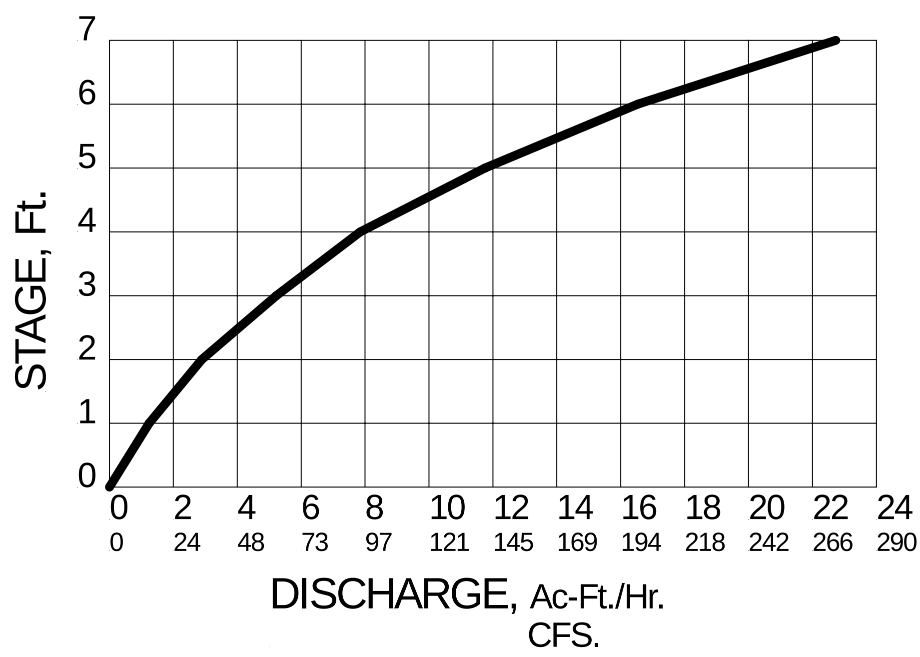


Figure 15-3 Example Stage-Discharge Curve

15.6 Outlet Hydraulics

15.6.1 General

Sharp-crested weir flow equations for no end contractions, two end contractions and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs, proportional weirs and orifices, or combinations of these facilities. If culverts are used as outlets works, procedures presented in the Culvert Chapter should be used to develop stage-discharge data. When analyzing release rates the tailwater influence on the control structure (orifice and/or weirs) must be considered to determine the effective head on each opening.

15.6.2 Sharp-Crested Weirs

A sharp-crested weir with no end contractions is illustrated in Figure 15-4. The discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = [(3.27 + 0.4(H/H_c)]\ LH^{1.5}} \qquad \textbf{(15.6)}$$

Where: Q = discharge, ft^3/sec
H = head above weir crest excluding velocity head, ft
H_c = height of weir crest above channel bottom, ft
L = horizontal weir length, ft

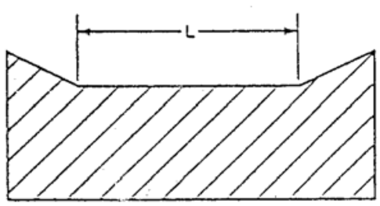


Figure 15-4

Sharp-Crested Weir (No End Contractions)

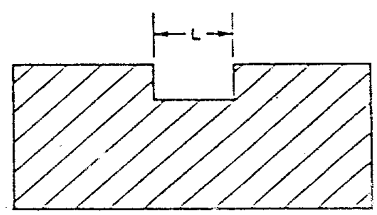


Figure 15-5

Sharp-Crested Weir (Two End Contractions)

A sharp-crested weir with two end contractions is illustrated in Figure 15-5. The discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = [(3.27 + 0.4(H/H_c)]\ (L - 0.2H)\ H^{1.5}} \qquad \textbf{(15.7)}$$

Where: Variables are the same as equation 15.6.

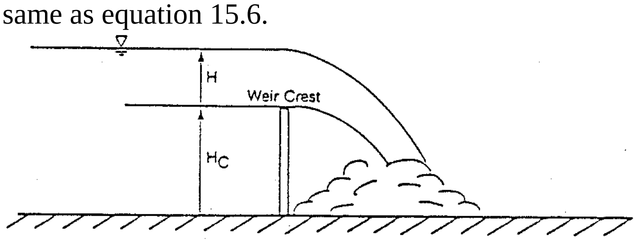


Figure 15-6 Sharp-Crested Weir And Head

15.6 Outlet Hydraulics (continued)

15.6.2 Sharp-Crested Weirs (continued)

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$\mathbf{Q_S = Q_f(1 - (H_2/H_1)^{1.5})^{0.385}} \qquad \textbf{(15.8)}$$

Where: Q_s = submergence flow, ft^3/sec
Q_f = free flow, ft^3/sec
H_1 = upstream head above crest, ft
H_2 = downstream head above crest, ft

15.6.3 Broad-Crested Weirs

The equation generally used for the broad-crested weir is (Brater and King, 1976):

$$\mathbf{Q = CLH^{1.5}} \qquad \textbf{(15.7)}$$

Where: Q = discharge, ft^3/sec
C = broad-crested weir coefficient
L = broad-crested weir length, ft
H = head above weir crest, ft

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Additional information on C values as a function of weir crest breadth and head is given in Table 15-3.

15.6 Outlet Hydraulics (continued)

15.6.3 Broad-Crested Weirs (continued)

Table 15-3 Broad-Crested Weir Coefficient C Values
As A Function Of Weir Crest Breadth And Head (ft)

Measured Head, H[1] (Ft)	Breadth Of The Crest Of Weir (Ft) 0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.0	15.0
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.22	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.32	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

[1]Measured at least 2.5H upstream of the weir.

Reference: Brater and King (1976).

15.6 Outlet Hydraulics (continued)

15.6.4 V-Notch Weirs

The discharge through a v-notch weir can be calculated from the following equation (Brater and King, 1976).

$$\mathbf{Q = 2.5 \tan(\theta/2)H^{2.5}} \qquad \mathbf{(15.10)}$$

Where: Q = discharge, ft^3/sec
θ = angle of v-notch, degrees
H = head on apex of notch, ft

15.6.5 Proportional Weirs

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head.

Design equations for proportional weirs are (Sandvik, 1985):

$$\mathbf{Q = 4.97\, a^{0.5}\, b(H - a/3)} \qquad \mathbf{(15.11)}$$
$$\mathbf{x/b = 1 - (1/3.17)\,(\arctan\,(y/a)^{0.5})} \qquad \mathbf{(15.12)}$$

Where: Q = discharge, ft/sec
Dimensions a, b, H, x and y are shown in Figure 15-7.

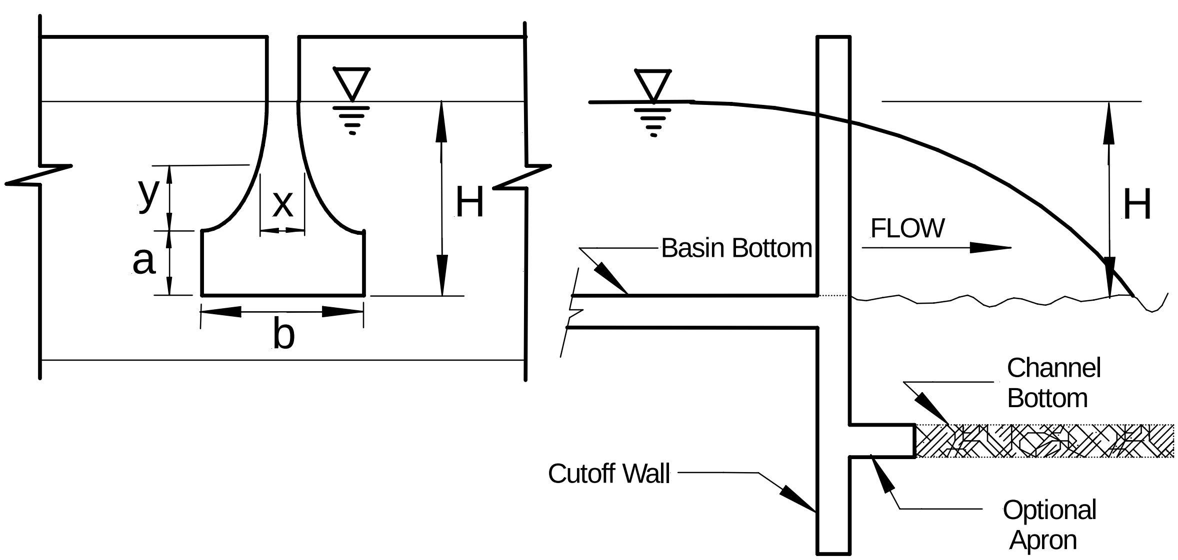


Figure 15-7 Proportional Weir Dimensions

15.6 Outlet Hydraulics (continued)

15.6.6 Orifices

Pipes smaller than 12 inch may be analyzed as a submerged orifice if H/D is greater than 1.5. For square-edged entrance conditions,

$$Q = 0.6A(2gH)^{0.5} = 3.78\ D^2H^{0.5} \tag{15.13}$$

Where: Q = discharge, ft^3/sec
A = cross-section area of pipe, ft^2
g = acceleration due to gravity, 32.2 ft/sec^2
D = diameter of pipe, ft
H = head on pipe, from the center of pipe to the water surface, ft *

- In cases where the tailwater is higher than the center of the opening, the head is calculated as the difference in water surface elevations.

15.7 Preliminary Detention Volume Calculations

15.7.1 Storage Volume

A preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 15-8 shown below.

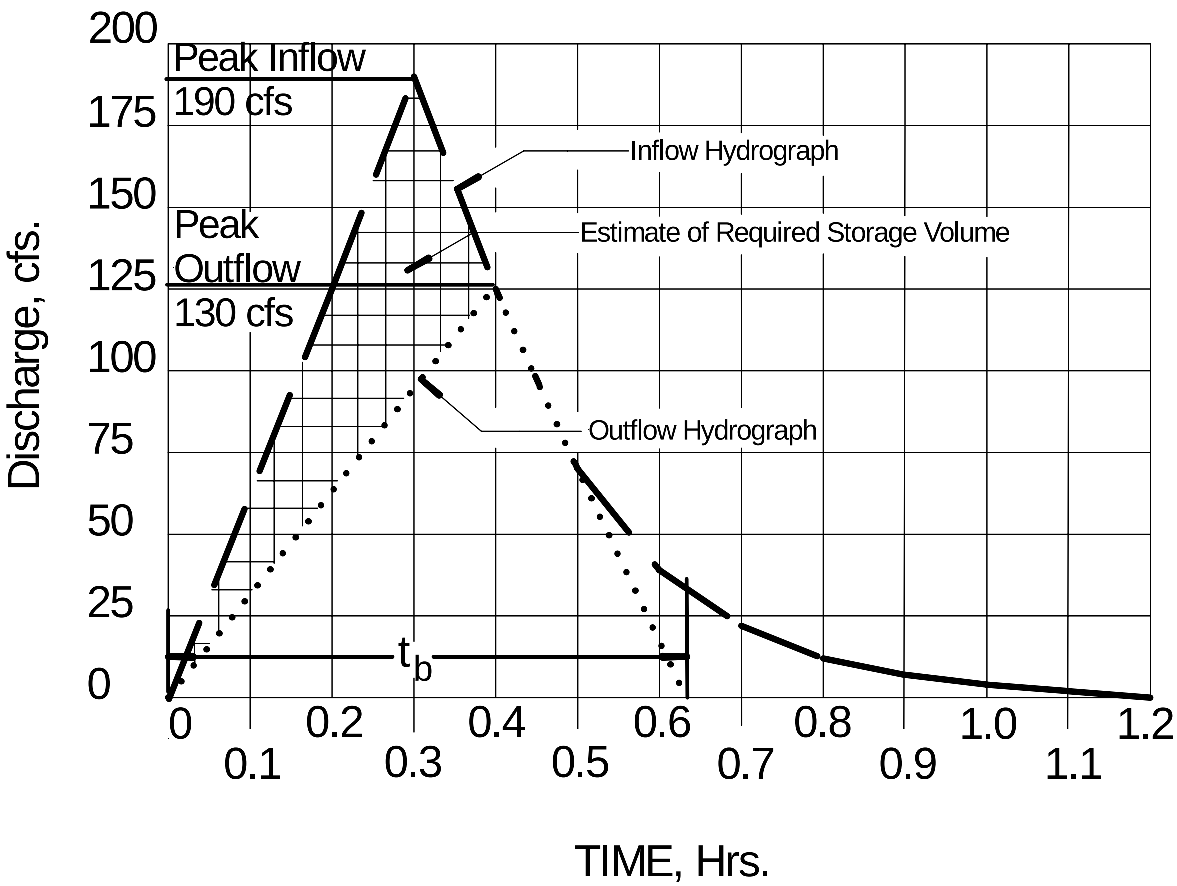


Figure 15-8 Triangular Shaped Hydrographs

(For Preliminary Estimate Of Required Storage Volume)

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$\mathbf{V_S = 0.5T_b(Q_i - Q_o)} \qquad \textbf{(15.14)}$$

Where: V_S = storage volume estimate, ft^3

Q_i = peak inflow rate, ft^3/sec

Q_o = peak outflow rate, ft^3/sec

T_b = duration of basin inflow, t_b

Any consistent units may be used for Equation 15.14.

15.7 Preliminary Detention Volume Calculations (continued)

15.7.1 Storage Volume (continued)

For basins which have a contributing drainage area less than 160 acres, the storage volume may be estimated assuming that all flow is retained. The volume is estimated as

$$V=CAP_{24}$$

Where: V = storage volume estimate, Ac-ft
C = Rational Runoff coefficient
A = Contributing drainage area, Acre
P_{24} = 24 hour rainfall amount, ft.

15.8 Routing Calculations

The routing calculations are based on the Puls method. The fundamental principle is the conservation of mass, i.e, the change in storage during a time interval is equal to the difference between the inflow and outflow volumes. The method described below is a finite difference solution of the conservation of mass relationship, it is commonly known as the storage-indication method. The calculations are the process of analyzing the difference between the flow entering and the flow leaving the basin for a series of time increments. The difference determines the change in volume and water surface elevation.

To perform the calculations one must have the inflow hydrograph, the stage-storage and stage-discharge relationships. The computation begins with inflow. At a given time interval, the inflow is known. Using the storage at the end of the previous time interval, add the inflow volume to get the intra-period Stage, using the intra-period stage get the outflow rate. Use the outflow rate to get the intra-period outflow volume. Use the beginning of period volume plus the intra-period inflow volume minus the intra-period outflow volume to determine the end of period volume. The next time interval is then calculated.

The general procedure described above is presented below.

Step 1 Develop an inflow hydrograph, stage-discharge curve and stage-storage curve for the proposed storage facility. Example stage-storage curve is shown below in Figure 15-9, and stage-discharge curve is shown in Figure 15-10.

Step 2 Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph ($\Delta t < T_c/5$).

Step 3 Use the storage-discharge and stage-storage data from Step 1 to develop storage characteristics curves that provide values of $S \pm (O/2)\Delta t$ versus stage. An example tabulation of storage characteristics curve data is shown in Table 15-4.

Step 4 I_1 and I_2 are known. Given the depth of storage or stage, H_1, at the beginning of that time interval, $S_1-(O_1/2)\Delta t$ can be determined from the appropriate storage characteristics curve (Figure 15-10).

15.8 Routing Calculations (continued)

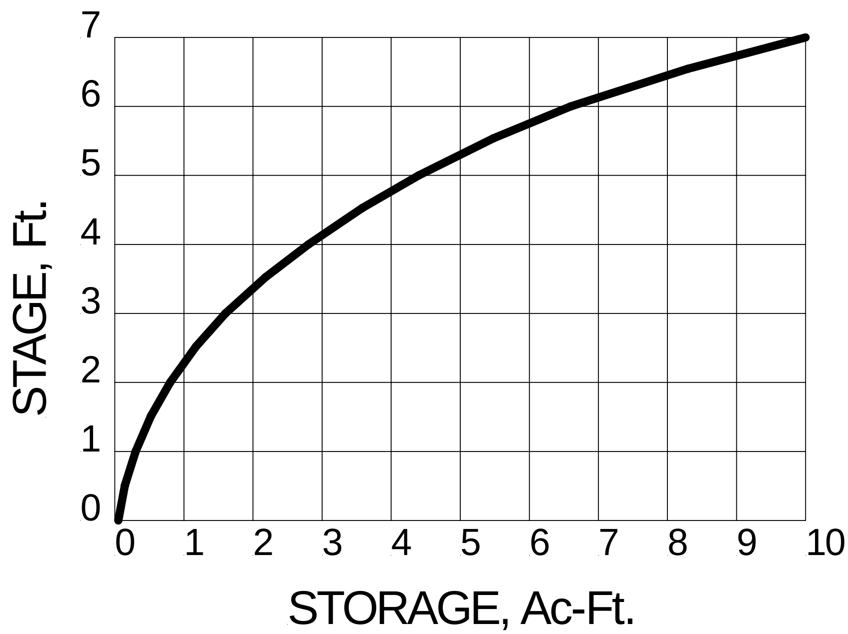


Figure 15-9 Example Stage-Storage Curve

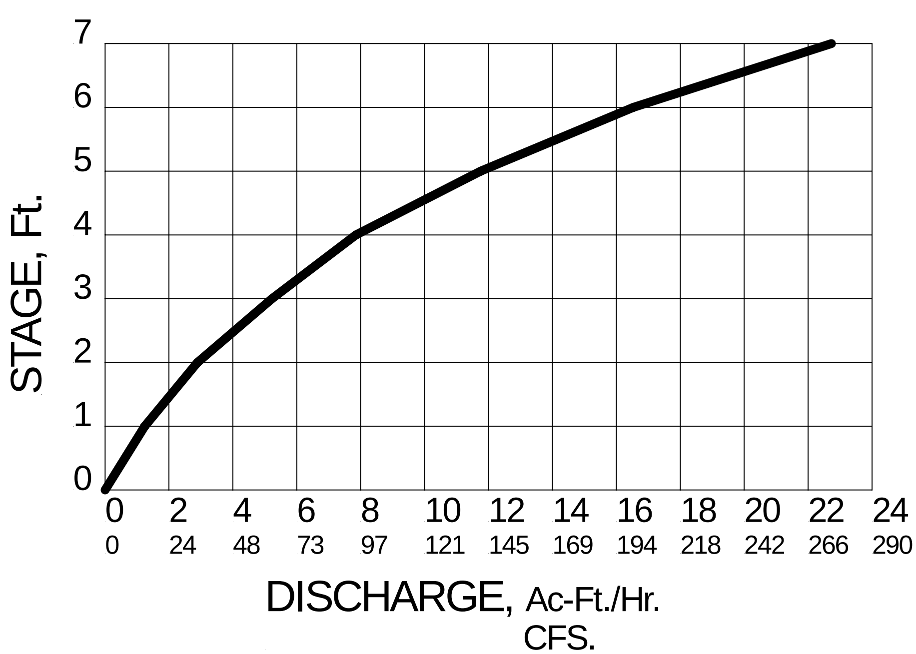


Figure 15-10 Example Stage-Discharge Curve

15.8 Routing Calculations (continued)

Table 15-4 Storage Characteristics

(1) Stage (ft)	(2) Storage[1] (Ac-Ft)	(3) Discharge[2] (ft^3/s)	(4) (Ac-Ft/hr)	(5) S-(O/2)Δt (Ac-Ft)	(6) S+(O/2)Δt (Ac-Ft)
100	0.05	0	0.0	0.05	0.05
101	0.3	15	1.24	0.20	0.40
102	0.8	35	2.89	0.56	1.04
103	1.6	63	5.21	1.17	2.03
104	2.8	95	7.85	2.15	3.45
105	4.4	143	11.72	3.41	5.39
106	6.6	200	16.53	5.22	7.98
107	10.0	275	22.73	8.11	11.89

[1] Obtained from the Stage-Storage Curve.
[2] Obtained from the Stage-Discharge Curve.
Note: Δt = 10 min = 0.167 h, and 1cfs=0.0826 ac-ft/hr.

Step 5 Determine the value of $S_2 + (O_2/2)\ \Delta t$ from the following equation

$$\mathbf{S_2 + (O_2/2)\ \Delta t = [S_1 - (O_1/2)\ \Delta t] + [(I_1 + I_2)/2]\Delta t} \qquad \textbf{(15.15)}$$

Where: S_2= storage volume at time 2, ft^3
O_2 = outflow rate at time 2, ft^3/sec
Δt = routing time period, sec
S_1 = storage volume at time 1,ft^3
O_1 = outflow rate at time 1, ft^3/sec
I_1 = inflow rate at time 1, ft^3/sec
I_2 = inflow rate at time 2, ft^3/sec
Other consistent units are equally appropriate.

Step 6 Enter the storage characteristics curve at the calculated value of $S_2+(O_2/2)\Delta t$ determined in Step 5 and read off a new depth of water, H_2.

Step 7 Determine the value of O_2, which corresponds to a stage of H_2 determined in Step 6, using the stage-discharge curve.

Step 8 Repeat Steps 1 through 7 by setting new values of I_1, O_1, S_1 and H_1 equal to the previous I_2, O_2, S_2 and H_2, and using a new I_2 value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

15.8 Routing Calculations (continued)

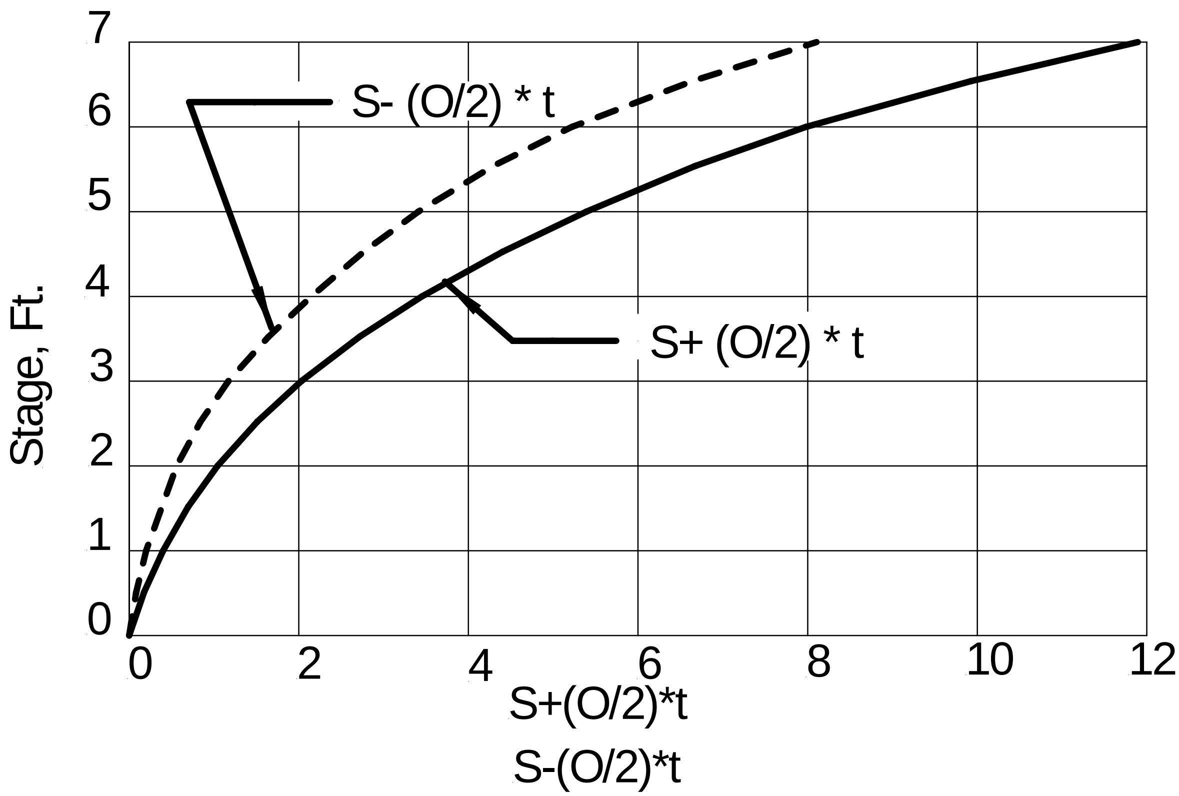


Figure 15-11 Storage Characteristic Curve

15.9 Example Problem

This example demonstrates the application of the methodology presented in this chapter for the design of a typical detention storage facility. Example inflow hydrographs and associated peak discharges for both pre- and post-development conditions have been developed.

15.9.1 Design Discharge And Hydrographs

Storage facilities are to be designed for runoff from both the 2- and 10-year design storms and an analysis done using the 100-year design storm runoff to ensure that the structure can accommodate runoff from this storm without damaging adjacent and downstream property and structures. Runoff hydrographs are shown in Table 15-5 below. Inflow durations from the post-development hydrographs are about 1.2 and 1.25 h, respectively, for runoff from the 2- and 10-year storms. Peak discharges from the 2- and 10-year design storm events are as follows:

Pre-development

2-year peak discharge	10-year peak discharge
150 ft^3/s	200 ft^3/s

Post-development

2-year peak discharge	10-year peak discharge
190 ft^3/s	250 ft^3/s

Since the post-development peak discharge must not exceed the pre-development peak discharge, the allowable design discharges are 150 and 200 ft^3/s for the 2- and 10-year storms, respectively.

Table 15-5

Runoff Hydrographs

	Pre-Development Runoff		Unrouted Post-Development Runoff	
(1)	(2)	(3)	(4)	(5)
Time (h)	2-Year (ft^3/s)	10-Year (ft^3/s)	2-Year (ft^3/s)	10-Year (ft^3/s)
0.0	0	0	0	0
0.1	18	24	38	50
0.2	61	81	125	178
0.3	127	170	190	250
0.4	150	200	125	165
0.5	112	150	70	90
0.6	71	95	39	50
0.7	45	61	22	29
0.8	30	40	12	16
0.9	21	28	7	9
1.0	13	18	4	5
1.1	10	15	2	3
1.2	8	13	0	1

15.9 Example Problem (continued)

15.9.2 Preliminary Volume Calculations

Preliminary estimates of required storage volumes are obtained using the simplified method outlined in Section 15.8. For runoff from the 2- and 10-year storms, the required storage volumes, V_S, are computed using equation 15.13:

$V_S = 0.5T_i(Q_i - Q_o)$

Using a time base of 0.62 hours (figure 15-8) for the 2-year event and 0.7 hours for the 10-year event,

2-year storm: $V_S = 0.5(0.62 \times 3600)(190\text{-}150)/43560 = 1.28$ Ac-Ft

10-year storm: $V_S = 0.5(0.70 \times 3600)(250\text{-}200)/43560 = 1.45$ Ac-Ft

Table 15-6

Stage-Discharge-Storage Data

(1) Stage (ft)	(2) Q (ft^3/s)	(3) S (Ac-Ft)	(4) S_1-(O/2)Δt (Ac-Ft)	(5) S_1+(O/2)Δt (Ac-Ft)
0.00	0	0	0.0	0.0
0.9	10	0.26	0.22	0.30
1.4	20	0.42	0.33	0.50
1.8	30	0.56	0.44	0.68
2.2	40	0.69	0.52	0.85
2.5	50	0.81	0.60	1.02
2.9	60	0.93	0.68	1.18
3.2	70	1.05	0.76	1.34
3.5	80	1.17	0.84	1.50
3.7	90	1.28	0.92	1.66
4.0	100	1.40	0.99	1.81
4.5	120	1.63	1.14	2.13
4.8	130	1.75	1.21	2.29
5.0	140	1.87	1.29.	2.44
5.3	150	1.98	1.36	2.60
5.5	160	2.10	1.44	2.76
5.7	170	2.22	1.52	2.92
6.0	180	2.34	1.60	3.08
6.4	200	2.58	1.76	3.41
6.8	220	2.83	1.92	3.74
7.0	230	2.95	2.00	3.90
7.4	250	3.21	2.17	4.24

Note: Δt = 10 min = 0.167 h, and 1cfs=0.0826 ac-ft/hr.

15.9 Example Problem (continued)

15.9.3 Routing Calculations

Stage-discharge and stage-storage characteristics of a storage facility are presented above in Table 15-6 that should provide adequate peak flow attenuation for runoff from both the 2- and 10-year design storms, which are shown in Tables 15-7 and 15-8. The storage-discharge relationship was developed by requiring the preliminary storage volume estimates of runoff for both the 2- and 10-year design storms to be provided when the corresponding allowable peak discharges occurred. Discharge values were computed by solving the broad-crested weir equation for head, H, assuming a constant discharge coefficient of 3.1, a weir length of 4 ft and no tailwater submergence. The capacity of storage relief structures was assumed to be negligible.

Storage routing was conducted for runoff from both the 2- and 10-year design storms to confirm the preliminary storage volume estimates and to establish design water surface elevations. Routing results using the Stage-Discharge-Storage data given on the previous page and the Storage Characteristics Curves given on Figures 15-9 and 15-10, and 0.1-hour time steps are shown below for runoff from the 2- and 10-year design storms, respectively. The preliminary design provides adequate peak discharge attenuation for both the 2- and 10-year design storms.

Table 15-7 Storage Routing For The 2-Year Storm

(1) Time (hrs)	(2) Inflow (cfs)	(3) $[(I_1+I_2)]/2$ (Ac-Ft)	(4) H_1 (ft)	(5)=(6-3) $S_1-(O_1/2)\Delta t$ (Ac-Ft)	(6)=(3+5) $S_2+(O_2/2)\Delta t$ (Ac-Ft)	(7) H_2 (ft)	(8) Outflow (cfs)
0.0	0.	0.0	0.00	0.00	0.00	0.00	0.00
0.1	38	0.16	0.00	0.00	0.16	0.43	3
0.2	125	0.67	0.43	0.10	0.77	2.03	36
0.3	190	1.30	2.03	0.50	1.80	4.00	99
0.4	125	1.30	4.00	0.99	2.29	4.80	130<150 OK
0.5	70	0.81	4.80	1.21	2.02	4.40	114
0.6	39	0.45	4.40	1.12	1.57	3.60	85
0.7	22	0.25	3.60	0.87	1.12	2.70	55
0.8	12	0.14	2.70	0.65	0.79	2.08	37
0.9	7	0.08	2.08	0.50	0.58	1.70	27
1.0	4	0.05	1.70	0.421	0.47	1.03	18
1.1	2	0.02	1.30	0.327	0.34	1.00	12
1.2	0	0.01	1.00	0.25	0.26	0.70	7
1.3	0	0.0	0.70	0.15	0.15	0.40	3

15.9 Example Problem (continued)

Table 15-8 Storage Routing For The 10-Year Storm

(1) Time (hrs)	(2) Inflow (cfs)	(3) $[(I_1+I_2)]/2$ (Ac-Ft)	(4) H_1 (ft)	(5) S_1-$(O_1/2)\Delta t$ (Ac-Ft)	(6) S_2+$(O_2/2)\Delta t$ (Ac-Ft)	(7) H_2 (ft)	(8) Outflow (cfs)
0.0	0.00	0.0	0.00	0.0	0.0	0.00	0.00
0.1	50	0.21	0.21	0.0	0.21	0.40	3
0.2	178	0.94	0.40	0.08	1.02	2.50	49
0.3	250	1.77	2.50	0.60	2.37	4.90	134
0.4	165	1.71	4.90	1.26	2.97	2.97	173<200OK
0.5	90	1.05	5.80	1.30	2.35	4.95	137
0.6	50	0.58	4.95	1.25	1.83	4.10	103
0.7	29	0.33	4.10	1.00	1.33	3.10	68
0.8	16	0.19	3.10	0.75	0.94	2.40	46
0.9	9	0.10	2.40	0.59	0.69	1.90	32
1.0	5	0.06	1.90	0.44	0.50	1.40	21
1.1	3	0.03	1.40	0.33	0.36	1.20	16
1.2	1	0.02	1.20	0.28	0.30	0.90	11
1.3	0	0.0	0.90	0.22	0.22	0.60	6

For the routing calculations the following equation was used:

$S_2 + (O_2/2)\ \Delta t = [S_1 - (O_1/2)\ \Delta t] + [(I_1 + I_2)/2\ \Delta t]$

Also, column 6 = column 3 + column 5
And column 5=column 6 - column 3.

Since the routed peak discharge is lower than the maximum allowable peak discharges for both design storm events, the weir length could be increased or the storage decreased. If revisions are desired, routing calculations must be repeated.

Although not shown for this example, runoff from the 100-year storm should be routed through the storage facility to establish freeboard requirements and to evaluate emergency overflow and stability requirements.

In addition, the preliminary design provides hydraulic details only. Final design should consider site constraints such as depth to water, side slope stability and maintenance, grading to prevent standing water and provisions for public safety.

15.9 Example Problem (continued)

15.9.4 Downstream Effects

An estimate of the potential downstream effects (i.e., increased peak flow rate and recession time) of detention storage facilities may be obtained by comparing hydrograph recession limbs from the pre-development and routed post-development runoff hydrographs. Example comparisons are shown below for the 10-year design storms.

Potential effects on downstream facilities should be minor when the maximum difference between the recession limbs of the pre-developed and routed outflow hydrographs is less than about 20%. As shown in Figure 15-12, the example results are well below 20%; downstream effects can thus be considered negligible and downstream flood routing omitted. However, it is important to be aware that the increased total volumes of water being released slowly over a longer period of time may contribute to bed and bank decay in the receiving channel.

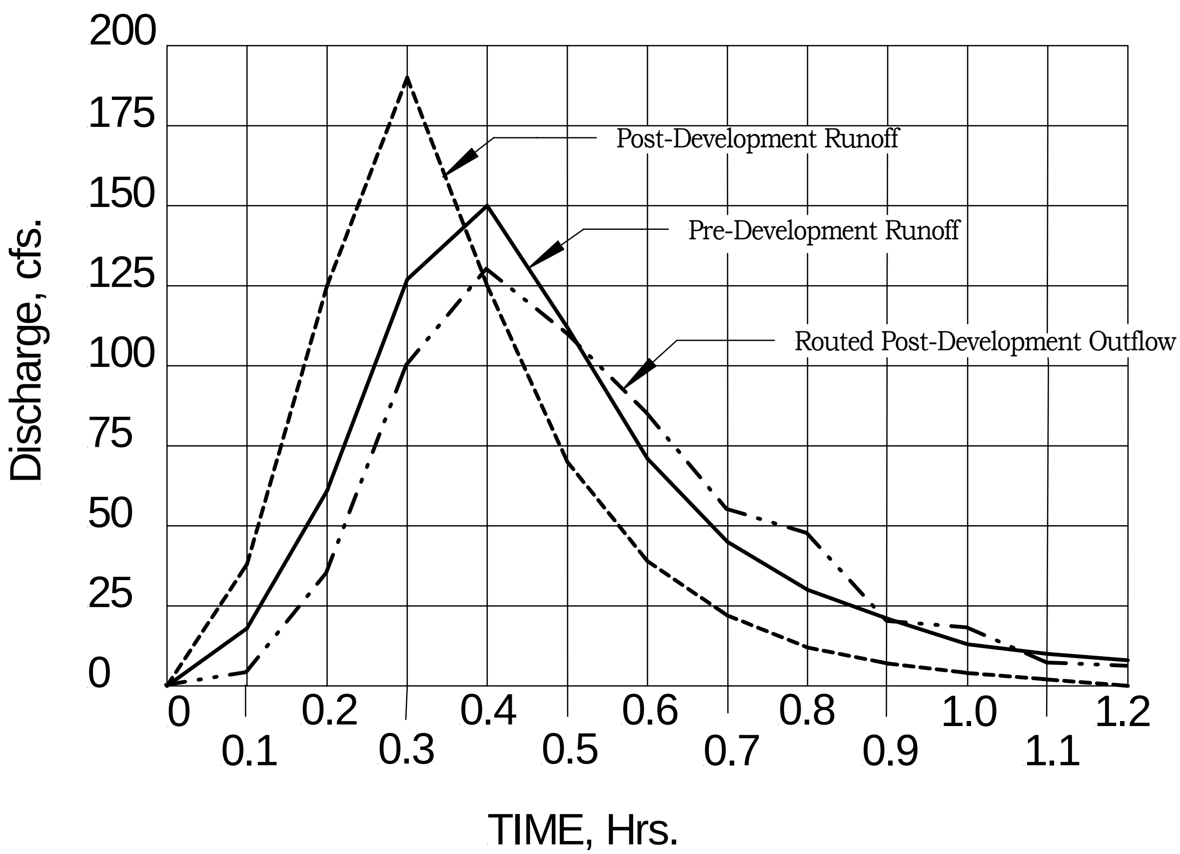


Figure 15-12 Runoff Hydrographs

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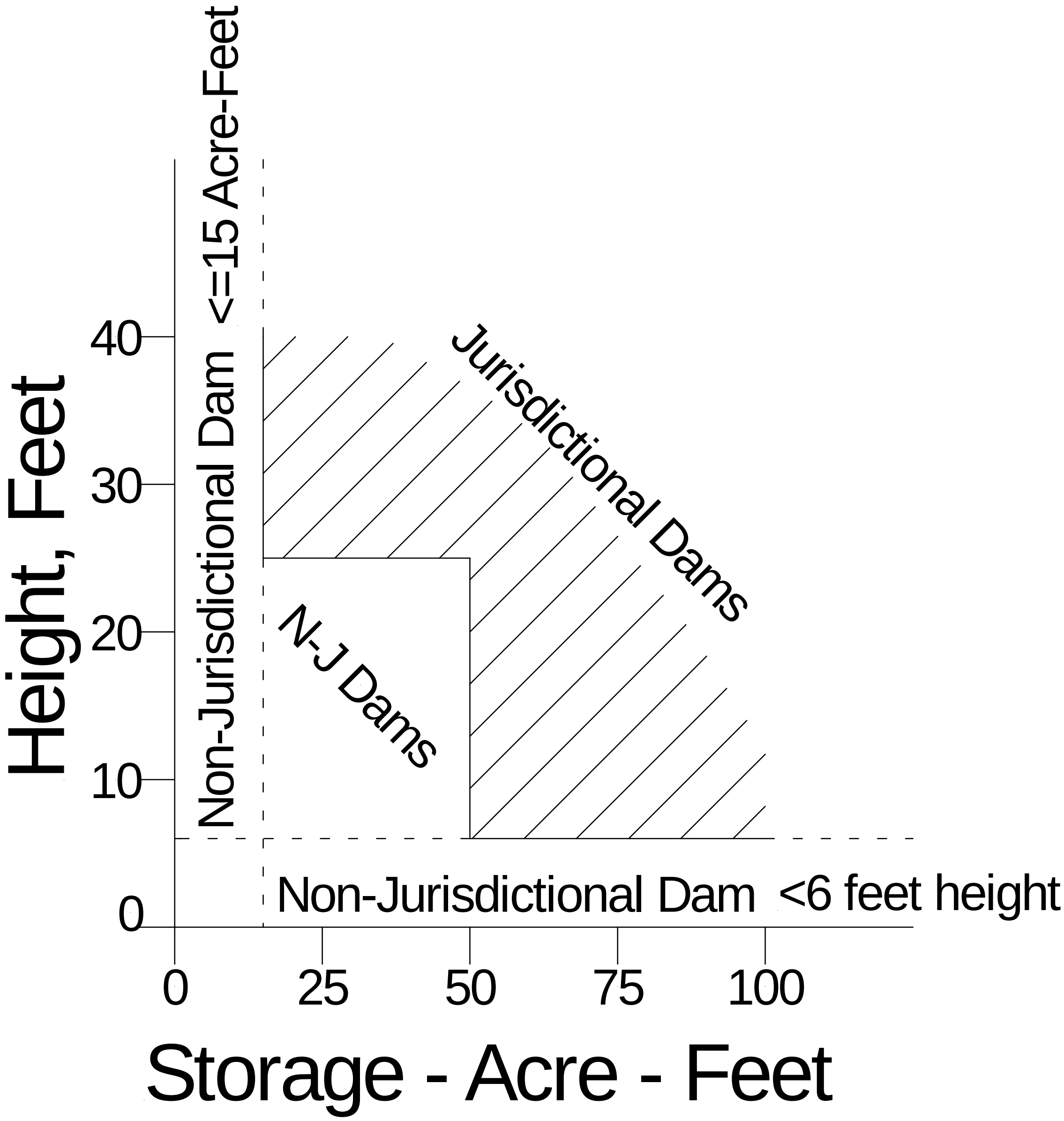
Appendix A ADWR Jurisdictional Dam Criteria



Appendix B HEC-1 Hydrograph for Rational Basin

There are times one will need a full 24 hour hydrograph for a basin that is analyzed using the Rational Method. The suggested procedure is described below. The procedure is a two-step process of first matching the volume of runoff and then matching the peak discharge. The procedure is demonstrated using the data from the Youngtown Basin – ADOT Hydrology Manual Page 3-19.

Youngtown Basin
Area = 0.13 sq. mi. = 83.2 Acres
Length = 4420 feet = 0.84 mi.
Urban watershed, K_b = 0.025 with ¼ acre lots
For 100 year rainfall, P1 = 2.53 in. C = 0.73

Assume slope = 1% = 52.8 ft/mi
$T_c = 11.4\ L^{0.5} K_b^{0.52} / S^{0.31} * i^{0.31}$
$T_c = 11.4\ (.84)^{0.5}(0.025)^{0.52}/(52.8)^{0.31} * i^{0.38}$
$T_c = 0.448/i^{0.38}$

Solving for T_c = 0.216 hr = 13 min., I = 6.8 in/hr.

Q = CiA = 0.73*6.8*83.2 = 413 cfs

Volume = $CP_{24}A$
Volume = $CP_{24}A$= 0.73*(3.8/12) *83.2 = 19.2 Ac-Ft

For HEC-1 with Tc = 0.216 hr,
$R = 0.37 * T_c^{1.11} * L^{0.8}/A^{0.57}$
$R = 0.37 * (0.216)^{1.11} * (0.84)^{0.8}/(0.13)^{0.57}$
R = 0.37*0.183*0.87/0.313 = 0.19

CP_{24}= 0.73*(3.8)=2.77
HEC-1 results:
Try 70% impervious
Runoff = 2.73 in, Qp = 280 cfs

Try 75% impervious
Runoff = 2.91 in, Qp = 296 cfs

Try 71% impervious
Runoff = 2.77 in, Qp = 283 cfs

Use 71% impervious and adjust T_c and R
Try T_c = 0.15 and R=0.14: Qp = 326 cfs
Try T_c = 0.10 and R=0.09: Qp = 390 cfs
Try T_c = 0.07 and R=0.06: Qp = 414 cfs, Use for hydrograph.