PART 2

Hydrology and Hydraulics (Pre-Rewrite Version)

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NOTE: **Revisions to these chapters occurring after January 1, 2012 are not included in this document, but have been included in the 2013 *Indiana Design Manual*. Users should consult Design Memoranda issued during 2012 for details related to revisions to this manual between January 1, 2012 and January 1, 2013.**

CHAPTER 201

Hydrology and Hydraulics (Pre-Rewrite Version)

Design Memorandum	Revision Date	Publication Date*	Sections Affected
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*Revisions will appear in the next published edition of the *Indiana Design Manual*.

TABLE OF CONTENTS

LIST OF FIGURES

Figure Title

CHAPTER TWENTY-EIGHT

GENERAL INFORMATION

In aggregate, Part IV discusses INDOT policies, practices, and procedures for performing hydraulic analyses on each project which is the responsibility of the Department. Specifically, this Chapter discusses general information on INDOT hydraulic practices. This includes defining the responsibilities of Department entities in hydraulic analyses, discussing the coordination between the *Indiana Design Manual* and the AASHTO *Model Drainage Manual*, describing the basic legal authority for drainage, providing the documentation requirements for the hydraulic analyses, and discussing the Department's Pipe Classification System.

28-1.0 GENERAL

28-1.01 Introduction

For highway applications, hydraulics is the science of collecting, transporting, and disposing of surface water originating on or near the highway right of way or flowing in a stream crossing or bordering such right of way. Proper drainage control is one of the essential elements of highway construction. The cost for the adequate removal of surface water justifies a scientific approach for the design of a drainage facility. A large percent of highway construction costs is devoted to culverts, bridges, or other drainage structures.

Drainage design is a unique field of Civil Engineering because there are no definitive methods or rules that provide absolute answers to engineering questions. The hydraulics designer must rely on engineering judgment, experience, and common sense to achieve meaningful results. The many drainage design methodologies available to the designer are only tools to aid him or her in making judgments. The drainage designer must fully understand each method that is employed, especially its limitations. Only with understanding can the designer be assured of producing a reasonable hydraulics design.

For this *Manual*, highway drainage design will be confined to methods of preventing the accumulation and retention of water on and by the roadway through the following:

1. anticipating the amount and frequency of storm runoff;
2. determining natural points of concentration and discharge and other hydraulic controls;
3. providing the most efficient disposal facility consistent with cost, the importance of the road, maintenance, and legal obligations; and

4. removing detrimental amounts of subsurface water.

In hydraulic design, the basic objective is to protect the highway against damage from storm and subsurface waters considering the effect of the proposed improvement on traffic and property. Therefore, unless the State will benefit, no improvement in the drainage of areas outside the right of way is warranted.

28-1.02 Responsibilities

28-1.02(01) Department-Designed Project

For a project designed in-house by INDOT personnel, the following summarizes the responsibilities of Department entities.

1. Designer. For roadway-drainage appurtenances, the designer is responsible for the hydrologic or hydraulic analyses of an open channel or pavement-surface drainage.

   The designer is responsible for gathering all needed information for the hydraulic analysis of a culvert or storm drain.

2. Hydraulics Team. The Hydraulics Team is responsible for the hydrologic or hydraulic analyses of the following:

   a. bridge waterway opening (in coordination with the designer);
   b. culvert;
   c. urban area where the drainage basin exceeds 200 acres;
   d. closed drainage system; and
   e. as requested by the designer.

   The Hydraulics Team is responsible for developing criteria and policies for each hydraulic analysis performed by the Department. The Team does not check the analysis performed by the designer. However, the Team serves as a technical resource as needed.

28-1.02(02) Consultant-Designed Project

For a project designed by a consultant, the following summarizes the division of responsibilities.

1. Consultant. The consultant is responsible for the following:

a. all hydrologic and hydraulic computations for a local-transportation project; and

b. all hydrologic and hydraulic computations for an INDOT project related to each storm drain or culvert except that titled in the system as a small-structure replacement.

The consultant is responsible for performing its analyses consistent with the policies and criteria adopted by the Department. The consultant must submit its calculations to the INDOT project manager for the design file for the following:

a. culvert of 36 in. diameter or less, or deformed pipe of equivalent area; and

b. storm drain.

2. Hydraulics Unit. For roadway drainage appurtenances, the Hydraulics Team will perform the following:

   a. review the consultant's computations for a culverts of greater than 36 in. diameter or deformed pipe of equivalent area;

   b. review the consultant's computations for a project in an urban area where the drainage basin exceeds 200 acres; and

   c. perform computations for an INDOT project listed as a small-structure replacement.

   The Hydraulics Team does not review storm-drain calculations except for a project in an urban area where the drainage basin exceeds 200 acres.

   For a bridge waterway opening, the Hydraulics Team will perform the following:

a. review all bridge hydraulic computations performed by the consultant for a local-agency project;

b. review scour computations performed by the consultant for a local-agency project; and

c. perform all bridge hydraulic computations for an INDOT project.

28-1.02(03) Pipe-Classification System

Section 28-6.0 discusses the designer responsibilities specifically for the INDOT Pipe-Classification System.

28-2.0 AASHTO *MODEL DRAINAGE MANUAL*

The AASHTO Task Force on Hydrology and Hydraulics has produced the *Model Drainage Manual* (*MDM*). The *MDM* provides design theories, concepts, guidelines, criteria, policies, and procedures for use by the hydraulics designer. It has been prepared in a format suitable for direct use, with State-specific modifications, by INDOT.

This Part has been prepared based on the AASHTO *MDM*. Where practical, the text and graphics in the *MDM* have been incorporated into this *Manual* with modifications to reflect INDOT practice. The following summarizes the disposition of each chapter of the *MDM* herein.

1. Chapter One, Introduction. Chapter One is not incorporated herein.

2. Chapter Two, Legal Aspects. Chapter Two has been edited and incorporated into Section 28-3.0.

3. Chapter Three, Policy. Sections 3.1 and 3.2 have been edited and incorporated into Section 28-4.0.

4. Chapter Four, Documentation. Chapter Four has been edited and incorporated into Section 28-5.0.

5. Chapter Five, Planning and Location. This *Manual* references Chapter Five.

6. Chapter Six, Data Collection. Part III discusses data collection for a drainage survey based on this *MDM* Chapter.

7. Chapter Seven, Hydrology. Chapter Seven has been used as a resource for the development of *Manual* Chapter Twenty-nine.

8. Chapter Eight, Channels. Chapter Eight has been edited and incorporated into *Manual* Chapter Thirty.

9. Chapter Nine, Culverts. Chapter Nine has been edited and incorporated into *Manual* Chapter Thirty-one.

10. Chapter Ten, Bridges. Chapter Ten has been used as a resource for the development of *Manual* Chapters Thirty-two and Thirty-three.

11. Chapter Eleven, Energy Dissipaters. Chapter Eleven has been edited and incorporated into *Manual* Chapter Thirty-four.

12. Chapter Twelve, Storage Facilities. Chapter Twelve has been edited and incorporated into *Manual* Chapter Thirty-five.

13. Chapter Thirteen, Storm Drainage Systems. Chapter Thirteen has been edited and incorporated into *Manual* Chapter Thirty-six.

14. Chapter Fourteen, Pump Stations. This *Manual* references Chapter Fourteen.

15. Chapter Fifteen, Surface Water Environment. This *Manual* references Chapter Fifteen.

16. Chapter Sixteen, Erosion and Sediment Control. Chapter Sixteen has been used as a resource for the development of *Manual* Chapter Thirty-seven.

17. Chapter Seventeen, Bank Protection. Chapter Seventeen has been edited and incorporated into *Manual* Chapter Thirty-eight.

18. Chapter Eighteen, Coastal Zone. Chapter Eighteen is not applicable to Indiana.

19. Chapter Nineteen, Construction. This *Manual* references Chapter Nineteen.

20. Chapter Twenty, Maintenance of Drainage Facilities. This *Manual* references Chapter Twenty.

21. Chapter Twenty-one, Restoration. This *Manual* references Chapter Twenty-one.

28-3.0 LEGAL ASPECTS

28-3.01 Overview

The drainage laws and rules that are applicable to a highway facility are discussed in this Section. It should not be treated as a basis for legal advice or legal decisions. It is not a summary of all existing drainage laws and is not intended as a substitute for legal counsel.

The following generalizations can be made for drainage liability.

1. A goal in highway-drainage design should be to perpetuate natural drainage as practical.

2. The courts look with disfavor upon infliction of injury or damage that can reasonably be avoided by a prudent designer, including where some alteration in flow is legally permissible.

3. The laws relating to the liability of government entities are undergoing radical change, with a trend toward increased government liability.

The descending order to law supremacy is Federal, State, then 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. The following summarizes the role of drainage law at each level of government.

28-3.02 Federal Laws

28-3.02(01) General

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. Compilations of Federal Statutory Law, revised annually, are available in the United States Code (USC) and the United States Code Service (USCS). Compilations of Federal regulatory material, revised annually, are available in the Code of Federal Regulations (CFR).

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

1. flood insurance and construction in a flood-hazard area;
2. navigation and construction in a navigable waterway;
3. water-pollution control;
4. environmental protection; and
5. protection of fish and wildlife.

Federal agencies formulate and promulgate rules and regulations to implement these laws. A highway-hydraulics designer should attempt to remain informed on proposed and final regulations.

28-3.02(02) Navigable-Waters Regulations

The Congress of the United States is granted constitutional power to regulate Interstate commerce, including navigable waters. The Federal agencies which implement existing Federal regulations are as follows.

1. United States Coast Guard (USCG). The USCG has regulatory authority under the Rivers and Harbors Act of 1899, Section 9 (33 U.S.C. 401), to approve plans and issue permits for bridges and causeways across navigable rivers. FHWA has the responsibility to determine that a USCG permit is not required. The USCG has the responsibility for the following:

    a. to determine whether or not a USCG permit is required for the improvement or construction of a bridge over a navigable waterway except for the exemption exercised by FHWA; and

    b. to approve the bridge location, alignment, and appropriate navigational clearances for each bridge permit application.

2. Corps of Engineers. The U.S. Army 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 of the Rivers and Harbors Act of 1899, Section 10 (33 U.S.C. 403), which prohibits the alteration or obstruction of a navigable waterway with the excavation or deposition of fill material in such waterway. The Clean Water Act, Section 404 (33 U.S.C. 1344) prohibits the unauthorized discharge of dredged or fill material into waters of the United States, including navigable waters. 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 the construction of a structure. The Corps of Engineers will grant a Regional General Permit for certain categories of minor activities involving discharge of fill material. Otherwise, an Individual 404 Permit is required.

3. Federal Highway Administration. The Federal Highway Administration has the authority to implement the Section 404 Permit Program (Clean Water Act of 1977) for each Federal-aid highway project processed under a 23 CFR 771.115 (b) categorical exclusion. This permit is granted for a project where the activity, work, or discharge is categorically excluded from environmental documentation because such activity does not have an individual or cumulative significant effect on the human environment.

4. Environmental Protection Agency (EPA). The EPA is authorized to prohibit the use of an area as a disposal site if it is determined that the discharge of materials at the site will have an unacceptable adverse effect on municipal water supplies, shellfish beds, fishery areas, or wildlife or recreational areas per the Clean Water Act, Section 404(c) (33 U.S.C. 1344). Also EPA is authorized under the Clean Water Act, Section 402 (33 U.S.C. 1344)

to administer and issue a National Pollutant Elimination Discharge System (NPDES) Permit for a point-source discharge, provided prescribed conditions are satisfied.

See Chapter Nine for more information on permits.

28-3.02(03) 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 the following:

> *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 or 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.

28-3.02(04) National Flood Insurance Program (NFIP)

The Flood Disaster Protection Act of 1973 denies Federal financial assistance to flood-prone communities that fail to qualify for flood insurance. The Act requires communities to adopt certain land use controls to qualify for flood insurance. These requirements could impose restrictions on the construction of highways in floodplains or floodways in communities which have qualified for flood insurance. A floodway is that portion of the floodplain required to pass a flood that has a 1-percent chance (i.e., a 100-year flood) of occurring in a 1-year period without cumulatively increasing the water surface elevation more than the allowable backwater. See Chapter Thirty-two for specific INDOT / IDNR criteria.

1. Flood Insurance. The National Flood Insurance Act of 1968 requires that communities adopt adequate land use and control measures to qualify for insurance. Federal criteria promulgated to implement this requirement include the following which can affect certain highways.

   In a riverine situation, the community must require that, until a floodway has been designated, no use, including landfill, may be permitted within the floodplain area having 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, this will not increase the water-surface elevation of the 100-year flood more than the allowable backwater at a given location within the community.

   After the floodplain area having 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. The floodway will convey the 100-year flood without increasing the water surface elevation of the flood more than the allowable backwater at a given point. It will prohibit, within the designated floodway, fill, encroachments, or new construction and substantial improvements of existing structures which can result in an increase in flood height within the community during the occurrence of the 100-year flood discharge.

   See Chapter Thirty-two for specific INDOT / IDNR criteria.

   The local community with land-use jurisdiction 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 action involving a regulatory floodway. The community, by necessity, must submit proposals to the Federal Emergency Management Agency (FEMA) for amendments to NFIP ordinances and maps in that community if necessary. 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.

2. 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. The types of NFIP maps that are published are as follows:

   a. Flood Hazard Boundary Map (FHBM);
   b. Flood Boundary and Floodway Map (FBFM); and
   c. Flood Insurance Rate Map (FIRM).

A FHBM is not based on a detailed hydraulic study and, therefore, the floodplain boundaries shown are approximate. A FBFM is 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 in the form of computer input data records for calculating the water-surface profile. The FIRM is produced at the same time using the same hydraulic model and has appropriate rate zones and base flood elevations added.

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

28-3.03 State Drainage Law

28-3.03(01) Types

State drainage law is derived from two sources as follows.

1. Common 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.

2. Statutory Law. Statutory laws of drainage are enacted by legislatures to enlarge, modify, clarify, or change the common law applicable to a particular drainage condition. This type of law is derived from constitutions, statutes, ordinances, and codes.

The common-law rules of drainage predominate unless they have been enlarged or superseded by statutory law.

28-3.03(02) Classification of Waters

The first step in the evaluation of a drainage problem is to classify the water as surface water, stream water, floodwater, or groundwater. 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. The following definitions apply.

1. Surface Water. Surface water is that which has been precipitated on the land from the sky or forced to the surface from a spring and which has then spread over the surface of the ground without being collected into a definite body or channel.

2. Stream Water. Stream water is former surface or groundwater which has entered and now flows 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. Legally, a watercourse refers to a definite channel with bed and banks within which water flows either continuously or intermittently.

3. Floodwater. Floodwater is former stream water which has escaped from a watercourse (and its overflow channels) and flows or stands over adjoining land. It remains a floodwater until it disappears from the surface as a result of infiltration or evaporation, or return to a natural watercourse.

4. Groundwater. In legal considerations, groundwater is either percolating water or an underground stream. The term percolating water includes all waters which pass through the ground beneath the surface of the earth without a definite channel. All underground waters are presumed to be percolating and, to remove them from the percolating class, the existence and course of a permanent channel must be clearly shown. An underground stream is water passing through the ground beneath the surface in a permanent, distinct, well-defined channel.

28-3.04 State Water Rules

28-3.04(01) Basic Concepts

Two rules have been developed by the courts regarding the disposition of surface waters. One is the civil-law rule of natural drainage. The other is the common-enemy doctrine. Much of the law regarding stream water is founded on a common-law maxim that states that water runs and ought to run as it is by natural law accustomed to run. Thus, an interference with the flow of a natural watercourse to the injury or damage of another will result in liability. However, there are qualifications as follows.

1. In common law, a floodwater is treated as a common enemy of all people, lands, and property attacked or threatened by it.

2. 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.

28-3.04(02) 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. The State has modified this rule so that the owner of upper lands has an easement over lower lands for drainage of surface water, 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. The following also applies.

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

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

    a. Was there reasonable necessity for the actor to alter the drainage to make use of his or her land?

    b. Was the alteration done in a reasonable manner?

    c. Does the utility of the actor's conduct reasonably outweigh the gravity of harm to others?

28-3.04(03) Stream water

Where a natural watercourse is unquestioned in fact, permanence, and stability, there is little difficulty in application of the rule. A highway crosses channel on a bridge or culvert, with some constriction of the width of the channel and obstruction by the 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 injuries or damages suffered.

Surface water from a highway is often discharged into the most convenient watercourse. The right is unquestioned if the water was naturally tributary to the watercourse and unchallenged if the watercourse has adequate capacity. However, if all or part of the surface water has been diverted from another watershed to a small watercourse, a downstream owner may complain and recover for ensuing damage.

28-3.04(04) Floodwater

Considering floodwater as a common enemy permits all affected landowners, including owners of highways, to act in a 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 the land of another without penalty, by gravity or pumping, by diverting dikes or ditches, or by other reasonable means.

The test of reasonableness has frequently been applied, and liability can result where unnecessary damage is caused. The highway designer should make provision for overflow in an area where it is foreseeable that it will occur. There is a definite risk of liability if such water is impounded on an upstream owner or, worse yet, is diverted into an area where it would not otherwise have gone. To label water as floodwater does not mean that it 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 or her own right and a reasonable use of his property in view of the similar right of his or her neighbors.

Although *reasonable* may be interpreted somewhat differently for each situation, it means that a landowner can utilize subsurface water on his or her property for the benefit of agriculture, manufacturing, irrigation, etc., pursuant to the reasonable development of the property, although such action may interfere with the underground waters of neighboring proprietors. However, it does preclude the withdrawal of underground waters for distribution or sale for uses not connected with beneficial ownership or enjoyment of the land from whence they were taken.

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

28-3.05 Statutory Law

The inadequacies of the common law or court-made law of drainage has led to a gradual enlargement and modification of the common-law rules by legislative mandate. If the common-law rules have been enlarged or superseded by statutory law, the statute prevails. Statutes have been enacted that affect drainage as described below.

28-3.05(01) Eminent Domain

In the absence of an existing right, a public agency 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 a public agency to take private property for public use.

The State constitution grants the right of eminent domain which allows the taking of property for public purposes, including the development of a watercourse or watershed area. However, if a property is taken under eminent domain, the private landowner must be compensated for his or her loss.

28-3.05(02) Water Right

The water right which attaches to a watercourse is a right to the use of the flow, and 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. The following applies.

1. Riparian Doctrine. Under the riparian doctrine, lands contiguous to a watercourse has prior claim to waters of the stream solely by reason of location and regardless of the relative productive capacities of riparian and non-riparian lands.

2. Doctrine of Prior Appropriation. The essence of this doctrine is the exclusive right to divert water from a source if 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.

A highway designer must consider that proposed work in the vicinity of a stream should not impair either the quality or quantity of flow of a water right to the stream.

28-3.06 Local Laws and Applications

Each local government (city, town, county, improvement district) has 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. The State is not legally required to comply with local ordinances except where compliance is required by specific

State statute. However, INDOT will, as practical, satisfy local ordinances as a courtesy, especially if it can be done without imposing a burden on the State.

A municipality is treated as a private party in State drainage matters. A municipality undertaking a public improvement is liable like an individual for damage resulting from negligence or an omission of duty. A municipality is under no legal duty to construct drainage improvements unless public improvements necessitate drainage (e.g., where street construction accelerates or alters storm runoff). A municipality is not liable for adoption or selection of a defective plan of drainage.

A municipality can be held liable for negligent construction of drainage improvements, for negligent maintenance and repair of drainage improvements, or if it fails to provide a proper outlet for drainage improvements. In the absence of negligence, a municipality will not be held liable for increased runoff occasioned by the necessary and desirable construction of storm drains, nor will a municipality be held liable for damages caused by overflow of its storm drains occasioned by extraordinary, unforeseeable rains or floods. Municipal liability will occur where a municipality does the following:

1. collects surface water and casts it in a body onto private property where it did not formerly flow;

2. diverts, by means of artificial drains, surface water from the course it would otherwise have taken, and casts it into a body large enough to do substantial injury on private land where, but for the artificial storm drain, it would not go; and

3. 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.

28-3.07 Legal Drains

Most counties have established a system of legal drains which are maintained by the county surveyor. State law grants the counties certain privileges where a project impacts an established legal drain. More detailed information on legal-drain regulations can be found in IC 36-2-12-15 and IC 36-9-27.

At the initiation of a bridge or road project, the designer should contact the appropriate county surveyor’s office to determine if an affected waterway is a legal drain. If so, the designer should invite the county surveyor to all field checks. This will provide an opportunity for the surveyor to express concerns and provide comments on the project. The designer should also request available information regarding legal flow lines or other requirements. This information must be included with the grade review plans. If not, include a note on the plans stating that the waterway is not a legal drain.

28-4.0 POLICY

28-4.01 Introduction

28-4.01(01) Purpose

Drainage concerns are one of the most important aspects of highway design and construction. The purpose of this Section is to outline specific policies that guide and determine the variables which influence drainage design.

28-4.01(02) Policy vs. Criteria

Policy and criteria statements are closely related. Criteria are INDOT's numerical or specific guidance which is founded in broad policy statements. For this *Manual*, the following definitions apply.

1. Policy. A definite course of action or method of action selected to guide and determine present and future decisions.

2. Design Criteria. The standards by which a policy is implemented or placed in action.

Thus, design criteria are needed for design; policy statements are not. The following is an example of a policy statement.

> *The designer will size the drainage structure to accommodate a flood compatible with the projected traffic volume.*

The following is an example of a design criterion.

> *For projected traffic volume less than or equal to 750 vehicles per day, each drainage structures should be designed for a 10-year flood (exceedance probability of 10%). For projected traffic volume greater than 750 vehicles per day, each drainage structure should be designed for a 25-year flood (exceedance probability of 4%).*

The following provides information on the hydraulic design of a drainage structure and related Federal, State, and local policies. Some of the following outline the relevant policies (with references indicating where details can be obtained). Some of the following state the policies and provide detailed information.

28-4.02 General Hydraulic Design Policies

28-4.02(01) Introduction

An adequate drainage structure is defined as one which satisfies the following:

1. the design of the structure satisfies or exceeds INDOT standard engineering practice; and

2. the design is consistent with what a reasonably competent and prudent designer will do under similar circumstances.

Hydrologic and hydraulic analyses, and engineering evaluation of selected alternatives, are conducted as a part of the design of a highway-drainage structure and serve as a means of achieving an adequate drainage design.

These studies are discussed further below.

28-4.02(02) Hydrologic Analysis

Present state-of-practice formulas and models for estimating flood flow are based on statistical analyses of rainfall and runoff records. The designer should select appropriate hydrologic estimating procedures and obtain runoff data where available for purposes of evaluation, calibration, and determination of the predicted value of the desired flood frequencies. The predicted value of the flood flow represents the designer’s best estimate, with varying degrees of error. The expected magnitude of this variation can be determined for some formulas or models as a part of the hydrologic design procedure.

28-4.02(03) 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 flow 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 over a range of flows.

28-4.02(04) Engineering Evaluation

The final step in the design process is the engineering evaluation of the preliminary designs and approval of the selected final design. This process involves consideration and balancing of a number of factors. Some of these factors are as follows:

1. legal considerations;
2. flood hazards to highway users or neighboring property owners;
3. hydraulic efficiencies;
4. costs;
5. environmental and social concerns; and
6. other site-specific concerns.

28-4.02(05) General Policies

Hydrologic and hydraulic analyses set forth the design-process representative of INDOT's standard engineering practice. Engineering evaluation outlines the approach to be followed by a reasonably competent and prudent designer in evaluating, selecting, and approving a final design. The following policies are made regarding this design process.

1. It is the designer's responsibility to provide an adequate drainage structure. The designer is not required to provide a structure that will accommodate all conceivable flood flows under all possible site conditions.

2. The detail of design studies should be commensurate with the risk associated with the encroachment and with other economic, engineering, social, or environmental concerns.

3. The overtopping or design flood may serve as criteria for evaluating the adequacy of a proposed design. The overtopping flood is the smallest recurrence-interval flood which will result in flow over the highway or other watershed boundary. The overtopping flood flow is the flow that overtops the highway or other watershed boundary limit. The design flood is the recurrence interval of the flood for which the drainage structure is sized to ensure that no traffic interruption or significant damage will result. The overtopping flood and the design flood may vary widely depending on the grade, alignment, and classification of the road and the characteristics of the watercourse and floodplain.

4. The predicted value of the 100-year or base flood serves as the present engineering standard for evaluating a flood hazard and as the basis for regulating a floodplain under the National Flood Insurance Program. The designer must make a professional judgment on the degree of risk that is tolerable for the base flood as required for each project.

5. The developed hydraulic performance curve of a drainage structure depicts the relationship between floodwater stage (or elevation) and flood-flow magnitude and frequency. The performance curve should include the 100-year flood. With the performance curve, the designer can evaluate the adequacy of the design for a range of flows and consider errors of estimate in the hydrologic estimating procedure. It is standard engineering practice to use the predicted value of the 100-year flood as the basis for evaluating a flood hazard. However, a flow larger than this value may be considered for a complex, high-risk, or unusual situation that requires special studies or risk analyses.

28-4.03 Water Main or Sanitary Sewer Construction

The INDOT *Standard Specifications* do not include most elements related to the construction of water main or sanitary sewer construction. However, many contracts, mostly those for local public agencies, include construction of these facilities. Therefore, special provisions should be developed and included each individual contract. The special provisions should adequately describe the work, determine material and construction requirements, and establish methods to measure and pay for the work.

There are many reference materials available from which information can be obtained to assist in the preparation of a required special provision. One such reference is the *Model Specifications For Water and Sewer Main Construction In Indiana*, prepared as a result of a joint effort between the Consulting Engineers of Indiana, Inc., Purdue University, and the Indiana Constructors Association. Other such reference documents have been prepared by individual public- and privately-owned entities.

The designer is free to develop a project-specific special provision or to incorporate a document into a contract by reference. The work must be completely addressed in the special provision. If a document is incorporated into a contract by reference, the special provision must indicate that the INDOT *Standard Specifications* govern if there is conflict with the referenced document. The designer should also be certain that the referenced document is readily available for use by the contractor or field personnel. The designer may indicate in a special provision how or where the document may be obtained.

28-5.0 DOCUMENTATION

28-5.01 Overview

28-5.01(01) Introduction

An important part of the design or analysis of a hydraulic facility is its documentation. Appropriate documentation of the design of a hydraulic facility is essential because of the following:

1. the importance of public safety;
2. justification of expenditure of public funds;
3. future reference by engineers (where improvements, changes, or rehabilitations are made to the highway facility);
4. information leading to the development of defense in litigation; and
5. public information.

It may be necessary to refer to plans, specifications, or analyses long after the actual construction has been completed. Documentation permits evaluation of the performance of a structure after a flood event to determine if the structure performed as anticipated or to establish the cause of unexpected behavior. If the structure fails, it is essential that contributing factors be identified so that recurring damage can be avoided.

28-5.01(02) Definition

The definition of hydrologic and hydraulic documentation as used in this Section is the compilation and preservation of the design and related details and all pertinent information on which the design and decisions were based. This should include the 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 or local residents who witnessed or had knowledge of an unusual event.

28-5.01(03) Purpose

This Section provides the documentation which should be included in the design file and on the construction plans. Although the Department’s documentation requirements for an existing or proposed drainage facility are similar, the data retained for an existing facility is often slightly different than that for a proposed facility, and these differences are discussed. This Section focuses on the documentation of the findings obtained in using the other chapters of this *Manual*. Thus, the designer should be familiar with all of the hydrologic and hydraulic design procedures associated with this Section. This Section identifies the Department’s system for organizing the documentation of a hydraulic design or review to provide as complete a history of the design process as is practical.

The purpose of providing documentation is to define the design procedure that was used and to show how the final design and decisions were selected. 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 seldom occurs, and documentation should be viewed as the record of reasonable and prudent design analysis based on the best available technology. Thus, good documentation can provide the following:

1. protecting the Department by proving that reasonable and prudent actions were, in fact, taken (such proof should certainly not increase the potential court award, and may decrease it by disproving any claims of negligence by the plaintiff);

2. identifying the situation at the time of design which might be very important if legal action occurs in the future;

3. documenting that rationally-accepted procedures and analyses were used at the time of the design which were commensurate with the perceived site importance and flood hazard (this should further disprove negligence claims);

4. providing a continuous site history to facilitate future reconstruction;

5. providing the file data necessary to quickly evaluate a future site problem that can occur during the facility's service life; and

6. expediting plan development by providing the reasons and rationale for specific design decisions.

28-5.01(04) Types

The types of documentation which should be considered are preconstruction, design, and construction or operation.

1. Preconstruction documentation should include the following, if available, or within the budgetary restraints of the project.

    a. aerial photographs;

    b. contour mapping;

c. watershed map or plan including flow directions, watershed boundaries, and watershed areas;

d. surveyed data reduced to include existing hydraulic facilities, existing controls, profiles (roadway, channel, drives), and cross sections (roadway, channel, faces of structure);

e. flood insurance studies and maps by FEMA;

f. Natural Resource Conservation Service soil maps;

g. field trip report(s) which may include videocassette recordings, audio tape recordings, still-camera photographs, movie-camera films, or written analysis of findings with sketches; and

h. reports from other agencies (local, State, or Federal), INDOT personnel, newspapers, and abutting property owners.

2. Design documentation should include all of the information used to justify the design, including the following:

   a. reports from other agencies;
   b. hydrological report;
   c. hydraulic report; and
   d. approvals.

3. Construction or operation documentation should include the following:

   a. plans;
   b. revisions;
   c. as-built plans and subsurface borings;
   d. photographs; and
   e. record of operation during flooding events, complaints, and resolutions.

The as-built plans should be prepared and maintained in a permanent file for each drainage structure to document subsurface foundation elements such as footing types and elevations, pile types, (driven) tip elevations, etc. There may be other information which should be included or may become evident as the design or investigation develops. This additional information shall be incorporated at the discretion of the designer.

28-5.01(05) Scheduling

Documentation should not be considered as occurring at specific times during the design, or as the final step in the process; this could be long after the final design is completed. Documentation should rather be an ongoing process and part of each step in the hydrologic and hydraulic analysis and design process. This will increase the accuracy of the documentation, provide data for future steps in the plan development process, and provide consistency in the design if different designers are involved at different times of the plan development process.

28-5.01(06) Responsibility

The designer should be responsible for determining which hydrologic analyses, hydraulic design, and related information should be documented during the plan-development process. The designer should make a determination that complete documentation has been achieved during the plan-development process which will include the final design. To assist in this determination, see Section 28-5.04.

28-5.02 Procedures

28-5.02(01) Introduction

A complete hydrologic and hydraulic design and analysis documentation file for each waterway encroachment or crossing should be maintained by the Hydraulics Team. If practical, this file should include the following:

1. identification and location of the facility;
2. photographs (ground and aerial);
3. hydrologic investigations;
4. drainage area maps, vicinity maps, and topographic maps;
5. contour maps;
6. interviews (local residents, adjacent property owners, or maintenance forces);
7. newspaper clippings;
8. design notes and correspondence relating to design decisions;
9. history of performance of the existing structure; and
10. assumptions.

The documentation file should include design and analysis data and information which influenced the facility design.

28-5.02(02) Practices

The following are the Department's practices related to documentation of hydrologic and hydraulic design and analysis.

1. Hydrologic and hydraulic data, preliminary calculations and analyses, and all related information used in developing conclusions and recommendations related to drainage requirements, including estimates of structure size and location, should be compiled in a documentation file.

2. The designer should document all design assumptions and selected criteria including the decisions related thereto.

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

4. Documentation should be organized to be as concise and complete as practicable so that future designers can understand what was done by predecessors.

5. Circumvent incriminating statements if possible by stating uncertainties in less than specific terms (e.g., *the culvert may back water* rather than *the culvert will back water*).

6. Provide all related references in the documentation file to include such things as published data and reports, memos and letters, or interviews. Include dates and signatures where appropriate.

7. Documentation should include data and information from the conceptual stage of project development through construction to provide successors with all information.

8. Documentation should be organized to logically lead the reader from past history through the problem background, into the findings, and through the performance.

9. A summary at the beginning of the documentation will provide an outline of the documentation file to assist the user in finding detailed information.

<u>28-5.03 Documentation Procedures</u>

28-5.03(01) Introduction

The following should be included in the documentation file. The intent is not to limit the data to only those items listed but, rather, to establish a minimum requirement consistent with the hydraulic design procedures as outlined in this *Manual*. 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 should appear in the documentation file. The designer should include the items listed below which are useful in understanding the analysis, design, findings, and final recommendations.

28-5.03(02) Hydrology

The items used in the design or analyses to be included in the documentation file are as follows:

1. contributing watershed-area size and identification of source (map name, etc.);
2. hydrologic discharge and hydrograph estimating method and findings;
3. IDNR Recommendation Letter (if an IDNR permit is required); and
4. method for estimating 500-year discharge (when applicable).

28-5.03(03) Bridge

The items to be included in the documentation file are as follows:

1. 100-year high-water elevation for natural, existing, and proposed conditions;
2. cross sections used in the design high-water elevation determination;
3. roughness coefficient (*n* value) assignments;
4. information on the method used for design high-water elevation determination;
5. observed high-water elevation, date, and discharge;
6. velocity measurements or estimates and locations (including both the through-bridge and channel velocity) for the 100-year flood;
7. calculated backwater, velocity, and scour for the 100-year and 500-year floods for scour evaluation;
8. magnitude and frequency of overtopping flood;

9. copies of computer analyses (existing and proposed) and disk containing all data files;

10. complete hydraulic study report;

11. economic analysis of design and alternatives;

12. bridge scour results;

13. roadway geometry (plan and profile); and

14. potential flood hazards to adjacent properties.

28-5.03(04) Culvert

The items to be included in the documentation file are as follows:

1. culvert performance curves;

2. allowable headwater elevation and basis for its selection;

3. cross sections used in the design high-water elevation determination;

4. roughness coefficient assignments (*n* values);

5. observed high-water elevation, date, and discharge;

6. stage-discharge curve for natural, existing, and proposed conditions to include the depth and velocity measurements or estimates and location for the 100-year flood;

7. performance curves showing the calculated backwater elevations, outlet velocity, and scour (if applicable), and the 100-year flood;

8. type of culvert-entrance condition;

9. culvert-outlet appurtenances and energy dissipation calculations and designs (if applicable);

10. copies of all computer analyses and a disk containing all data files;

11. roadway geometry (plan and profile); and

12. potential flood hazard to adjacent properties.

28-5.03(05) Open Channel

The items to be included in the documentation file are as follows:

1. stage-discharge curves for the 100-year and historical water-surface elevations;
2. cross sections used in the design water-surface determinations and their locations;
3. roughness coefficient assignments (*n* values);
4. information on the method used for design water-surface determinations;
5. observed high water elevation, date, and discharge;
6. channel velocity measurements or estimates and locations;
7. water-surface profiles through the reach for the 100-year or historical flood;
8. design or analysis of materials proposed for the channel bed and banks;
9. energy dissipation calculations and designs; and
10. copies of all computer analyses, including data disks.

28-5.03(06) Storm Drain

The items to be included in the documentation file are as follows:

1. computations for inlets and pipes, including hydraulic grade lines;

2. copies of the standard computation sheets shown in Chapter Thirty-six or the computer printout;

3. complete drainage area map;

4. design frequency (10-year gravity and 50-year pressure flow);

5. information concerning outfalls, existing storm drains, or other design considerations; and

6. a schematic indicating storm-drain system layout.

28-5.03(07) Pumping Station

The items to be included in the documentation file are as follows:

1. inflow design hydrograph from drainage area to pump,
2. flood-frequency curve for the attenuated peak discharge,
3. maximum allowable headwater elevation and related probable damage,
4. starting sequence and elevations,
5. sump dimensions,
6. available storage amounts,
7. pump size and operation,
8. pump calculations and design report, and
9. line storage and pit storage capacity.

28-5.03(08) Computer Files

The input data listing (hard copy and data disk) and output results of selected alternatives should be included in the documentation file and should be labeled.

28-5.04 Documentation Project Checklist

The Documentation Project Checklist is shown as Figure 28-5A. An editable version of this form may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/.

28-6.0 PIPE CLASSIFICATION SYSTEM

28-6.01 Introduction

INDOT has developed and implemented a Pipe Classification System which is intended to enhance the performance and longevity of pipe material used for a culvert, storm drain, underdrain, or other drainage facility. This is a comprehensive system which impacts all INDOT procedures and documents related to pipes, including the following:

1. the *Indiana Design Manual*,
2. the INDOT *Standard Specifications*, and
3. the INDOT *Standard Drawings*.

The information is segregated as follows:

1. Section 28-6.0 discusses information which applies to each pipe regardless of type of drainage appurtenance.

2. Chapter Twenty-nine has incorporated those elements of the System which apply to hydrology (e.g., choice of hydrologic method).

3. Chapter Thirty-one has incorporated those elements of the System which apply to a culvert (e.g., culvert design process, cover height).

4. Chapter Thirty-six has incorporated those elements of the System which apply to a storm drain or underdrain (e.g., minimum velocity, inlet spacing).

28-6.02 Description

The Pipe Classification System consists of the following.

1. Type 1 Pipe. Culvert under mainline and or public-road-approach pavement.
2. Type 2 Pipe. Storm-drain pipe.
3. Type 3 Pipe. Culvert under drive or field entrance.
4. Type 4 Pipe. Underdrain or drain tile.
5. Type 5 Pipe. Broken-back or other pipe installation which requires coupled pipe.

The INDOT *Standard Specifications* lists the materials that have been approved for each pipe type.

Although the Pipe Classification System serves as the foundation of drainage-structure design, other structure types are available for use as appropriate. One such category is referred to as Specialty Structure. A Specialty Structure should be used if the design process indicates that the materials included in the Pipe Classification System do not provide an adequate hydraulic structure. Specialty Structures include the following:

1. precast reinforced concrete box section;
2. precast reinforced concrete three-sided structure; or
3. structural-plate arch.

Specific-Application Structures are also not included in the Pipe Classification System but are available for use as appropriate. These structures include the following:

1. concrete-culvert extension;
2. pipe extension;
3. slotted-drain pipe or slotted-vane-drain pipe; and
4. end-bent-drain pipe.

28-6.03 Design Process

The drainage-structure design process, excluding a Specific-Application Structure, begins based on the assumption that the Pipe Classification System includes material that can provide a structure that satisfies all design requirements. A Specialty Structure must not be considered until it has been demonstrated that the appropriate System pipe type cannot provide a hydraulically-adequate structure.

Specific design requirements relative to culvert (Chapter Thirty-one) or storm-drain (Chapter Thirty-six) sizing are detailed elsewhere herein. The general concepts that apply to the implementation of the Pipe Classification System are discussed below.

1. Interior Designation. Sizing of a pipe-type structure is based interior designation. An interior designation of smooth or corrugated has been assigned to each type 1, 2, 3, or 5 pipe material. A type 4 pipe size is not determined by means of hydraulic calculations. Individual materials must not be considered during the sizing process for a pipe-type structure. More information on interior designation is included in Chapter Thirty-one.

2. Material. Each pipe type in the Pipe Classification System includes a list of approved materials. However, except for a type 4 pipe, the approval is general in nature. For example, an individual mainline culvert site may possess features that render the site unsuitable for some approved type 1 Pipe materials. Therefore, it is necessary to perform a Structure Site Analysis for each type 1, 2, 3, or 5 pipe structure. Features to be considered during the analysis include cover-height and service-life criteria (i.e., service life duration, abrasive or non-abrasive site designation, and structure pH). See Section 28-6.04 for additional information on Structure Site Analysis.

28-6.04 Structure Site Analysis

A Structure Site Analysis is required for each type 1, 2, 3, or 5 pipe structure. Unless otherwise specified, the analysis is not required for a type 4 pipe, Specialty, or Specific-Application Structure. The scope of the analysis is discussed below.

28-6.04(01) Cover

Cover is measured from the pipe crown to the bottom of the proposed pavement. The depth of aggregate base under HMA pavement or subbase under concrete pavement is included in the cover dimension. The allowable cover depth can vary based on pipe material. For a circular pipe, the minimum cover should be at least 1 ft., and the maximum cover should be at least 100 ft. For a deformed pipe with a corrugated interior designation, the minimum cover should be at least 1.5 ft. If these requirements cannot be satisfied, it is necessary to consider other structure types before continuing with the Structure Site Analysis.

The cover depth must be determined for a structure with a precast reinforced-concrete box section.

28-6.04(02) Pipe-Service-Life Duration

This indicates the desired length of service for the drainage structure. The duration is based on the functional classification of the mainline roadway. If the mainline roadway is a freeway or expressway, or is functionally classified as an arterial, the required service-life duration for each type 1, 2, 3, or 5 pipe structure is 75 years. If the mainline roadway is functionally classified as a collector or local road, the required service-life duration for each such structure is 50 years.

28-6.04(03) Abrasive or Non-Abrasive Site Designation

A site is considered abrasive if it is probable that runoff will transmit material which can damage the pipe. Each mainline culvert site or each site where a public-road-approach or drive culvert is installed in a natural channel is considered abrasive.

A storm-drain site or public-road-approach or drive culvert site on a constructed side-ditch line is considered non-abrasive. However, the designer must use judgment to confirm that abrasive elements are not likely to impact such a site. If the designer concludes that a storm-drain- or side-ditch-culvert site can have abrasive materials transported by runoff, an abrasive site designation must be assigned to each affected structure.

28-6.04(04) Structure pH

Acidic runoff may have contributed to service-life problems with a pipe structure. To mitigate these problems, the designer must determine a pH value for each type 1, 2, 3, or 5 pipe structure. The pH data may be provided in the Engineer's or Geotechnical Reports. The data should include the stream pH-test result for each type of existing structure as follows:

1. mainline culvert;
2. public-road-approach or drive culvert in a natural channel;
3. storm-drain-system outlet pipe; or
4. the most-downstream culvert on each constructed ditch line.

The designer will use the following guidelines to establish each proposed structure's pH value.

1. Culvert. Assign the data provided for each existing mainline culvert to the corresponding proposed pipe structure. Likewise, assign the data associated with each existing public-road-approach or drive culvert located in a natural channel to the corresponding proposed structure. Each proposed public-road-approach or drive culvert installed on a constructed ditch line should be assigned the report's pH value for the most-downstream culvert on the corresponding existing ditch line.

2. Storm Drain. If a proposed storm-drain system will replace an existing system, assign the pH value obtained at the existing system's outlet pipe to each pipe structure in the proposed system. If the proposed system is replacing an existing open-drainage system, apply the pH value collected at the most-downstream existing side-ditch culvert to each structure in the proposed system.

The final structure pH is the lowest of the following values.

1. Preliminary Field Check Plans pH Value. This value is obtained from one of the following sources.

    a. Engineer's Report.

    b. pH Testing. If pH data is not available from the Engineer's Report, the designer is required to perform pH testing of water samples taken at the structure. The scope of the testing required is below and is illustrated by the flowcharts included in the following figures.

    28-6B Structure pH Determination Procedure for Proposed Mainline Culvert or Other Culvert in Natural Channel (Area Where Map pH = 7.0)
    28-6C Structure pH Determination Procedure for Proposed Storm-Drain Structure (Area Where Map pH = 7.0)
    28-6D Structure pH Determination Procedure for Proposed Side-Ditch Culvert (Area Where Map pH = 7.0)
    28-6E Structure pH Determination Procedure for Proposed Mainline Culvert or Other Culvert in Natural Channel (Area Where Map pH < 7.0)
    28-6F Structure pH Determination Procedure for Proposed Storm-Drain Structure (Area Where Map pH < 7.0)
    28-6G Structure pH Determination Procedure for Proposed Side-Ditch Culvert (Area Where Map pH < 7.0)

c. pH Map. If the Engineer's Report does not provide structure pH data, and pH testing is not appropriate, Figure 28-6A, pH Map, is used to determine the Preliminary Field Check pH value.

2. <u>Final Check Prints pH Value</u>. This value is obtained from one of the following sources.

   a. Geotechnical Report.

   b. pH Testing. If a structure pH value is not available from the Geotechnical Report and testing is appropriate (see Item 1.b. above), pH testing of a water sample taken from the corresponding existing structure site is required.

   c. pH Map. Use of the pH map is appropriate only if a structure pH value is not available from the two sources listed above.

3. <u>Final Tracings pH Value</u>. If the pH values from Items 1 and 2 for a structure are not within 0.5 of each other, a third value must be obtained for comparison. The third value is obtained from one of these two sources.

   a. pH Testing. If pH testing is appropriate, testing of water samples at the corresponding existing structure is required.

   b. pH Map. If pH testing is not appropriate, the pH map is the appropriate source for the third pH value.

Before pH testing is performed, the project location must be determined from Figure 28-6A, pH Map. If the project is located in a county with a posted 7.0 pH value, the testing scope is as follows:

1. <u>Identify Structure Requiring Testing</u>. The structure type to be considered for testing is as follows:

   a. mainline culvert;
   b. public-road-approach or drive culvert located in a natural channel;
   c. outlet pipe of storm-drain system; or
   d. the most downstream culvert on a constructed ditch line.

2. <u>Structure Inspection</u>. The testing process begins by inspecting the structure. If an existing structure does not show signs of corrosion, pH testing is not required. If the structure shows signs of corrosion, a water sample at the structure must be obtained and the pH of the sample must be determined.

If the project is located in a county with a pH map value < 7.0, the structure-inspection step described in Item 2 does not apply. Each structure identified in Item 1 requires obtaining a water sample for pH determination.

The following apply to the determination of a structure pH value, regardless of the source of the data.

1. Maximum Structure pH Value. The pH value for a structure cannot exceed the map pH value for the project location. If the pH value obtained from a report on pH testing is greater than the map pH value, the obtained value is ignored and the map value is used for the structure.

2. Precision of pH Value. The pH value is expressed to the nearest 0.5. If a report or pH testing yields a value that is more precise, the structure pH is rounded to the next lower 0.5.

3. Lack of Sample Availability. If pH testing is required, but a sample is not available at a structure site, the structure pH value will equal the value for the nearest adjacent structure. If a water sample is not available at an appropriate structure within the project limits, the pH map value is used for all structures.

4. Storm-Drain-Structure pH Determination. The structure pH assigned to the outlet pipe of a storm-drain system is assigned to each structure in the proposed system.

5. Side-Ditch-Culvert Structure pH Determination. The structure pH assigned to the most downstream pipe in a segment of side ditch is assigned to each culvert installed in that ditch line segment.

28-6.05 Pipe-Material-Selection Process

The data collected during the Structure Site Analysis is used to determine which pipe materials are acceptable for installation at an individual structure site. A computer program has been developed to perform the required material selection for a type 1, 2, 3, or 5 pipe structure.

The input required for the Pipe-Material-Selection Software includes the following:

1. required pipe type;
2. required pipe-interior designation, if applicable (see Chapter Thirty-one);
3. pipe size;
4. cover;
5. required service-life duration;

6. abrasive or non-abrasive site designation; and
7. structure pH.

The software analyzes the input and lists all pipe materials that are acceptable for installation at each individual structure site.

For material selection, each corrugated-metal pipe's protective coating or invert treatment is considered to define a unique material. For example, an acceptable-materials list showing zinc-coated corrugated steel pipe and zinc-coated corrugated steel pipe with bituminous-paved invert is considered to include two materials.

The following apply to the performance of the Pipe-Material-Selection Process.

1. <u>Software Indicates No Acceptable Materials for Structure</u>. If this occurs, the cause is likely to be incorrect-input-data entry. If a review of the input reveals that there are no errors, the designer must contact the INDOT Hydraulics Team for additional instructions.

2. <u>Software Indicates Only One Acceptable Material for Structure</u>. By definition, a pipe-type designation indicates that a contractor may select from a list of materials that have been determined to be acceptable for an individual structure. If the list includes only one acceptable material, the pipe-type designation is meaningless. If this occurs, the structure cannot refer to a pipe type. See Section 28-6.08 for more information on contract document requirements for such a structure.

3. <u>Software Indicates Two or More Materials are Acceptable for Structure</u>. By definition, a pipe-type designation remains appropriate for this structure.

4. <u>Pipe-Extension Structure</u>. A pipe extension requires the selection of a specific material. If possible, the selected material should match the existing pipe material. However, the material thickness and coating combination or material-strength classification must satisfy the cover and service-life-criteria requirements. By definition, a pipe-extension structure is a structure that involves attaching a new pipe to an existing pipe.

5. <u>Selection of Corrugated-Metal Pipe Optimum Corrugation Profile</u>. The Pipe-Material-Selection Software may indicate that more than one corrugation-profile and material-thickness combination is acceptable for a structure. It is then necessary to determine the optimum corrugation profile. The procedure for determining the optimum corrugation profile is as follows.

    a. Select the Profile with the Minimum Thickness. If the acceptable corrugation profiles require different material thicknesses, select the profile with the minimum thickness.

b. Select the Smallest Profile. If all acceptable corrugation profiles require the same material thickness, select the smallest profile. By definition, a 2⅔" x ½" corrugation profile is considered smaller than a 3" x 1" profile.

28-6.06 Draintile Structure

If it is known that the proposed construction will require the removal of existing field tile, the drainage will be perpetuated in the following manner.

1. Tile Replacement Within Temporary Right of Way. Type 4 pipe is used to perpetuate the drainage. The pipe size will match the existing tile and must be perforated in accordance with the INDOT *Standard Specifications*.

2. Tile Outlet in Ditch Prior to Crossing Mainline Pavement. Type 4 non-perforated pipe and a 10-ft long segment of draintile terminal section are required between the right-of-way line and the proposed outlet. If necessary, a concrete collar is used to connect to the existing pipe at the right-of-way line, and a rodent screen is required at the terminal-section outlet. Revetment riprap or other gradation [as required to satisfy the clear-zone criteria (see Chapter Forty-nine)] is required between the tile outlet and the ditch flow line to prevent erosion.

3. Tile Outlet in Ditch After Crossing Mainline Pavement. Type 1 pipe is required between the right-of-way line and the proposed outlet. The concrete collar, rodent screen, and riprap requirements described in Item 2 above will apply to the type 1 pipe installation. The acceptable type 1 pipe materials must satisfy the cover and service-life criteria. The site is assumed to be non-abrasive and the map pH can be assigned to the structure.

4. Tile Outlet in Storm Drain System. Type 2 pipe is required between the right-of-way line and the outlet location. A concrete collar is required. The acceptable type 2 pipe materials must satisfy the cover and service-life criteria. The site is assumed to be non-abrasive, and the structure pH must match the value for the storm-drain structure that serves as the tile outlet.

5. Tile is Perpetuated Across Right of Way. Type 1 pipe is required from right-of-way line to right-of-way line. A concrete collar is required. The acceptable type 1 pipe materials must satisfy the cover and service-life criteria. The site is assumed to be non-abrasive, and the pH map value for the project location is assigned to the structure.

28-6.07 [blank section]

28-6.08 Contract Documents

Part II discusses the INDOT requirements on the preparation of contract documents (e.g., plans preparation, quantity estimate, cost estimate). This section provides additional information on contract-document preparation for drainage-structures requirements.

28-6.08(01) Plans Content

The following is necessary to incorporate drainage-structures information into a set of plans.

1. Typical Cross Sections Sheet. This is the appropriate location for details related to the installation of longitudinal underdrains.

2. Plan and Profile Sheet. This is the appropriate location for drainage-structure identification. Samples are as follows.

   a. 60 ft of 36-in. Pipe and 2 Pipe End Sections Required

   b. 60 ft of 36-in. Smooth Pipe and 2 Pipe End Sections, or 42-in. Corrugated Pipe Type 2 and 2 Concrete Anchors Required

   c. 60 ft of 36-in. Smooth Pipe and 2 Pipe End Sections Required

   d. Manhole Type C-4 and 300 ft of 18-in. Pipe Required.

   e. 60 ft of 12-in. Slotted Drain Pipe Required.

   f. 225 ft of 96” x 48” Precast Reinforced Concrete Box Section Required. Skew 30° Rt.

   Each culvert structure or storm-drain outlet-structure length will be expressed to the nearest 1 ft. Other storm-drain structure lengths are expressed to the nearest 3 in. with the measurement taken from outside face to outside face of adjacent manholes, inlets, catch basins, or similar structures.

   A structure that includes a pipe type or requires a specific pipe material is identified only as Pipe. A Specialty Structure or Specific-Application Structure must be identified as such.

3. Detail Sheet. All drainage-structure-related features that are not included on the *Standard Drawings* must be detailed here. The features that require such detailing include the concrete collar required to join existing and proposed pipes or Specialty-Structure backfill requirements.

4 Structure Data Sheet. The following apply to the preparation of this sheet.

   a. For a structure that refers to a pipe type, identify it in the Pipe Type column. The word *Pipe* is entered in the Description column.

   b. For a structure that requires a specific pipe material, an entry is not placed in the Pipe Type column. The word *Pipe* is entered in the Description column.

   c. A pipe-type structure that requires different sizes based on the interior designation requires separate rows of input data for each interior designation.

   d. A Specialty Structure or Specific-Application Structure is identified in the Description column. The identification for a pipe-extension structure is *Pipe Extension*. A concrete-culvert extension using precast reinforced-concrete box sections is identified with *Precast Reinforced-Concrete Box-Section Culvert Extension* in the Description column. No entry is made in the Pipe Type column.

5. Pipe Material Sheet. This is used to list the acceptable pipe materials for each pipe-type structure excluding type 4 pipe, specific-pipe-material structure, or pipe-extension structure.

6. Underdrain Table. This is used to summarize the complete underdrain design for the project. For more details regarding underdrain design procedures, see Chapter Fifty-two.

28-6.08(02) Backfill Material

See Section 17-2.09 for backfill-material requirements.

28-6.08(03) Pay Items

Sample drainage-system-related pay items are listed below.

1. Pipe, Type 1, Circular, 36 in. This is the pay-item-name format for a pipe-type structure. Its use indicates that at least two materials are acceptable for the structure. Also, for type 1, 3, or 5 pipe, the lack of an interior designation in the pay item name indicates that the

materials acceptable for installation include those with a smooth-interior designation and others with a corrugated-interior designation.

2. Pipe, Type 2, Circular, 12 in. This is the pay-item-name format for a storm-drain pipe structure. Since all type 2 pipe materials have a smooth-interior designation, the interior designation is not included in the pay item name.

3. Pipe, Type 1, Circular, 36 in. Smooth or 42 in., Corrugated. This pay-item name indicates that there are at least two materials acceptable for the structure. At least one has a smooth-interior designation and at least one has a corrugated-interior designation. The hydraulic design indicates that different-sized smooth- and corrugated-interior pipe sizes are required.

4. Pipe Type 3, Deformed, Min. Area = 1.8 ft^2, Corrugated. This pay-item name indicates that there are at least two materials acceptable for installation and all have a corrugated interior designation.

5. Pipe, RCP, Class II, $D_{0.3}$ = 50, 36 in. This pay-item name indicates that reinforced-concrete pipe of the specified strength classification is the only acceptable material for a new pipe structure. A reinforced-concrete-pipe pay-item name must include the required-strength classification and the D-load rating.

6. Pipe Extension, ZC CSP w/BPI, 0.10 in., 18 in. This pay-item name indicates that a zinc-coated corrugated steel pipe with a bituminous-paved invert and a material thickness of 0.10 in. is the only acceptable material for a structure that involves placing a new pipe on an end of an existing pipe. A corrugated-metal-pipe pay-item name must include the protective coating, required invert treatment, and the required material thickness.

7. Structure, Precast Reinforced-Concrete Three-Sided, 20 ft x 10 ft. This is the pay-item-name format for a Specialty Structure or Specific-Application Structure. The Specialty Structure or Specific-Application Structure must be identified in the pay item name.

8. Pipe End Section, 36 in. This is the pay-item-name format for an object placed on each end of a structure that has only one specified pipe size.

9. Concrete Anchor 42 in. or 48 in. This is the pay-item-name format for a structure with different-sized smooth-interior and corrugated-interior alternatives. A concrete anchor is required regardless of the pipe size that is actually installed.

10. Pipe End Section, 36 in., or Concrete Anchor, 42 in. This is the pay-item-name format for a structure with different-sized smooth-interior and corrugated-interior alternatives which each require a different object to be placed on each pipe end. Pipe end sections are

required if the 36-in. structure alternative is installed, and concrete anchors are required if the 42-in. structure alternative is installed.

28-6.09 English-to-Metric Conversion Information

Figures 28-6H through 28-6S provide conversion information related to drainage structures. These figures are provided to give the designer the ability to convert results from English-units design software, nomographs, etc., to the appropriate corresponding metric-units dimensions. The figure designations with their titles are listed below.

28-6H	Circular Smooth Pipe (Conversion from English-Units Designed Pipe Size to Metric-Units Pay-Item Pipe Size)
28-6 I	Circular Corrugated Pipe (Conversion from English-Units Designed Pipe Size to Metric-Units Pay-Item Pipe Size)
28-6J	Circular Corrugated Structural-Plate Pipe (Conversion from English-Units Designed Pipe Size to Metric-Units Pay-Item Pipe Size)
28-6K	Deformed Corrugated Pipe (Conversion from English-Units Designed Pipe Size to Metric-Units Pay-Item Pipe Size)
28-6L	Deformed Corrugated Structural-Plate Pipe (Conversion from English-Units Designed Pipe Size to Metric-Units Pay-Item Pipe Size)
28-6M	Deformed Smooth Pipe (Conversion from English-Units Designed Pipe Size to Metric-Units Pay-Item Pipe Size)
28-6N	Precast Reinforced-Concrete Box Sections (Conversion from English-Units Designed Pipe Size to Metric-Units Pay-Item Pipe Size)
28-6 O	Precast Reinforced-Concrete Three-Sided Structure (Conversion from English-Units Designed Pipe Size to Metric-Units Pay-Item Pipe Size)
28-6P	Non-Reinforced-Concrete Pipe Class 3 Wall Thickness
28-6Q	Reinforced-Concrete Pipe Wall Thickness
28-6R	Precast Reinforced-Concrete Box Section Wall Thickness
28-6S	Reinforced-Concrete Horizontal Elliptical Pipe Wall Thickness

Project Documentation Checklist

Route Project No. City/Town: County:

Description: Engineer:

Check Appropriate Items

REFERENCE DATA

Maps:
- ☐ USGS – Quad., Scale, Date
- ☐ USGS – Other
- ☐ Local Zoning Maps
- ☐ Flood Hazard Delineation (Quad.) for 100 Yr. Flood Plain Delineation (HUD)
- ☐ Local Land Use
- ☐ Soils Maps
- ☐ Geologic Maps
- ☐ Aerial Photos – Scale, Date

Studies By External Agencies:
- ☐ FEMA Flood Insurance Studies
- ☐ SCS Watershed Studies
- ☐ USGS Gages & Studies
- ☐ Interim Flood Plain Studies
- ☐ Water Resource Data
- ☐ Regional Planning Data
- ☐ Forestry Service
- ☐ Utility Company Plans

Studies By Internal Sources:
- ☐ Hydraulics Section Records
- ☐ Flood Records (High Water, Newspaper)

HYDROLOGY

Technical Resources:
- ☐ Part IV, *Indiana Design Manual*
- ☐ INDOT Directives
- ☐ Technical Library

Discharge Calculations:
- ☐ Drainage Areas
- ☐ Gaging Data – Regional Analysis
- ☐ Regression Equations (if no other method available)
- ☐ Area-Discharge Coordinated Curves
- ☐ Log-Pearson Type III Gage Rating (B 17B)
- ☐ Computer Programs: HYDRO, SCS (TR-20), HEC-1

HYDROLOGY (Cont'd.)

High Water Elevations:
- ☐ INDOT Survey
- ☐ External Sources
- ☐ Personal Reconnaissance

Flood History:
- ☐ External Sources
- ☐ Personal Reconnaissance
- ☐ Maintenance Records
- ☐ Photographs
- ☐ IDNR Historic Flood Profiles

HYDRAULIC DESIGN

Calibration of High Water Data:
- ☐ Discharge and Frequency of H.W. el.
- ☐ Influences Responsible for H.W. el.
- ☐ Analyze Hydraulic Performance of Existing Facility for 100 Yr.
- ☐ Analyze Hydraulic Performance of Proposed Facility

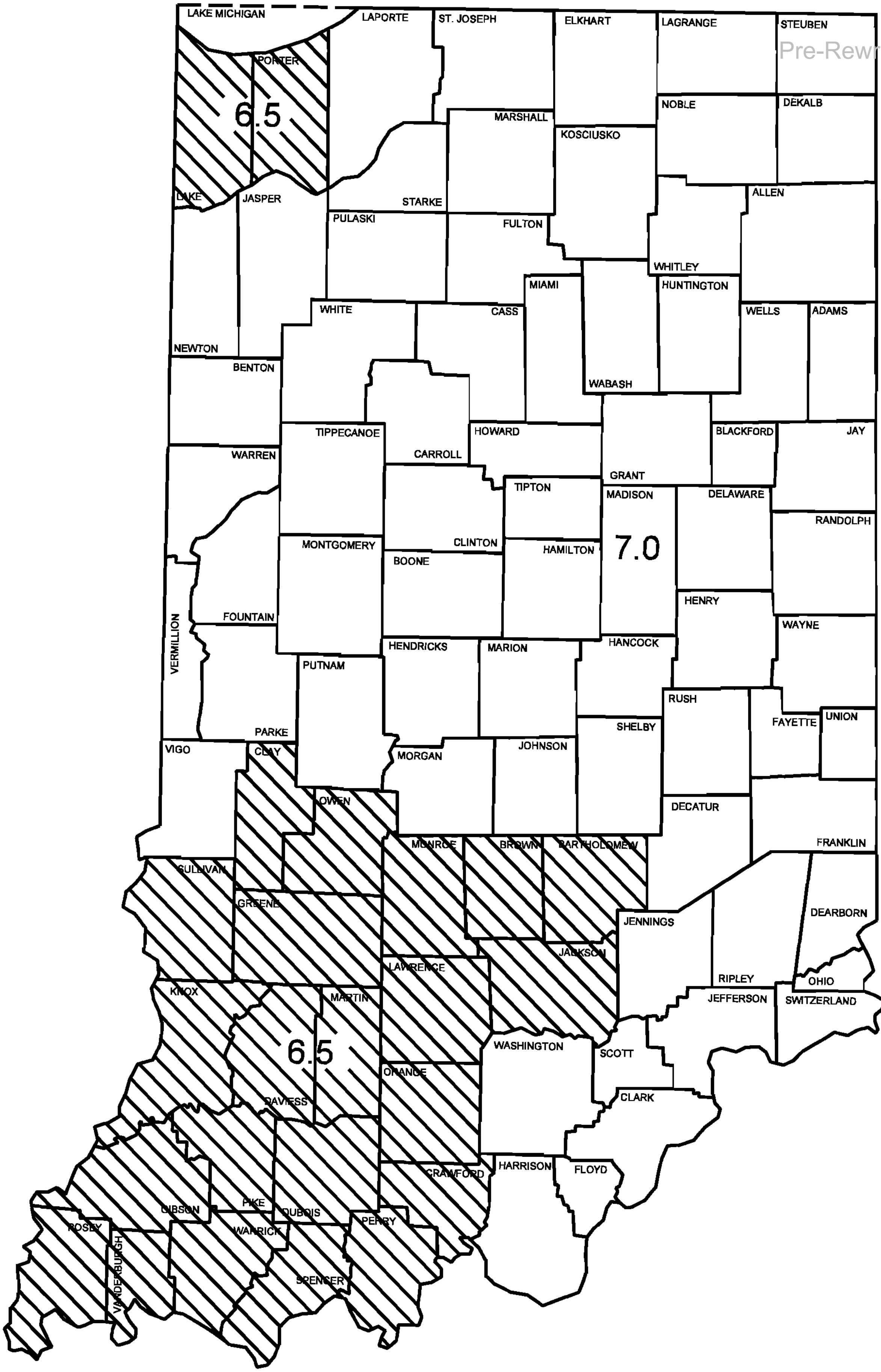
Design Appurtenances:
- ☐ Dissipators
- ☐ Riprap
- ☐ Erosion and Sediment Control

Technical Aids:
- ☐ *Indiana Design Manual*, Part IV
- ☐ INDOT and FHWA Directives
- ☐ Technical Library

Computer Programs:
- ☐ FESWMS-2DH
- ☐ HEC-RAS
- ☐ HEC-2
- ☐ HYDRAIN
- ☐ HY8
- ☐ WSPRO

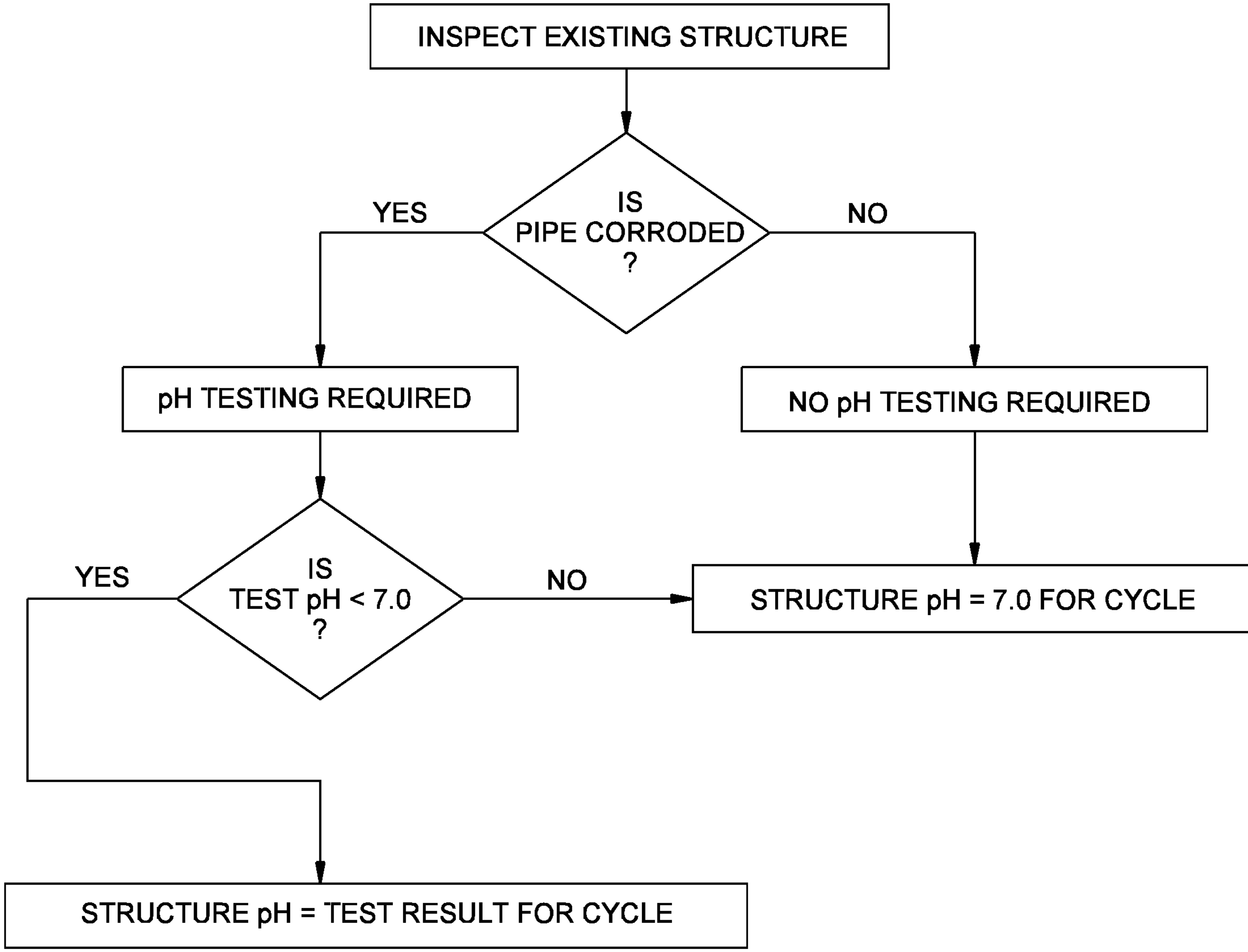
HYDROLOGIC-HYDRAULIC REPORTS

Data Reports:
- ☐ Environmental Reports
- ☐ Reconnaissance Report
- ☐ Hydraulic Recommendations from Engineer's Report
- ☐ INDOT Data



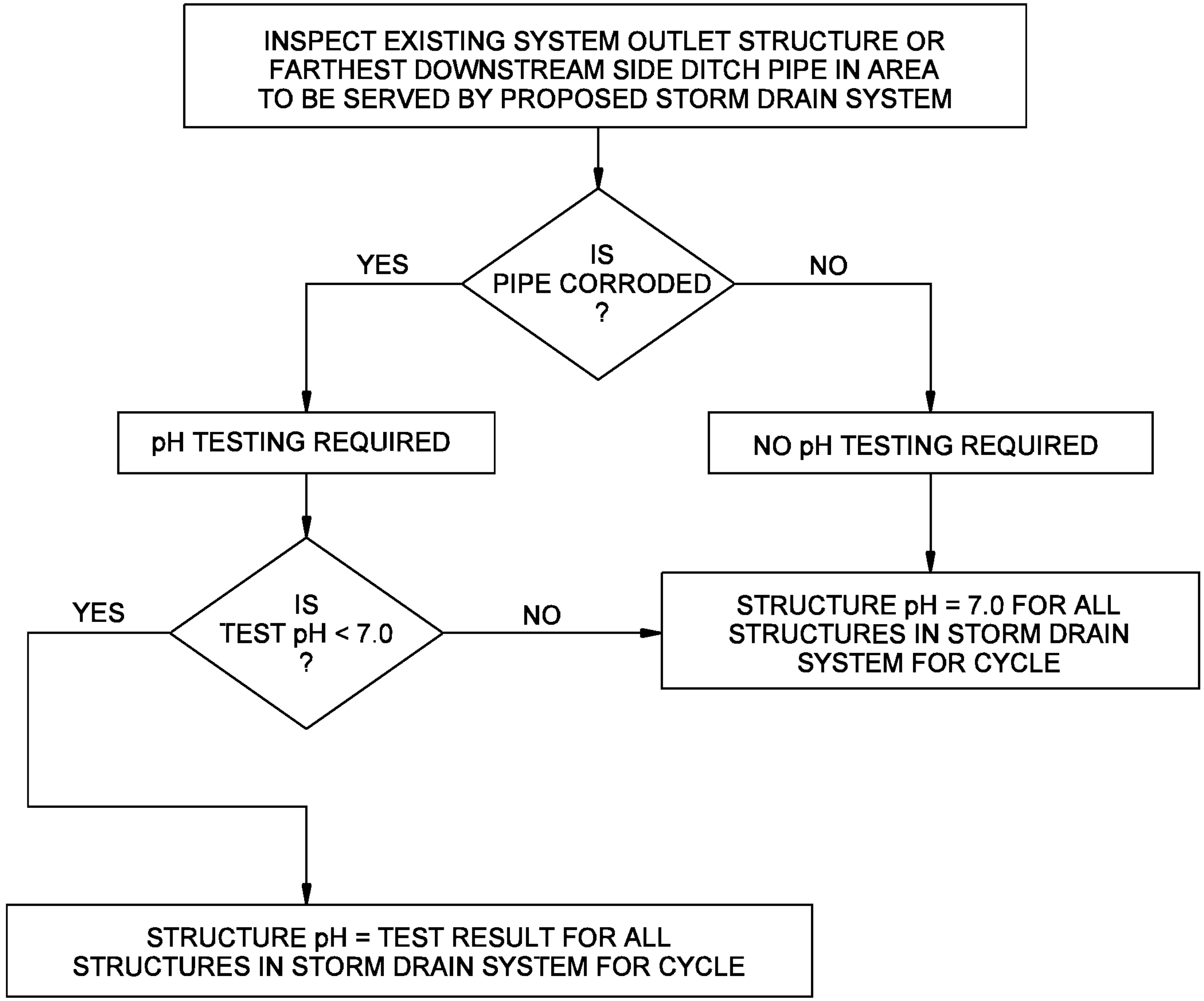
pH MAP

Figure 28-6A



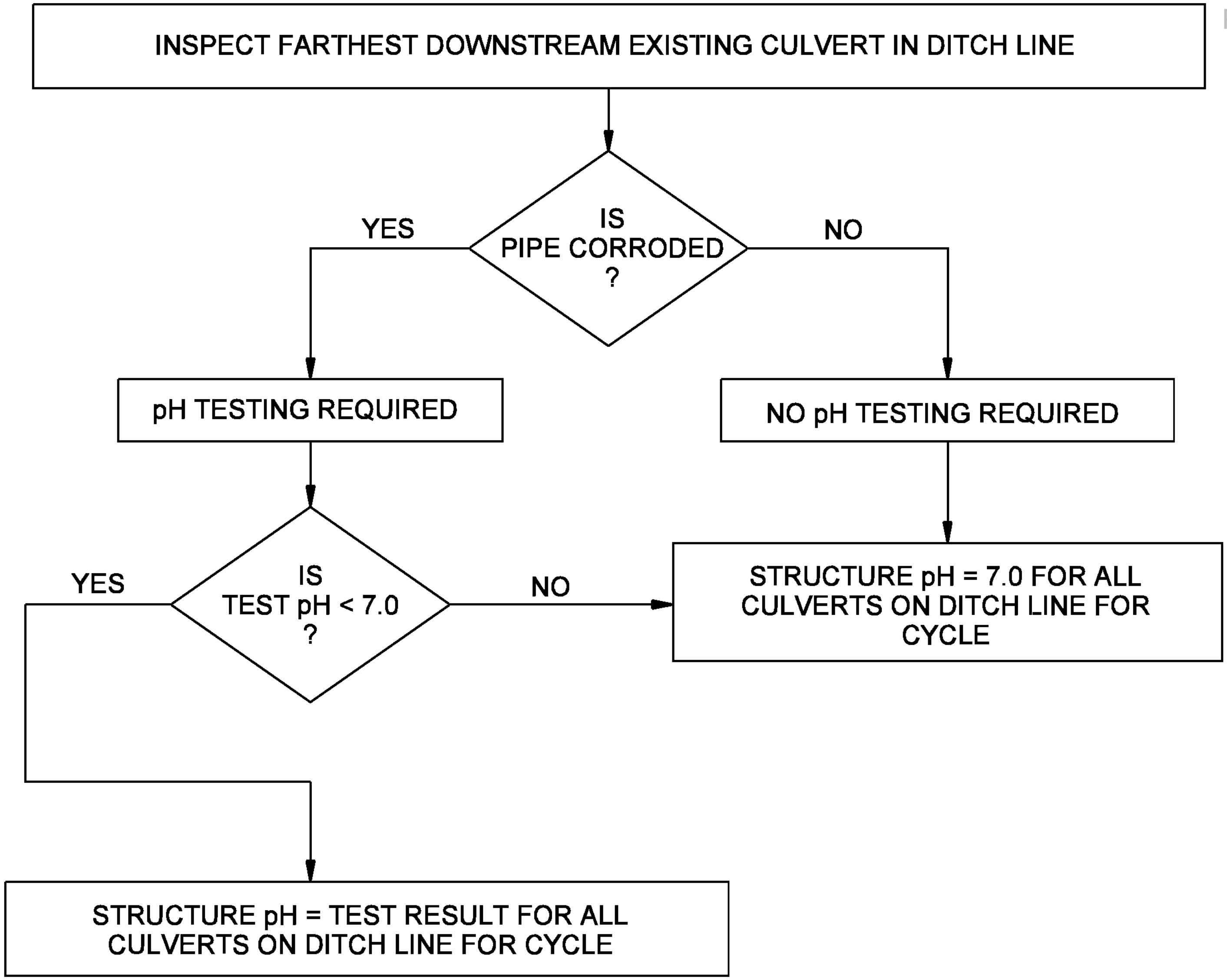
STRUCTURE pH DETERMINATION PROCEDURE
(Proposed Mainline Culverts & Other Culverts in Natural Channels)
(Project in Area Where Map pH = 7.0)

Figure 28-6B



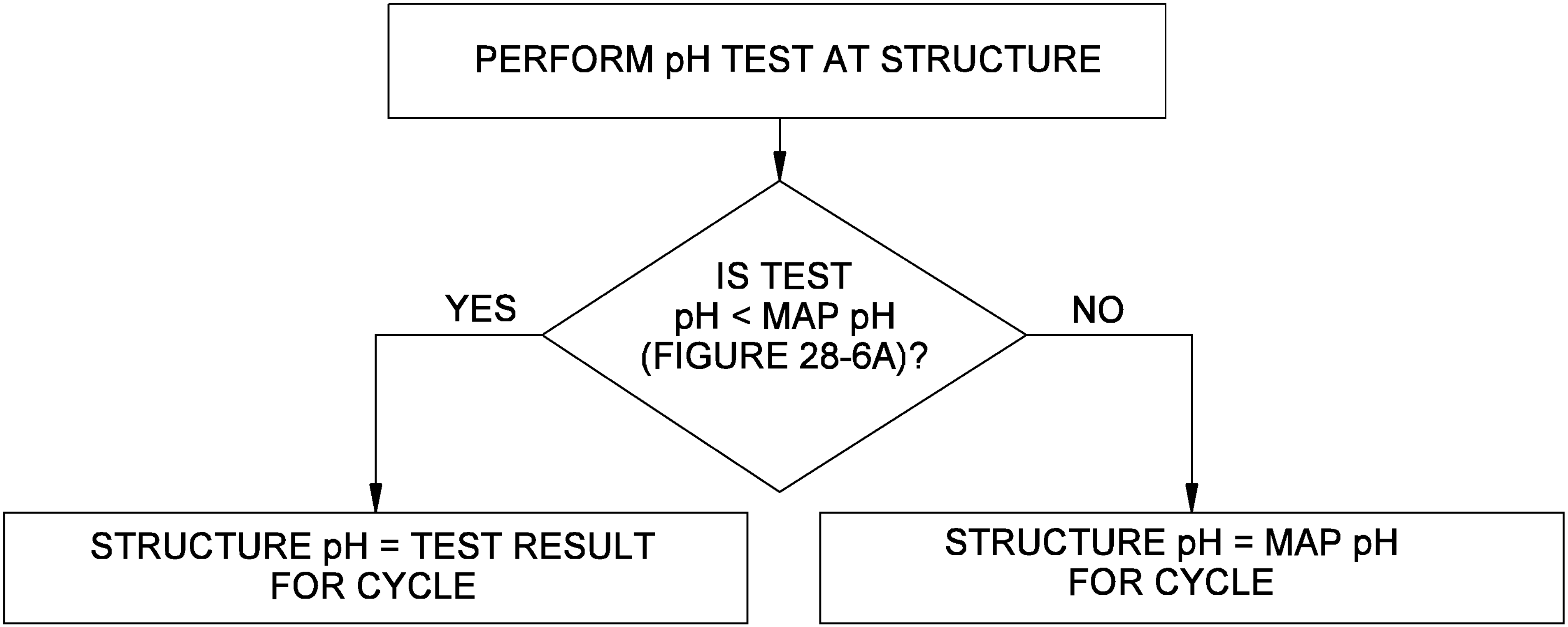
STRUCTURE pH DETERMINATION PROCEDURE
(Proposed Storm Drain Structures)
(Project in Area Where Map pH = 7.0)

Figure 28-6C



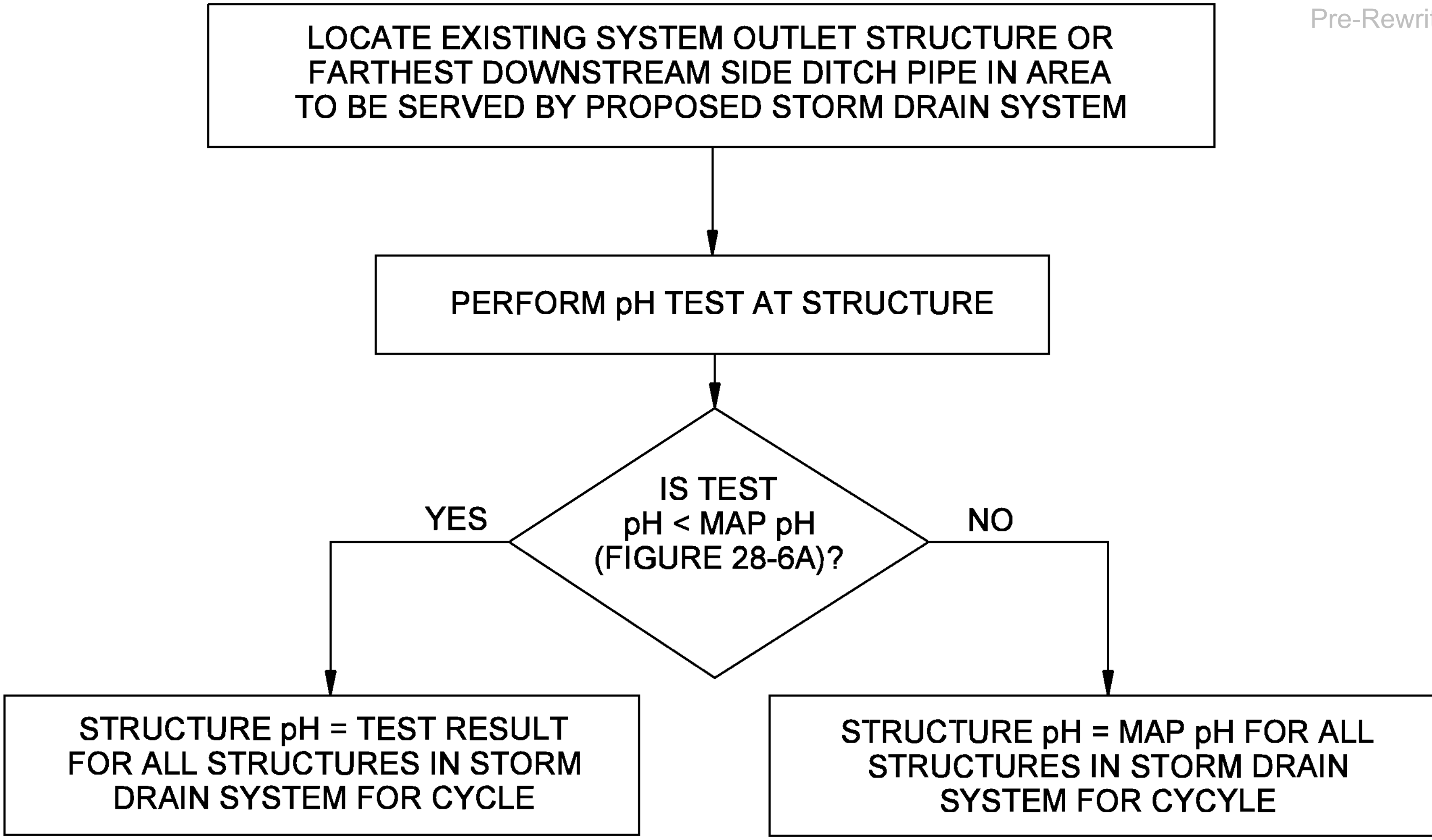
STRUCTURE pH DETERMINATION PROCEDURE
(Proposed Side Ditch Culverts)
(Project in Area Where Map pH = 7.0)

Figure 28-6D



STRUCTURE pH DETERMINATION PROCEDURE
(Proposed Mainline Culverts & Other Culverts in Natural Channels)
(Project in Area Where Map pH < 7.0)

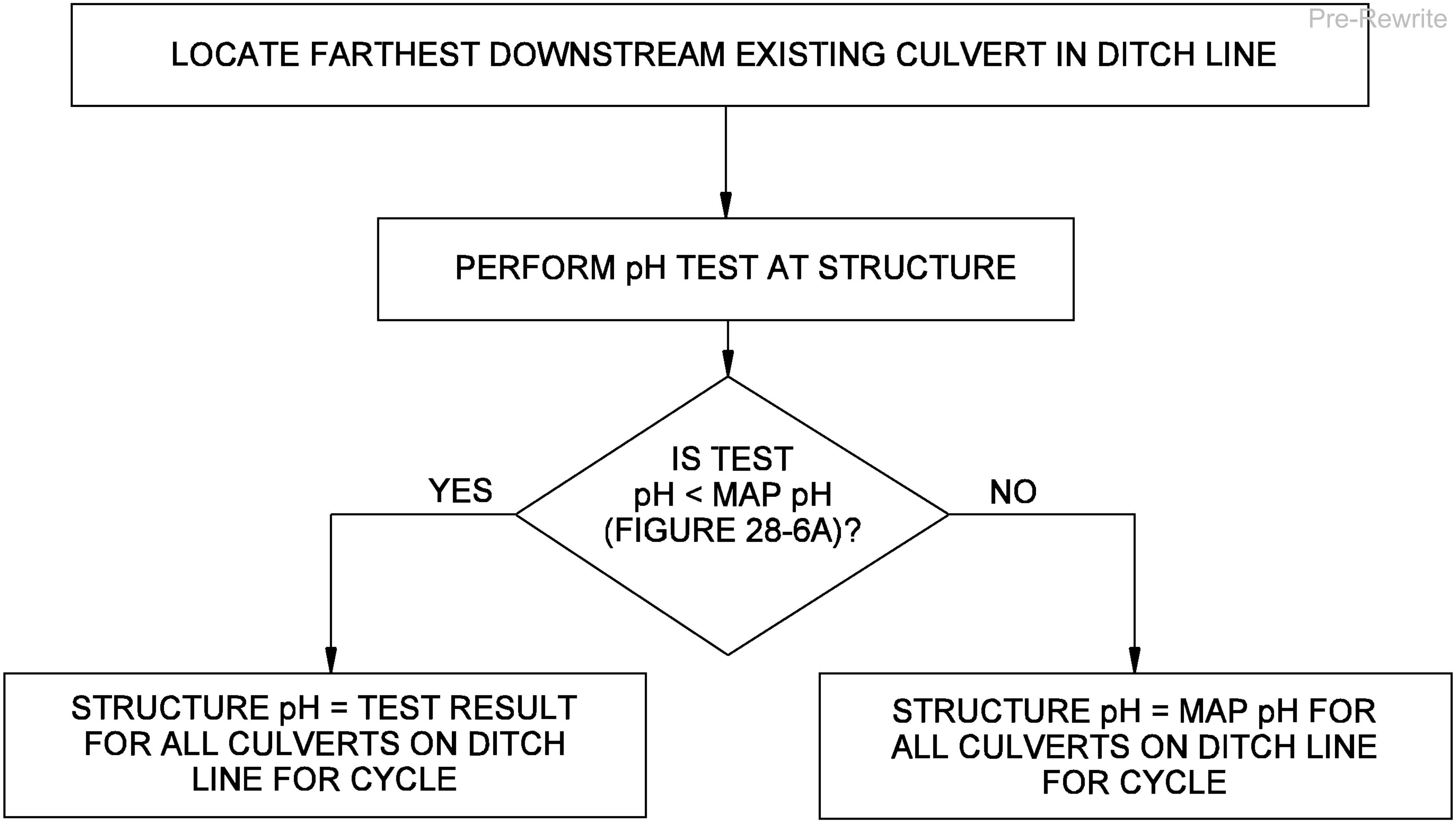
Figure 28-6E



STRUCTURE pH DETERMINATION PROCEDURE

(Proposed Storm Drain Structures)
(Project in Area Where Map pH < 7.0)

Figure 28-6F



STRUCTURE pH DETERMINATION PROCEDURE
(Proposed Side Ditch Culverts)
(Project in Area Where Map pH < 7.0)

Figure 28-6G

DESIGN DIAMETER (in.)	PAY-ITEM DIAMETER (mm)
12	300
15	375
18	450
21	525
24	600
27	675
30	750
33	825
36	900
42	1050
48	1200
54	1350
60	1500
66	1650
72	1800
78	1950
84	2100
90	2250
96	2400
102	2550
108	2700
114	2850
120	3000
126	3150
132	3300
138	3450
144	3600

CIRCULAR SMOOTH PIPE
(Conversion from English-Units-Design Pipe Size to Metric-Units Pay-Item Pipe Size)

Figure 28-6H

DESIGN DIAMETER (in.)	PAY-ITEM DIAMETER (mm)
12	300
15	375
18	450
21	525
24	600
27	675
30	750
33	825
36	900
42	1050
48	1200
54	1350
60	1500
66	1650
72	1800
78	1950
84	2100
90	2250
96	2400
102	2550
108	2700
114	2850
120	3000
126	3150
132	3300
138	3450
144	3600

CIRCULAR CORRUGATED PIPE
(Conversion from English-Units-Design Pipe Size to Metric-Units Pay-Item Pipe Size)

Figure 28-6I

DESIGN DIAMETER (ft-in.)	PAY-ITEM DIAMETER (mm)
5′-0″	1500
5′-6″	1655
6′-0″	1810
6′-6″	1965
7′-0″	2120
7′-6″	2275
8′-0″	2430
8′-6″	2585
9′-0″	2740
9′-6″	2895
10′-0″	3050
10′-6″	3205
11′-0″	3360
11′-6″	3515
12′-0″	3670
12′-6″	3825
13′-0″	3980
13′-6″	4135
14′-0″	4290
14′-6″	4445
15′-0″	4600
15′-6″	4755
16′-0″	4910
16′-6″	5065
17′-0″	5220
17′-6″	5375
18′-0″	5530
18′-6″	5685
19′-0″	5840
19′-6″	5995
20′-0″	6150
20′-6″	6305
21′-0″	6460

CIRCULAR CORRUGATED STRUCTURAL-PLATE PIPE
(Conversion from English-Units-Design Pipe Size to Metric-Units Pay-Item Pipe Size)

Figure 28-6J

DESIGN PIPE SIZE (in. x in.)	DESIGN AREA (ft^2)	PAY-ITEM AREA (m^2)
17″ x 13″	1.1	0.10
21″ x 15″	1.6	0.15
24″ x 18″	2.2	0.20
28″ x 20″	2.9	0.27
35″ x 24″	4.5	0.42
42″ x 29″	6.5	0.60
49″ x 33″	8.9	0.83
57″ x 38″	11.6	1.08
60″ x 46″	15.6	1.45
64″ x 43″	14.7	1.37
66″ x 51″	19.3	1.79
71″ x 47″	18.1	1.68
73″ x 55″	23.2	2.16
77″ x 52″	21.9	2.03
81″ x 59″	27.4	2.55
83″ x 57″	26.0	2.42
87″ x 63″	32.1	2.98
95″ x 67″	37.0	3.44
103″ x 71″	42.4	3.94
112″ x 75″	48.0	4.46
117″ x 79″	54.2	5.04
128″ x 83″	60.5	5.62
137″ x 87″	67.4	6.26
142″ x 91″	74.5	6.92

DEFORMED CORRUGATED PIPE
(Conversion from English-Units-Design Pipe Size to Metric-Units Pay-Item Pipe Size)

Figure 28-6K

MATERIAL	DESIGN PIPE SIZE (ft-in x ft-in)	DESIGN AREA (ft^2)	PAY-ITEM AREA (m^2)
STEEL	6′-1″ x 4′-7″	22	2.0
	6′-4″ x 4′-9″	24	2.2
	6′-9″ x 4′-11″	26	2.4
	7′-0″ x 5′-1″	28	2.6
	7′-3″ x 5′-3″	31	2.9
	7′-8″ x 5′-3″	33	3.1
	7′-11″ x 5′-7″	35	3.3
	8′-2″ x 5′-9″	38	3.5
	8′-7″ x 5′-11″	40	3.7
	8′-10″ x 6′-1″	43	4.0
	9′-4″ x 6′-3″	46	4.3
	9′-6″ x 6′-5″	49	4.6
	9′-9″ x 6′-7″	52	4.8
	10′-3″ x 6′-9″	55	5.1
	10′-8″ x 6′-11″	58	5.4
	10′-11″ x 7′-1″	61	5.7
	11′-5″ x 7′-3″	64	5.9
	11′-7″ x 7′-5″	67	6.2
	11′-10″ x 7′-7″	71	6.6
	12′-4″ x 7′-9″	74	6.9
	12′-6″ x 7′-11″	78	7.2
	12′-8″ x 8′-1″	81	7.5
	12′-10″ x 8′-4″	85	7.9
	13′-3″ x 9′-4″	97	9.0
	13′-6″ x 9′-6″	102	9.5
	14′-0″ x 9′-8″	105	9.8
	14′-2″ x 9′-10″	109	10.1
	14′-5″ x 10′-0″	114	10.6
	14′-11″ x 10′-2″	118	11.0
	15′-4″ x 10′-4″	123	11.4
	15′-7″ x 10′-6″	127	11.8
	15′-10″ x 10′-8″	132	12.3
	16′-3″ x 10′-10″	137	12.7
	16′-6″ x 11′-0″	142	13.2
	17′-0″ x 11′-2″	146	13.6
	17′-2″ x 11′-4″	151	14.0

DEFORMED CORRUGATED STRUCTURAL-PLATE PIPE
(Conversion From English-Units Design Pipe Size to Metric-Units Pay-Item Pipe Size)

Figure 28-6L

MATERIAL	DESIGN PIPE SIZE (ft-in x ft-in)	DESIGN AREA (ft^2)	PAY-ITEM AREA (m^2)
STEEL	17′-5″ x 11′-6″	157	14.6
	17′-6″ x 11′-8″	161	15.0
	18′-1″ x 11′-10″	167	15.5
	18′-7″ x 12′-0″	172	16.0
	18′-9″ x 12′-2″	177	16.4
	19′-3″ x 12′-4″	182	16.9
	19′-6″ x 12′-6″	188	17.5
	19′-8″ x 12′-8″	194	18.0
	19′-11″ x 12′-10″	200	18.6
	20′-5″ x 13′-0″	205	19.0
	20′-7″ x 13′-2″	211	19.6
ALUMINUM ALLOY	6′-7″ x 5′-8″	29	2.7
	6′-11″ x 5′-9″	31	2.9
	7′-3″ x 5′-11″	34	3.2
	7′-9″ x 6′-0″	36	3.3
	8′-1″ x 6′-1″	39	3.6
	8′-5″ x 6′-3″	41	3.8
	8′-10″ x 6′-4″	44	4.1
	9′-3″ x 6′-5″	47	4.4
	9′-7″ x 6′-6″	49	4.6
	9′-11″ x 6′-8″	52	4.8
	10′-3″ x 6′-9″	55	5.1
	10′-9″ x 6′-10″	58	5.4
	11′-1″ x 7′-0″	61	5.7
	11′-5″ x 7′-1″	64	5.9
	11′-9″ x 7′-2″	67	6.2
	12′-3″ x 7′-3″	70	6.5
	12′-7″ x 7′-5″	73	6.8
	12′-11″ x 7′-6″	77	7.2
	13′-1″ x 8′-2″	83	7.7
	13′-1″ x 8′-4″	86	8.0
	13′-11″ x 8′-5″	90	8.4
	14′-0″ x 8′-7″	94	8.7
	13′-11″ x 9′-5″	101	9.4
	14′-3″ x 9′-7″	105	9.8
	14′-8″ x 9′-8″	109	10.1

DEFORMED CORRUGATED STRUCTURAL-PLATE PIPE
(Conversion From English-Units Design Pipe Size to Metric-Units Pay-Item Pipe Size)

Figure 28-6L (Contd.)

MATERIAL	DESIGN PIPE SIZE (ft-in x ft-in)	DESIGN AREA (ft^2)	PAY-ITEM AREA (m^2)
ALUMINUM ALLOY	14′-11″ x 9′-10″	114	10.6
	15′-4″ x 10′-0″	118	11.0
	15′-7″ x 10′-2″	123	11.4
	16′-1″ x 10′-4″	127	11.8
	16′-4″ x 10′-6″	132	12.3
	16′-9″ x 10′-8″	136	12.6
	17′-0″ x 10′-10″	141	13.1
	17′-3″ x 11′-0″	146	13.6
	17′-9″ x 11′-2″	151	14.0
	18′-0″ x 11′-4″	156	14.5
	18′-5″ x 11′-6″	161	15.0
	18′-8″ x 11′-8″	167	15.5
	19′-2″ x 11′-9″	172	16.0
	19′-5″ x 11′-11″	177	16.4
	19′-10″ x 12′-1″	182	16.9
	20′-1″ x 12′-3″	188	17.5
	20′-1″ x 12′-6″	194	18.0
	20′-10″ x 12′-7″	199	18.5
	21′-1″ x 12′-9″	205	19.0
	21′-6″ x 12′-11″	211	19.6
	20′-1″ x 13′-11″	216	20.1
	20′-7″ x 14′-3″	224	20.8
	21′-5″ x 14′-3″	241	22.4
	21′-11″ x 14′-11″	254	23.6
	22′-8″ x 15′-3″	267	24.8

DEFORMED CORRUGATED STRUCTURAL-PLATE PIPE
(Conversion From English-Units Design Pipe Size to Metric-Units Pay-Item Pipe Size)

Figure 28-6L (Contd.)

DESIGN DIAMETER (in. x in.)	DESIGN AREA (ft^2)	PAY-ITEM AREA (m^2)
23″ x 14″	1.8	0.17
30″ x 19″	3.3	0.31
34″ x 22″	4.1	0.38
38″ x 24″	5.1	0.47
42″ x 27″	6.3	0.59
45″ x 29″	7.4	0.68
49″ x 32″	8.8	0.82
53″ x 34″	10.2	0.95
60″ x 38″	12.9	1.20
68″ x 43″	16.6	1.55
76″ x 48″	20.5	1.90
83″ x 53″	24.8	2.30
91″ x 58″	29.5	2.73
98″ x 63″	34.6	3.21
106″ x 68″	40.1	3.73
113″ x 72″	46.1	4.28
121″ x 77″	52.4	4.87
128″ x 82″	59.2	5.49
136″ x 87″	66.4	6.17
143″ x 92″	74.0	6.87
151″ x 97″	82.0	7.63
166″ x 106″	99.2	9.22
180″ x 116″	118.6	11.02

DEFORMED SMOOTH PIPE
(Conversion from English-Units-Design Pipe Size to Metric-Units Pay-Item Pipe Size)

Figure 28-6M

DESIGN BOX SIZE (ft x ft)	PAY-ITEM SIZE (mm x mm)
3′ x 2′	900 x 600
3′ x 3′	900 x 900
4′ x 2′	1200 x 600
4′ x 3′	1200 x 900
4′ x 4′	1200 x 1200
5′ x 3′	1500 x 900
5′ x 4′	1500 x 1200
5′ x 5′	1500 x 1500
6′ x 3′	1800 x 900
6′ x 4′	1800 x 1200
6′ x 5′	1800 x 1500
6′ x 6′	1800 x 1800
7′ x 4′	2100 x 1200
7′ x 5′	2100 x 1500
7′ x 6′	2100 x 1800
7′ x 7′	2100 x 2100
8′ x 4′	2400 x 1200
8′ x 5′	2400 x 1500
8′ x 6′	2400 x 1800
8′ x 7′	2400 x 2100
8′ x 8′	2400 x 2400
9′ x 5′	2700 x 1500
9′ x 6′	2700 x 1800
9′ x 7′	2700 x 2100
9′ x 8′	2700 x 2400
9′ x 9′	2700 x 2700
10′ x 5′	3000 x 1500
10′ x 6′	3000 x 1800
10′ x 7′	3000 x 2100
10′ x 8′	3000 x 2400
10′ x 9′	3000 x 2700
10′ x 10′	3000 x 3000
11′ x 4′	3300 x 1200
11′ x 6′	3300 x 1800
11′ x 8′	3300 x 2400
11′ x 10′	3300 x 3000
11′ x 11′	3300 x 3300
12′ x 4′	3600 x 1200
12′ x 6′	3600 x 1800
12′ x 8′	3600 x 2400
12′ x 10′	3600 x 3000
12′ x 12′	3600 x 3600

PRECAST REINFORCED-CONCRETE BOX SECTIONS
(Conversion from English-Units-Design Pipe Size to Metric-Units Pay-Item Pipe Size)

Figure 28-6N

DESIGN STRUCTURE SIZE (ft x ft)	PAY-ITEM SIZE (mm x mm)
12′ x 3′	3600 x 900
12′ x 4′	3600 x 1200
12′ x 5′	3600 x 1500
12′ x 6′	3600 x 1800
12′ x 7′	3600 x 2100
12′ x 8′	3600 x 2400
12′ x 9′	3600 x 2700
12′ x 10′	3600 x 3000
13′ x 3′	3900 x 900
13′ x 4′	3900 x 1200
13′ x 5′	3900 x 1500
13′ x 6′	3900 x 1800
13′ x 7′	3900 x 2100
13′ x 8′	3900 x 2400
13′ x 9′	3900 x 2700
13′ x 10′	3900 x 3000
14′ x 3′	4200 x 900
14′ x 4′	4200 x 1200
14′ x 5′	4200 x 1500
14′ x 6′	4200 x 1800
14′ x 7′	4200 x 2100
14′ x 8′	4200 x 2400
14′ x 9′	4200 x 2700
14′ x 10′	4200 x 3000
15′ x 3′	4500 x 900
15′ x 4′	4500 x 1200
15′ x 5′	4500 x 1500
15′ x 6′	4500 x 1800
15′ x 7′	4500 x 2100
15′ x 8′	4500 x 2400
15′ x 9′	4500 x 2700
15′ x 10′	4500 x 3000

PRECAST REINFORCED-CONCRETE THREE-SIDED STRUCTURE
(Conversion from English-Units Design Pipe Size to Metric-Units Pay-Item Pipe Size)

Figure 28-6 O

DESIGN STRUCTURE SIZE (ft x ft)	PAY-ITEM SIZE (mm x mm)
16′ x 3′	4800 x 900
16′ x 4′	4800 x 1200
16′ x 5′	4800 x 1500
16′ x 6′	4800 x 1800
16′ x 7′	4800 x 2100
16′ x 8′	4800 x 2400
16′ x 9′	4800 x 2700
16′ x 10′	4800 x 3000
17′ x 3′	5100 x 900
17′ x 4′	5100 x 1200
17′ x 5′	5100 x 1500
17′ x 6′	5100 x 1800
17′ x 7′	5100 x 2100
17′ x 8′	5100 x 2400
17′ x 9′	5100 x 2700
17′ x 10′	5100 x 3000
18′ x 3′	5400 x 900
18′ x 4′	5400 x 1200
18′ x 5′	5400 x 1500
18′ x 6′	5400 x 1800
18′ x 7′	5400 x 2100
18′ x 8′	5400 x 2400
18′ x 9′	5400 x 2700
18′ x 10′	5400 x 3000
19′ x 3′	5700 x 900
19′ x 4′	5700 x 1200
19′ x 5′	5700 x 1500
19′ x 6′	5700 x 1800
19′ x 7′	5700 x 2100
19′ x 8′	5700 x 2400
19′ x 9′	5700 x 2700

PRECAST REINFORCED-CONCRETE THREE-SIDED STRUCTURE
(Conversion from English-Units Design Pipe Size to Metric-Units Pay-Item Pipe Size)

Figure 28-6 O (Contd.)

DESIGN STRUCTURE SIZE (ft x ft)	PAY-ITEM SIZE (mm x mm)
19′ x 10′	5700 x 3000
20′ x 3′	6000 x 900
20′ x 4′	6000 x 1200
20′ x 5′	6000 x 1500
20′ x 6′	6000 x 1800
20′ x 7′	6000 x 2100
20′ x 8′	6000 x 2400
20′ x 9′	6000 x 2700
20′ x 10′	6000 x 3000
21′ x 3′	6300 x 900
21′ x 4′	6300 x 1200
21′ x 5′	6300 x 1500
21′ x 6′	6300 x 1800
21′ x 7′	6300 x 2100
21′ x 8′	6300 x 2400
21′ x 9′	6300 x 2700
21′ x 10′	6300 x 3000
22′ x 3′	6600 x 900
22′ x 4′	6600 x 1200
22′ x 5′	6600 x 1500
22′ x 6′	6600 x 1800
22′ x 7′	6600 x 2100
22′ x 8′	6600 x 2400
22′ x 9′	6600 x 2700
22′ x 10′	6600 x 3000
23′ x 3′	6900 x 900
23′ x 4′	6900 x 1200
23′ x 5′	6900 x 1500
23′ x 6′	6900 x 1800
23′ x 7′	6900 x 2100
23′ x 8′	6900 x 2400

PRECAST REINFORCED-CONCRETE THREE-SIDED STRUCTURE
(Conversion from English-Units Design Pipe Size to Metric-Units Pay-Item Pipe Size)

Figure 28-6 O (Contd.)

DESIGN STRUCTURE SIZE (ft x ft)	PAY-ITEM SIZE (mm x mm)
23′ x 9′	6900 x 2700
23′ x 10′	6900 x 3000
24′ x 3′	7200 x 900
24′ x 4′	7200 x 1200
24′ x 5′	7200 x 1500
24′ x 6′	7200 x 1800
24′ x 7′	7200 x 2100
24′ x 8′	7200 x 2400
24′ x 9′	7200 x 2700
24′ x 10′	7200 x 3000
25′ x 3′	7500 x 900
25′ x 4′	7500 x 1200
25′ x 5′	7500 x 1500
25′ x 6′	7500 x 1800
25′ x 7′	7500 x 2100
25′ x 8′	7500 x 2400
25′ x 9′	7500 x 2700
25′ x 10′	7500 x 3000
26′ x 3′	7800 x 900
26′ x 4′	7800 x 1200
26′ x 5′	7800 x 1500
26′ x 6′	7800 x 1800
26′ x 7′	7800 x 2100
26′ x 8′	7800 x 2400
26′ x 9′	7800 x 2700
26′ x 10′	7800 x 3000
27′ x 3′	8100 x 900
27′ x 4′	8100 x 1200
27′ x 5′	8100 x 1500
27′ x 6′	8100 x 1800
27′ x 7′	8100 x 2100

PRECAST REINFORCED-CONCRETE THREE-SIDED STRUCTURE
(Conversion from English-Units Design Pipe Size to Metric-Units Pay-Item Pipe Size)

Figure 28-6 O (Contd.)

DESIGN STRUCTURE SIZE (ft x ft)	PAY-ITEM SIZE (mm x mm)
27′ x 8′	8100 x 2400
27′ x 9′	8100 x 2700
27′ x 10′	8100 x 3000
28′ x 3′	8400 x 900
28′ x 4′	8400 x 1200
28′ x 5′	8400 x 1500
28′ x 6′	8400 x 1800
28′ x 7′	8400 x 2100
28′ x 8′	8400 x 2400
28′ x 9′	8400 x 2700
28′ x 10′	8400 x 3000
29′ x 3′	8700 x 900
29′ x 4′	8700 x 1200
29′ x 5′	8700 x 1500
29′ x 6′	8700 x 1800
29′ x 7′	8700 x 2100
29′ x 8′	8700 x 2400
29′ x 9′	8700 x 2700
29′ x 10′	8700 x 3000
30′ x 3′	9000 x 900
30′ x 4′	9000 x 1200
30′ x 5′	9000 x 1500
30′ x 6′	9000 x 1800
30′ x 7′	9000 x 2100
30′ x 8′	9000 x 2400
30′ x 9′	9000 x 2700
30′ x 10′	9000 x 3000

PRECAST REINFORCED-CONCRETE THREE-SIDED STRUCTURE
(Conversion from English-Units Design Pipe Size to Metric-Units Pay-Item Pipe Size)

Figure 28-6 O (Contd.)

PIPE SIZE (in.)	WALL THICKNESS (in.)
12	1-3/4
15	2
18	2-1/4
21	2-3/4
24	3-1/2
27	3-3/4
30	4-1/4
33	4-1/2
36	4-3/4

NON-REINFORCED CONCRETE PIPE
(Class 3 Wall Thickness)

Figure 28-6P

PIPE SIZE (in.)	WALL THICKNESS (in.)
12”	2-3/4”
15”	3”
18”	3-1/4”
21”	3-1/2”
24”	3-3/4”
27”	4”
30”	4-1/4”
33”	4-1/2”
36”	4-3/4”
42”	5-1/4”
48”	5-3/4”
54”	6-1/4”
60”	6-3/4”
66”	7-1/4”
72”	7-3/4”
78”	8-1/4”
84”	8-3/4”
90”	9-1/4”
96”	9-3/4”
102”	10-1/4”
108”	10-3/4”
114”	11-1/4”
120”	11-3/4”
126”	12-1/4”
132”	12-3/4”
138”	13-1/4”
144”	13-3/4”

REINFORCED-CONCRETE-PIPE WALL THICKNESS

Figure 28-6Q

SPAN x RISE	WALL THICKNESS	
	COVER < 2.0 ft	COVER ≥ 2.0 ft
36" x 24"	7”	4”
36" x 36"	7”	4”
48" x 24"	7.6”	5”
48" x 36"	7.6”	5”
48" x 48"	7.6”	5”
60" x 36"	8”	6”
60" x 48"	8”	6”
60" x 60"	8”	6”
72" x 36"	8”	7”
72" x 48"	8”	7”
72" x 60"	8”	7”
72" x 72"	8”	7”
84" x 48"	8”	8”
84" x 60"	8”	8”
84" x 72"	8”	8”
84" x 84"	8”	8”
96" x 48"	8”	8”
96" x 60"	8”	8”
96" x 72"	8”	8”
96" x 84"	8”	8”
96" x 96"	8”	8”
108" x 60"	9”	9”
108" x 72"	9”	9”
108" x 84"	9”	9”
108" x 96"	9”	9”
108" x 108"	9”	9”
120" x 60"	10”	10”
120" x 72"	10”	10”
120" x 84"	10”	10”
120" x 96"	10”	10”
120" x 108"	10”	10”
120" x 120"	10”	10”
132" x 48"	11”	11”
132" x 72"	11”	11”
132" x 96"	11”	11”
132" x 120"	11”	11”
132" x 132"	11”	11”
144" x 48"	12”	12”
144" x 72"	12”	12”
144" x 96"	12”	12”
144" x 120"	12”	12”
144" x 144"	12”	12”

PRECAST REINFORCED CONCRETE BOX SECTION
(Wall Thickness)

Figure 28-6R

SPAN x RISE	WALL THICKNESS
23" x 14"	2-3/4"
30" x 18"	3-1/4"
34" x 22"	3-1/2"
38" x 24"	3-3/4"
42" x 27"	3-3/4"
45" x 29"	4-1/2"
49" x 32"	4-3/4"
53" x 34"	5"
60" x 38"	5-1/2"
68" x 43"	6"
76" x 48"	6-1/2"
83" x 53"	7"
91" x 58"	7-1/2"
98" x 63"	8"
106" x 68"	8-1/2"
113" x 72"	9"
121" x 77"	9-1/2"
128" x 82"	9-3/4"
136" x 87"	10"
143" x 92"	10-1/2"
151" x 97"	11"
166" x 106"	12"
180" x 116"	13"

REINFORCED CONCRETE HORIZONTAL ELLIPTICAL PIPE
(Wall Thickness)

Figure 28-6S

CHAPTER 202

Hydrology (Pre-Rewrite Version)

Design Memorandum	Revision Date	Publication Date*	Sections Affected
12-18	July 2012	Jan. 2013	Ch. 202

*Revisions will appear in the next published edition of the *Indiana Design Manual*.

TABLE OF CONTENTS

LIST OF FIGURES

CHAPTER TWENTY-NINE

HYDROLOGY

29-1.0 HYDROLOGIC DESIGN POLICIES

29-1.01 Introduction

The following is a summary of policies which apply to hydrologic analysis. For more information, see the AASHTO *Highway Drainage Guidelines*.

29-1.02 Surveys

Hydrologic considerations can influence the selection of a highway corridor and the alternative routes within the corridor. Studies and investigations should be performed, including the consideration of the environmental and ecological impact of the project. The magnitude and complexity of these studies should be commensurate with the importance and magnitude of the project and the problems encountered. The data to be included in these surveys or studies include topographic maps, aerial photographs, streamflow records, historical high-water elevations, flood discharges, or locations of hydraulic features such as reservoirs, water projects, or designated or regulatory floodplain areas.

29-1.03 Flood-Hazard Areas

A hydrologic analysis is a prerequisite to identifying flood-hazard areas and determining the locations at which construction and maintenance will be unusually expensive or hazardous.

29-1.04 Coordination

Interagency coordination is necessary because many levels of government plan, design, and construct highway and water resource projects which can have an effect on each other. Agencies can share data and experiences within project areas to assist in the completion of accurate hydrologic analyses. The agencies include the Indiana Department of Natural Resources (IDNR), U.S. Fish and Wildlife Service (USFS), U.S. Army Corps of Engineers (USACOE), watershed management organizations, Natural Resources Conservation Service (NRCS), U.S. Geological Survey (USGS), and counties and cities.

29-1.05 Documentation

The design of a highway drainage facility should be adequately documented. It is often necessary to refer to plans or specifications long after the actual construction has been completed. Documentation should include final computations, method of analysis selected, drainage area map, designer's name and date, project correspondence relative to hydraulic considerations, and permit information. See Section 28-5.0 for Department guidelines on documentation for hydrologic information.

29-1.06 Evaluation of Runoff Factors

For each hydrologic analysis, the following must be evaluated and included if they will have a significant effect on the final results.

1. Drainage-basin characteristics including size, shape, slope, land use, geology, soil type, surface infiltration, and storage.

2. Stream channel characteristics including geometry and configuration, natural and artificial controls, channel modification, aggradations or degradation, and ice and debris.

3. Floodplain characteristics.

4. Meteorological characteristics such as precipitation amounts and type, distribution characteristics, and time rate of precipitation (hyetograph).

5. Where appropriate, the designer should evaluate future land use changes that can occur during the service life of the proposed facility and that can result in an inadequate drainage system.

29-1.07 Flood History

Each hydrologic analysis must consider the flood history of the area and the effect of such historical floods on each existing or proposed structure. The flood history must include the historical floods and the flood history for each existing structure.

29-2.0 OVERVIEW

29-2.01 Introduction

The analysis of the peak rate of runoff, volume of runoff, and time distribution of flow is fundamental to the design of a drainage facility. The design of each highway drainage facility requires the determination of discharge-frequency relationships. One design may require a peak-flow rate while another requires a runoff hydrograph providing an estimate of runoff volume. The peak-flow rate is used in the design of a bridge, culvert, roadside ditch, or small storm-sewer system. A drainage system involving detention storage, pumping stations, or large or complex storm-sewer systems require the development of a runoff hydrograph.

Errors in the estimates will result in a structure that is either undersized and causes more drainage problems, or oversized and costs more than necessary. A hydrologic analysis is only an approximation. The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex. Insufficient data is available concerning the factors influencing the rural and urban rainfall-runoff relationship to expect exact solutions.

29-2.02 Definition

Hydrology is defined as a science which explores the interrelationship between water on and under the earth and in the atmosphere. For this *Manual*, hydrology will address estimating flood magnitudes as the result of precipitation. In the design of a highway-drainage structure, floods are considered in terms of peak runoff or discharge in cubic feet per second (ft^3/s) and hydrographs as discharge per time. For a structure which is designed to control the volume of runoff (e.g., detention storage facility) or where flood routing through a culvert is used, the entire discharge hydrograph will be of interest. Wetland hydrology, the water-related driving force to create a wetland, is addressed in the AASHTO *Highway Drainage Guidelines*, Volume X.

29-2.03 Factors Affecting a Flood

In the hydrologic analysis for a drainage structure, the designer must recognize that there are many variables that affect a flood. Some of the factors which must be considered on an individual site-by-site basis include the following:

1. rainfall amount and storm distribution;
2. drainage area size, shape, and orientation;
3. ground cover and soil type;
4. slopes of terrain and streams;
5. antecedent moisture condition;
6. storage potential (overbank, pond, wetlands, reservoir, channel, etc.);

7. watershed-development potential;
8. type of precipitation (rain, snow, hail, or combinations thereof); and
9. elevation and mixed-population events.

<u>29-2.04 Sources of Information</u>

The type and source of information available for hydrologic analysis will vary from site to site, and it is the responsibility of the designer to determine what information is available and applicable to each analysis. Sources of information include, but are not limited to the following:

1. discharge information, IDNR;
2. topographic maps, USGS;
3. county soil maps, NRCS;
4. stream flow data and regression equations, USGS;
5. hydrology studies, NRCS;
6. flood insurance studies, IDNR;
7. watershed data studies done by other units of government;
8. rainfall data, U.S. Weather Bureau;
9. aerial photos, INDOT;
10. flood data, U.S. Army Corps of Engineers;
11. site visits; and
12. State, county, or local maps, as appropriate.

29-3.0 SYMBOLS AND DEFINITIONS

To provide consistency within this Chapter and throughout this *Manual*, the symbols in Figure 29-3A, Hydrologic Symbols and Definitions, will be used. These symbols have been selected because of their widespread use in hydrologic publications.

29-4.0 CONCEPT DEFINITIONS [REV. JAN. 2011]

The following discusses concepts which will be important in a hydrologic analysis. These concepts will be used throughout the remainder of this Chapter in addressing different aspects of hydrologic studies.

1. <u>Antecedent Moisture Conditions</u>. These are the soil-moisture conditions of the watershed at the beginning of a storm. These conditions affect the volume of runoff generated by a specific storm event. They affect the peak discharge only in the lower range of flood

magnitude (i.e., below about the 15-year event threshold). As flooding becomes rarer, antecedent moisture has a rapidly-decreasing influence on runoff.

2. Depression Storage. This consists of the natural depressions within a watershed which store runoff. After the depression storage is filled, runoff will commence.

3. Frequency. This is the number of times a flood of a given magnitude can be expected to occur on average over a long period of time. Frequency analysis is the estimation of peak discharges for various recurrence intervals. Frequency can also be expressed by means of probability. Probability analysis seeks to define the flood flow with a probability of being equaled or exceeded in a given year.

4. Hydraulic Roughness. This is a composite of the physical characteristics which influence the flow of water across the earth's surface, whether natural or channelized. It affects both the time response of a watershed and drainage channel and the channel storage characteristics.

5. Hydrograph. This is a graph of the time distribution of runoff from a watershed.

6. Hyetographs. This is a graph of the time distribution of rainfall over a watershed.

7. Infiltration. This is a complex process of allowing runoff to penetrate the ground surface and flow through the upper soil surface. The infiltration curve is a graph of the time distribution at which this occurs.

8. Interception. This consists of storage of rainfall on foliage or another intercepting surface during a rainfall event.

9. Lag Time. This is the time from the centroid of the excess rainfall to the peak of the hydrograph.

10. Peak Discharge. Also identified as peak flow, is the maximum rate of flow of water passing a given point during or after a rainfall event or snowmelt.

11. Rainfall Excess. This is the water available to runoff after interception, depression storage, and infiltration requirements have been satisfied.

12. Rainfall Intensity. This is the amount of rainfall occurring in a unit of time, converted to its equivalent in inches per hour.

13. Recurrence Interval. This is the average number of years between occurrences of a discharge or rainfall that equals or exceeds the given magnitude.

14. Runoff. This is the portion of the precipitation which runs off the surface of a drainage area after all abstractions are accounted for.

15. Runoff Coefficient. This is a factor representing the portion of runoff resulting from a unit rainfall. It is dependent on topography, land use, and soil characteristics.

16. Stage. The stage of a river is the elevation of the water surface above an elevation datum.

17. Time of Concentration. This is the time required for a drop of water falling on the hydraulically most-remote point in the watershed to travel through the watershed to the point under investigation.

18. Ungaged Stream Site. This is a location at which no systematic records are available for actual stream flow.

19. Unit Hydrograph. This is the direct runoff hydrograph resulting from a rainfall event which has a specific temporal and spatial distribution, which lasts for a specific duration, and which has unit volume (or results from a unit depth of rainfall). The ordinates of the unit hydrograph are such that the volume of direct runoff represented by the area under the hydrograph is equal to one inch of runoff from the drainage area. If a unit hydrograph is shown with units of cubic feet per second, it is implied that the ordinates are cubic feet per second per inch of direct runoff.

20. Volumetric Runoff Coefficient, R_V. This is used in water-quality calculations. It represents the portion of rainfall that becomes runoff and is dependent on percentage of impervious cover.

21. Water-Quality Volume. This is the treatment volume or accumulated direct runoff depth that should be treated to remove a significant percentage of the stormwater pollution load, or approximately 80% of the average annual post-development total suspended solids loading. Water-quality volume is identified as WQ_V for units of acre-feet, or Q_{WV} for units of inches. Water-quality volume is used to design a detention-based water-quality-treatment system such as a stormwater wetland, wet detention pond, or water-quality swale.

22. Water-Quality-Treatment Rate, Q_{WA}. This is the design flow rate for flow through a water-quality system. A properly designed flow-through system, such as an infiltration trench or hydrodynamic separator, should be able to satisfy pollutant-removal

requirements at the design-flow rate, and should be able to bypass inflows greater than the design treatment rate.

For a more-complete discussion of these concepts and others related to hydrologic analysis, the designer should see *Hydrologic Design For Highways*, Federal Highway Administration, Hydraulic Design Series 2, 1995; and *Guidelines for Hydrology - Volume II Highway Drainage Guidelines*, Task Force On Hydrology and Hydraulics, AASHTO Highway Subcommittee on Design.

29-5.0 DESIGN FREQUENCY

29-5.01 Overview

Because it is not economically feasible to design a structure for the maximum runoff that a watershed is capable of producing, a design frequency must be established. The design frequency for a given flood is defined as the reciprocal of the probability or chance that a flood will be equaled or exceeded in a given year. If a flood has a 20 percent chance of being equaled or exceeded each year over a long period of time, the flood will be equaled or exceeded on average once every five years. This is called the return period or recurrence interval (RI). Thus, the exceedance probability equals 100/RI. The 5-year flood is not one that will necessarily be equaled or exceeded every five years. There is a 20 percent chance that the flood will be equaled or exceeded in a given year. Therefore, the 5-year flood can conceivably occur in several consecutive years. The same reasoning applies to a flood with another return period.

INDOT has related design frequency to roadway serviceability. Roadway serviceability may be defined as travel lanes open to traffic with no floodwaters encroaching into the travel lanes during a design storm. The higher functional classifications require design-flood frequencies of less-frequent storms than the lower functional classifications.

29-5.02 Design Frequency

The design frequency used to design a hydraulic facility is determined by the type, size, and location of the structure. The following applies to the design frequency for the indicated drainage application.

1. Cross Drainage. A drainage facility should be designed to accommodate a discharge with a given return period(s) for the following circumstances. The design should be such that the backwater (the headwater) caused by the structure for the design storm does not cause the following:

a. significantly increases the flood hazard for property;
b. overtop the highway; or
c. exceeds a certain depth on the highway embankment.

Based on these design criteria, a design involving temporary roadway overtopping for a flood larger than the design event is acceptable practice. If overtopping is allowed, the structure may be designed to accommodate a flood of a lesser frequency without overtopping.

2. Storm Drain. A storm drain should be designed to accommodate a discharge with a given return period(s) for the following circumstances. The design shall be such that the storm runoff does not cause the following:

   a. significantly increases the flood hazard for property;
   b. encroach onto the street or highway so as to cause a significant traffic hazard; or
   c. limit traffic, emergency vehicles, or pedestrian movement to an unreasonable extent.

   Based on these design criteria, a design involving temporary street or road inundation for a flood larger than the design event is acceptable practice.

See Figure 29-5A, Design Frequency (Return Period – Years).

29-5.03 Review Frequency

Where appropriate, the design of a hydraulic structure should include an assessment of flood hazards inherent in the proposed facility for frequencies other than the design frequency. After sizing a drainage facility using a flood or possibly the hydrograph corresponding to the design frequency, it is necessary to review the proposed facility with a base discharge. This is done to ensure that there are no unexpected flood hazards inherent in the proposed facility. Where the design Q is less than Q_{100}, the review flood should be the 100-year event. If available, discharges should be obtained from the coordinated discharge curves, which are shown in the IDNR publication *Coordinated Discharges of Selected Streams in Indiana* and in FEMA/NFIP publications.

Potential impacts to consider include possible flood damage due to a high embankment where overtopping is not practical, backup due to the presence of a median barrier or noise wall, or flood damage due to a storm-sewer backup. Potential scour damage to a bridge substructure should be reviewed for the 500-year frequency.

29-5.04 Rainfall Curves

Rainfall data are available for many geographic areas. From these data, rainfall intensity-duration-frequency (IDF) curves have been developed for the commonly-used design frequencies. The IDF curves are shown in Section 29-8. The curves have been developed using HYDRAIN's HYDRO module, and they are based on National Weather Service (NWS) technical memorandum, HYDRO-35. HYDRO may be used to develop the IDF for a specific location with known latitude and longitude for a duration up to 60 min.

29-6.0 HYDROLOGIC PROCEDURE SELECTION

29-6.01 Overview

Streamflow measurements for determining a flood frequency relationship at a site are usually unavailable. Therefore, it is accepted practice to estimate peak runoff rates and hydrographs using statistical or empirical methods. Results from using several methods should be compared, not averaged. The designer should review the design discharge for other structures on the stream and historical data and consider previous studies including flood-insurance studies. INDOT's practice is to use the discharge that best reflects local project conditions with the reasons documented. The following discusses INDOT's use for each procedure.

29-6.02 Peak-Flow Rate or Hydrograph

A consideration of peak-runoff rate for the design condition is adequate for a conveyance system such as a storm drain or open channel. However, if the design must include flood routing (e.g., storage basin, complex conveyance network), a flood hydrograph is required. Although the development of a runoff hydrograph (more complex than estimating peak-runoff rate) is accomplished using computer programs, some methods are adaptable to nomographs or other desktop procedures.

29-6.03 Hydrologic Methods

Where feasible, for a large structure, more than one method of computing discharge should be checked, comparing the results to what other structures in the area are designed for and the historical data for the area. Engineering judgment should then be used to select the discharge. If practical, the method should be calibrated to local conditions and tested for accuracy and reliability.

Figure 29-6A, Selection of Discharge Computation Method, summarizes the recommended hydrologic methods currently acceptable for use and their application in the design of a highway structure. The following provides additional guidance on the selection of hydrologic methods.

1. IDNR Coordinated Discharge Curves. This is the preferred method for a stream for which the information is available. The reference is *Coordinated Discharges of Selected Streams in Indiana*.

2. IDNR Letter of Discharge. The IDNR Letter of Discharge must be prepared for a structure that requires a Construction in a Floodway Permit.

3. NRCS (formerly SCS) Unit Hydrograph Method (TR-20). This method can be used to determine peak discharge and hydrograph in a rural area for a given basin size.

4. HEC I. This hydrograph method can be used to determine peak discharge and hydrograph in a rural area for a given basin size, or in an urban area with large watersheds.

5. Indiana USGS Regression Equations. This method can be used in a rural area for estimating if no other method is available.

6. Rational Method. This is the preferred method for a developed area. It can be used for a drainage area of less than 100 acres in an urban area or that of less than 200 acres in a rural area.

7. FEMA. The 100-year discharge specified in the applicable FEMA flood-insurance study should be used to analyze impacts of a proposed crossing on a regulatory floodway. However, if the discharge is considered outdated, the discharge based on current methods may be used subject to receiving the necessary regulatory approvals.

8. Frequency Analysis of Stream-Gaging Records. The IDNR Division of Water maintains a database of discharges for various frequencies computed using methodologies included in Water Resources Council Bulletin 17B. Comparisons of discharges computed for nearby gages can be of value.

29-7.0 TIME OF CONCENTRATION

29-7.01 Overview

The time of concentration, t_C, is the time required for water to flow from the hydraulically most-remote point of the drainage area to the point under investigation. Time of concentration is an important variable in many hydrologic methods, including the Rational and Natural Resources Conservation Service (formerly SCS) procedures. For the same size watershed, the shorter the t_C, the larger the peak discharge.

29-7.02 Procedure

Water moves through a watershed as a combination of overland and channelized flow. The type that occurs is a function of the conveyance system and is best determined by field inspection. In designing a drainage system, the overland flow path is not necessarily perpendicular to the contours shown on available mapping. The land will often be graded, and swales will intercept the natural contour and conduct the water to the streets which reduces the time of concentration. The overland-flow path should be less than 100 ft.

29-7.02(01) Available Methods

See Figure 29-7A, Methods for Calculating Time of Concentration.

29-7.02(02) Selection of Method

The methods included in this Chapter are applicable for both the Rational Equation and the NRCS (formerly SCS) Peak Flow or Hydrograph Methods. In the Rational Equation, t_C is expressed in minutes. In the NRCS procedures, t_C is expressed in hours.

To choose a method, consider the conditions for which the equation was developed and how they compare to the drainage area being designed. If NRCS methods will be used to compute discharge, t_C should be determined using the methods recommended by the NRCS.

29-7.02(03) Total Time of Concentration

To obtain the total time of concentration, the channel-flow time must be calculated and added to the overland-flow time. After first determining the average flow velocity in the pipe or channel, the travel time, t_t, is obtained by dividing velocity into the pipe or channel length.

$$t_t = \frac{L}{60V} \qquad \text{(Equation 29-7.1)}$$

Where:
- t_t = travel time, min
- L = length which runoff must travel, ft
- V = estimated or calculated velocity, ft/s

The total time of concentration is as follows:

$$t_C = t_O + t_t \qquad \text{(Equation 29-7.2)}$$

Where:
- t_C = total time of concentration
- t_O = overland flow time
- t_t = travel time

29-7.02(04) Storm-Drainage System

For a storm-drainage system, the time of concentration for an area consists of an inlet time plus the time of flow in a closed conduit or open channel to the design point. Inlet time is the sum of the time required for water to move across the pavement or overland back of the curb to the gutter, plus the time required for flow to move through the length of gutter to the inlet. If the total time of concentration for pavement-drainage inlets is less than 5 min, a minimum of 5 min should be used to estimate the duration of rainfall.

29-7.03 NRCS Curve Number

The Natural Resources Conservation Service (NRCS) (formerly SCS) Curve Number method may be used to estimate the total time of concentration for a small rural area of 3 acres to 2000 acres (NRCS, 1989). Use Equation 29-7.3 to estimate the time of concentration from a natural, homogeneous watershed with the same curve number as follows:

$$t_C = \frac{(L)^{0.8}\left(\frac{1000}{CN} - 9\right)^{0.7}}{1140Y^{0.5}} \qquad \text{(Equation 29-7.3)}$$

Where:
- t_C = time of concentration, h
- L = length of mainstream to farthest divide, ft
- Y = average watershed slope, %
- CN = NRCS curve number (Section 29-10)

The above equation should only be used for a rural watershed with a flow length between 200 ft and 26,000 ft (5 mi), and an average watershed slope between 0.5% and 64%. This method is included in HYDRO as an option for calculating t_C.

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Example 29-7.1

Given: L = 660 ft Y = 2 % CN = 77

Find: Time of concentration, t_C, using Equation 29-7.3.

Solution:
$$t_C = \frac{(660)^{0.8}\left(\frac{1000}{77} - 9\right)^{0.7}}{1140(2)^{0.5}} = 0.29\ h = 17\ \text{min}$$

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29-7.04 Kinematic Wave Equation

HEC No. 12 (FHWA, 1984) recommends the kinematic wave equation as the most-realistic method for estimating overland flow time of concentration. The equation is as follows:

$$t_O = \frac{0.93 L^{0.6} n^{0.6}}{(Ci)^{0.4} S^{0.3}} \qquad \text{(Equation 29-7.4)}$$

Where:
- t_O = time of overland flow, min
- L = overland flow length, ft
- n = Manning roughness coefficient
- C = runoff coefficient
- i = rainfall rate, in./h
- S = average slope of the overland area, decimal

In using the equation, both the time of concentration and rainfall intensity are unknown and iteration is required. A value for i is first assumed and the related time of concentration found. The assumed rainfall intensity must then be checked against the rainfall Intensity-Duration-Frequency curve for the frequency of the event chosen for the particular design problem, and the procedure repeated until the assumed rainfall intensity approximately agrees with the intensity associated with the time of concentration. This method is included in HYDRO.

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Example 29-7.2

Given: L = 150 ft
S = 0.02
n = 0.24 (dense grass)
C = 0.40 (impervious soil with turf)
Design frequency = 10 yr
Location: Indianapolis

Find: Overland flow time, t_O, using Equation 29-7.4.

Solution:

1. Assume i = 4.8 in./h and calculate t_O as follows:

$$t_O = \frac{0.93(150)^{0.6}(0.24)^{0.6}}{[(0.40)(4.8)]^{0.4}(0.02)^{0.3}} = 20 \text{ min}$$

2. Based on calculated t_O, find i from Figure 29-8D, Rainfall Intensity-Duration-Frequency Curve (Indianapolis):

i = 4 in./h

3. Calculate t_O as follows:

$$t_O = \frac{0.93(150)^{0.6}(0.24)^{0.6}}{[(0.40)(4)]^{0.4}(0.02)^{0.3}} = 21 \text{ min}$$

4. Based on calculated t_O, find i from Figure 29-8D.

i = 3.92 in./h, assumed value was 4 in./h; therefore, t_O = 21 min

29-7.05 Manning's Kinematic Solution

For sheet (overland) flow of less than 300 ft, TR-55 (NRCS, 1986) recommends Manning's kinematic solution (Overton and Meadows, 1976) to compute t_O. This method is included in the TR-55 computer program. The equation is as follows:

$$t_O = \frac{0.42(nL)^{0.8}}{(P_2)^{0.5} S^{0.4}} \qquad \text{(Equation 29-7.5)}$$

Where:
- t_O = overland flow time, min
- n = Manning's roughness coefficient, Figure 29-7B, Roughness Coefficients for the Rational Formula
- L = flow length, ft
- P_2 = 2-year, 24-h rainfall, in (from TP-40)
- S = slope of hydraulic grade line (land slope), decimal

This simplified form of the Manning's kinematic solution is based on the following:

1. shallow steady uniform flow;
2. constant intensity of rainfall excess (rain available for runoff);
3. rainfall duration of 24 h; and
4. minor effect of infiltration on travel time.

This overland time of concentration is acceptable for use within the TR-20 hydrologic methodology.

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<u>Example 29-7.3</u>

Given:
L = 150 ft
S = 0.02
n = 0.24 (dense grass)
Location: Indianapolis

Find: Overland flow time, t_O, using Equation 29-7.5.

Solution:

1. For Indianapolis, P_2 = 2.6 in, from TP 40.

2. Determine t_O as follows:

$$t_O = \frac{0.42[(0.24)(150)]^{0.8}}{(2.6)^{0.5}(0.02)^{0.4}} = 0.36\ h = 21.8\ \text{min}$$

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29-7.06 Federal Aviation Administration Method

For design conditions that do not involve complex drainage conditions, the Federal Aviation Administration Equation (FAA, 1970) can be used to estimate overland flow time. Equation 29-7.6 was developed from airport-drainage data, and it is best suited for a small drainage area with fairly homogeneous surfaces. For each drainage area, the distance is determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. Figures 29-8A and 29-8B, Runoff Coefficients for the Rational Formula, provide values for the Rational Method runoff coefficient, *C*.

$$t_O = \frac{(1.1 - C)L^{0.5}}{2.63S^{0.33}} \qquad \text{(Equation 29-7.6)}$$

Where:

- t_O = overland flow travel time, min
- L = overland flow path length, ft
- S = slope of overland flow path, decimal
- C = Rational Method runoff coefficient (Figures 29-8A and 29-8B)

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Example 29-7.4

Given:
L = 150 ft
S = 0.02
Surface: grass

Find: Overland flow time, t_O, using Equation 29-7.6

Solution:

1. Determine C from Figure 29-8A.
   For lawn, heavy soil, 2% to 7% slope, use C = 0.18.

2. Determine t_O as follows:

$$t_O = \frac{(1.1 - 0.18)(150)^{0.5}}{2.63(0.02)^{0.33}} = 16 \text{ min}$$

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29-7.07 NRCS Upland Method

The Upland Method (NRCS, 1972) can be used to determine flow velocity to estimate time of concentration. This method relates watershed slope and surface to flow velocity. HYDRO includes the Grassed Waterway relationship to calculate channel travel time. TR-55 (1986) includes the relations for Grassed Waterway for an unpaved area and paved area to determine the travel time for shallow concentrated flow as follows:

Unpaved $V = 16.393\ S^{0.5}$ (Equation 29-7.7)

Paved $V = 20.653\ S^{0.5}$ (Equation 29-7.8)

Where: V = average velocity, ft/s
S = slope of hydraulic grade line (watercourse slope), decimal

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Example 29-7.5

Given: L = 500 ft
S = 0.025 (gutter slope)
Surface: concrete (paved)

Find: Gutter travel time, t_t (shallow concentrated flow)

Solution:

1. Determine V from Equation 29-7.8 as follows:

$$V = 20.653\ (0.025)^{0.5} = 3.27 \text{ ft/s}$$

2. Determine t_t from Equation 29-7.1 as follows:

$$t_t = \frac{500\ ft}{(3.27\ ft/s)(60 \sec/\min)} = 2.6 \min$$

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29-7.08 Triangular Gutter Flow

The travel time for gutter flow can be estimated using an average velocity of the flow. Equation 29-7.9 can be used to determine the velocity in a triangular gutter section given the watercourse slope, gutter cross slope, and water spread.

$$V = \frac{1.12}{n} S^{0.5} S_x^{0.67} T^{0.67} \qquad \text{(Equation 29-7.9)}$$

Where:

	V	=	flow velocity in gutter, ft/s
	n	=	Manning's roughness coefficient for sheet flow (Figure 29-7B)
	S	=	longitudinal slope, decimal
	S_x	=	gutter cross slope, decimal
	T	=	water spread, ft

For a triangular channel with uniform inflow per length and zero flow at the upstream end, the average velocity will occur where the spread is 65% of the maximum. HYDRO includes this method as an option to determine travel time.

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Example 29-7.6

Given: S = 0.025 (longitudinal slope)
S_x = 0.02 (cross slope)
T = 10 ft (design spread at inlet)
L = 500 ft (flow length)
n = 0.016 (concrete)

Find: Travel time of flow in gutter

Solution:

1. Use $T_{avg} = 2.17T_{design} = (2.17)(3) = 6.51$ ft

2. From Equation 29-7.9:

$$V = \frac{1.12}{0.016} (0.025)^{0.5} (0.02)^{0.67} (6.51)^{0.67} = 2.82 \text{ ft/s}$$

3. From Equation 29-7.1:

$$t_t = \frac{500\,ft}{(2.82\,ft/s)(60\sec/\min)} = 2.9\min$$

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29-7.09 Mannings' Equation

In a watershed with storm drains or channels, the travel time must be added to the overland flow time to find the total time of concentration where appropriate. The velocity can be determined using Manning's equation as follows:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad \text{(Equation 29-7.10)}$$

Where: V = mean velocity of flow, ft/s
n = Manning's roughness coefficient
R = hydraulic radius = Area/Wetted Perimeter (ft)
S = slope of the hydraulic grade line, decimal

29-7.09(01) Pipe Flow

For ordinary conditions, a storm drain should be sized assuming that it will flow full or almost full for the design discharge. For non-pressure flow, the velocity can be determined using Manning's equation. For a circular pipe flowing full, the equation becomes the following:

$$V = \frac{0.593}{n} D^{2/3} S^{1/2} \qquad \text{(Equation 29-7.11)}$$

Where: D = diameter of circular pipe, ft

Pipe flow charts can be used to determine the velocity for either full or partially-full flow conditions.

29-7.09(02) Open Channel

An open channel is assumed to begin where the surveyed cross-section information has been obtained, where the channel is visible on an aerial photograph, or where a blue line (indicating a stream) appears on a United States Geological Survey (USGS) quadrangle sheet. Manning's equation or the water-surface profile information can be used to estimate average flow velocity. Equation 29-7.10 can be used to determine the average flow velocity. It is determined for bank-full elevation.

29-7.10 Continuity Equation

If the pipes of a storm-drainage system will operate under pressure flow, the continuity equation should be used to determine velocity as follows:

$$V = Q/A \qquad \text{(Equation 29-7.12)}$$

Where:
- V = mean velocity of flow, ft/s
- Q = discharge in pipe, ft^3/s
- A = area of pipe, ft^2

29-7.11 Reservoir or Lake

It may be necessary to compute t_C for a watershed having a relatively large body of water within its flow path. Therefore, t_C is computed to the upstream end of the lake or reservoir. For the body of water, the travel time is computed using the following equation (King, 1967).

$$V_w = (gD_m)^{0.5} \qquad \text{(Equation 29-7.13)}$$

Where:
- V_w = the wave velocity across the water, ft/s
- g = the acceleration due to gravity, or 32.2 ft/s^2
- D_m = mean depth of lake or reservoir, ft

V_w will be 8.3 ft/s to 30 ft/s. Equation 29-7.13 only estimates travel time across the lake. It does not account for the travel time involved with the passage of the inflow hydrograph through spillway storage and the reservoir or lake outlet. This time is much longer and is added to the travel time across the lake. The travel time through lake storage and its outlet can be determined by the storage routing procedures described in Chapter Thirty-five.

Equation 29-7.13 can be used for a swamp with considerable open water, but where the vegetation or debris is relatively thick (less than about 25 percent open water) Manning's equation is more appropriate.

29-7.12 Kerby's Equation

The time of concentration for overland flow using Kerby's Equation is calculated as follows:

$$t_O = K\left(LNS^{-0.5}\right)^{0.467} \qquad \text{(Equation 29-7.14)}$$

Where:
- t_O = time of overland flow, min
- K = 0.67
- L = length of flow, ft
- N = retardance roughness coefficient (Figure 29-7C)
- S = average slope of overland flow, decimal

The length used in the equation, *L*, is the straight-line distance from the most-distant point of the watershed to the outlet, measured parallel to the slope of the land until a well-defined channel is reached. A watershed of less than 10 acres was used to calibrate the model. The slope was less than 1%, the *N* value was 0.8 or less, and surface flow dominated.

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Example 29-7.7

Given:
- L = 666.7 ft
- S = 0.5%
- Grass cover
- N = 0.40 from Figure 29-7C

Find: Overland t_O using Kerby's Equation.

Solution: Using Equation 29-7.14:

$$t_O = 0.67\left[(666.7)(0.40)(0.005)^{-0.5}\right]^{0.467} = 38.8 \text{ min}$$

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29-7.13 Kirpich's Equation

Kirpich's Equation is an empirical watershed equation based on data which account for length, slope, and soil cover. It derives from work done to determine the rate of runoff from a small agricultural watershed. The Equation is considered applicable to a watershed from 3 ac to 200 ac.

Kirpich's Equation is expressed as follows:

$$t_C = 0.0078\left(L^{0.77}\right)H^{-0.385} \qquad \text{(Equation 29-7.15)}$$

Where: t_C = time of concentration, h
L = length of the longest waterway from the point in question to the basin divide, ft
H = difference in elevation between the point in question and the basin divide (omitting drops due to gully overfills, waterfalls, etc.), ft

Kirpich's Equation works well for a natural, rural basin with well-defined channels, for overland flow on bare earth, or for a mowed earth roadside channel. Using the Equation, a paved basin and a forested one will have identical times of concentration if the lengths and reliefs are the same. This cannot occur; therefore the Equation should be adjusted if it is used elsewhere using the following guidelines:

For overland flow on a grass surface, multiply t_C by 2.0.
For overland flow on a concrete or asphalt surface, multiply t_C by 0.4.
For flow in a concrete-lined channel, multiply t_C by 0.2.

The application of Kirpich's Equation to a basin is as follows:

1 Compute the length, L, in feet between the basin divide and the point in question.

2. Compute the relief, H, in feet between the basin divide and the point in question. The elevation of the basin divide should represent an average of the elevations in the immediate vicinity of the termination point of the longest watercourse. This procedure avoids bias in the t_C computation due to an isolated peak in the headwater area. The elevation of the site should be interpolated between successive contours crossing the stream.

Compute t_C in hours using Equation 29-7.15.

Apply an adjustment factor, if applicable, based on surface type.

The t_C produced by Step 4 is appropriate for an urban area or a steep area. For the use of Kirpich's Equation, steep is defined as an overall basin slope of greater than 0.6% to 0.7%. For other than an urban area or a steep area, the t_C produced by Step 4 should be divided by 0.6. Because a basin may not clearly be rural or urban, or flat or steep, divide Kirpich's Equation by 0.8.

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Example 29-7.8

Given: L = 2666.7 ft
H = 33 ft
Surface: grass

Find: Time of concentration, t_C, using Kirpich's Equation, 29-7.15.

Solution:

1. Using Equation 29-7.15:

$$t_C = 0.0078\left(2666.7^{0.77}\right)\left(0.0125\right)^{-0.385} = 18.3\,\text{min}$$

2. For overland flow on a grass surface, multiply t_C by 2.0 as follows:

t_C = 2 (18.3) = 37 min

3. The basin slope = 33.3 / 2666.7 = 1.25%. Therefore, this is defined as steep for the use of Kirpich's Equation. No other adjustments are necessary. Therefore t_C = 37 min.

t_C = 2 (0.31) = 0.62 h = 37 min

29-8.0 RATIONAL METHOD

29-8.01 Introduction

The Rational Method is used to calculate the peak flow from a small drainage area. It is recommended for estimating the design-storm peak runoff for a rural area of up to 200 ac or an urban area of up to 100 ac.

29-8.02 Application

The precautions to be considered in applying the Rational Method are as follows.

1. The first step in applying the Rational Method is to obtain a topographic map and to define the boundaries of the drainage area under study. A field inspection of the area should also be made to determine if the natural drainage divides have been altered.

2. Restrictions to the natural flow such as highway crossings or dams that exist in the drainage area should be investigated to determine how they affect the design flows.

3. The charts, graphs, and tables included herein are not intended to replace reasonable and prudent engineering judgment which should permeate each step in the design process.

29-8.03 Characteristics

The Rational-Method formula applies a developed area with a significant amount of pavement, gutters, or storm sewers. The assumptions within the Rational Method include the following.

1. Basin Size. The rate of runoff resulting from rainfall intensity is a maximum if the rainfall intensity lasts as long as or longer than the time of concentration. That is, the entire drainage area does not contribute to the peak discharge until the time of concentration has elapsed.

   This assumption limits the size of the drainage basin that can be evaluated by the Rational Method. For a large drainage area, the time of concentration can be so large that constant rainfall intensity for such a long period does not occur, and a shorter, more-intense rainfall can produce a larger peak flow.

2. Frequency of Peak Discharge. The frequency of peak discharges is the same as that for the rainfall intensity for the given time of concentration.

   The frequency of peak discharge depends on rainfall frequency, antecedent moisture conditions in the watershed, and the response characteristics of the drainage system. For a small and largely impervious area, rainfall frequency is the dominant factor. For a larger drainage basin, the response characteristics control.

3. Runoff. The fraction of rainfall that becomes runoff, *C*, is independent of rainfall intensity or volume.

   The assumption is reasonable for an impervious area such as a street, rooftop, or parking lot. For a pervious area, the fraction of runoff varies with rainfall intensity and the accumulated volume of rainfall. The selected runoff coefficient must be appropriate for the storm, soil, and land-use conditions.

4. Peak Rate. The peak rate of runoff is sufficient information for design.

29-8.04 Equation

The Rational-Method formula estimates the peak rate of runoff at a location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for duration equal to the time of concentration. Because the result of using the Rational-Method formula to estimate peak discharge is sensitive to the parameters used, the designer must use engineering judgment in estimating values that are used in the Method. The formula is expressed as follows:

$$Q = CIA \qquad \text{(Equation 29-8.1)}$$

Where: Q = maximum rate of runoff, ft^3/s

C = runoff coefficient representing a ratio of runoff to rainfall

I = average rainfall intensity for a duration equal to the time of concentration for a selected return period, in./h

A = drainage area tributary to the design location, acres.

Due to assumptions made in the formula, C can vary depending on the design storm. The value for C can be expressed as follows:

$$C = kC_1$$

Where: k = factor to adjust formula. It should be taken as follows:

Design Storm	*k*
2- to 10-year	1.0
25-year	1.1
50-year	1.2
100-year	1.25

C_1 = runoff coefficient representing a ratio of runoff to rainfall.

29-8.05 Time of Concentration

The time of concentration is the time required for water to flow from the hydraulically most-remote point of the drainage area to the point under investigation. Use of the Rational Formula requires the time of concentration, t_C, for each design point within the drainage basin to determine the rainfall intensity. Section 29-7.0 provides the methods for computing time of concentration.

29-8.05(01) Storm-Drainage System

For a storm-drainage system, the designer is interested in two different times of concentration, one for inlet spacing and one for pipe sizing. There is a difference between the two times as discussed in the following.

1. 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. This is the sum of the time required for water to move across the pavement or overland back of the curb to the gutter, plus the time required for flow to move through the length of the gutter to the inlet. For pavement drainage, if the total time of concentration to the upstream inlet is less than 5 min, a minimum t_C of 5 min 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. Travel time between inlets is not considered.

2. Pipe Sizing. The time of concentration for a 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. If there is more than one source of runoff to a given point in a storm-drainage system, the longest t_C is used to estimate the rainfall intensity, I. There can be an exception to this, 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 aware of this possibility if joining drainage areas and determining which drainage area governs.

29-8.05(02) Common Errors

Two common errors should be avoided when calculating t_c. First, runoff from a portion of the drainage area which is highly impervious may result in a greater peak discharge than can occur if the entire area is considered. An adjustment can be made to the drainage area by disregarding those areas where flow time is too slow to add to the peak discharge. It may be necessary to estimate several different times of concentration to determine the design flow that is critical for a specific application.

Second, the overland flow path is not necessarily perpendicular to the contours shown on available mapping. The land may be graded and swales will intercept the natural contour and conduct the water to the streets which reduces the time of concentration. The overland flow path should not exceed 200 ft in an urban area or 300 ft in a rural area.

29-8.06 Runoff Coefficient

The runoff coefficient, *C*, requires engineering judgment and an understanding by the designer. A typical coefficient represents the integrated effects of many drainage basin parameters. The selected value must be appropriate for the storm, soil, and land-use conditions.

Two sets of runoff coefficients for various types of surfaces are shown in Figures 29-8A and 29-8B. The designer may select a runoff coefficient from either set as deemed appropriate for the specific site application. The total *CA* value should be based on a ratio of the drainage areas associated with each *C* value as follows:

$$\text{Total } CA = A_1C_1 + A_2C_2 + A_3C_3 \ldots \qquad \text{(Equation 29-8.2)}$$

The coefficients provided in Figures 29-8A and 29-8B are applicable to a storm of five- to ten-year frequency. A less frequent, higher intensity storm will require a higher coefficient because infiltration and other losses have a proportionately smaller effect on runoff (Wright, McLaughlin, 1969).

As the slope of the drainage basin increases, the selected *C* value should also increase. This is because, as the slope of the drainage area increases, the velocity of overland and channel flow will increase allowing less opportunity for water to infiltrate the ground surface. Thus, more of the rainfall will become runoff from the drainage area.

Figure 29-8A, Runoff Coefficient for the Rational Formula, provides an example for the calculation of a weighted runoff coefficient.

29-8.07 Rainfall Intensity

The rainfall intensity, *I*, is the average rainfall rate in inches per hour for a duration equal to the time of concentration for a selected return period. Once a return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from the rainfall Intensity-Duration-Frequency (IDF) curves.

IDF curves are located at the NOAA website,
http://hdsc.nws.noaa.gov/hdsc/pfds/orb/in_pfds.html.

29-8.08 Rational-Method Example Problem

The following example problem illustrates the application of the Rational Method to estimate the peak discharge. The peak runoff is needed at the storm-sewer catch basin for a 10-yr return period.

Step 1: Determine site data.

The following data were measured from a topographic map and field survey:

Residential area (single family) = 1.725 ac
Pavement area (concrete) = 0.3 ac

Length of overland flow = 150 ft — Average overland slope = 2.0%
Length of concrete gutter = 500 ft — Slope of gutter = 0.025

Step 2: Choose runoff coefficient from Figure 29-8A and find total *CA*.

Land Use	Area	Runoff Coefficient	*CA*
Residential (single family)	1.725	0.40	0.69
Concrete Pavement	0.3	0.90	0.27
		Total Weighted *CA* =	0.96

Step 3: Calculate the total time of concentration to the inlet.

Overland flow time: t_O calculated using the kinematic wave equation in Example 29-7.2 to be 14 min

Channel flow time: t_t calculated using the triangular gutter method in Example 29-7.6 to be 3 min

Total Time of Concentration: $t_C = t_O + t_t = 14 + 3 = 17$ min

Step 4: Find Rainfall Intensity, *I*, from NOAA website, http://hdsc.nws.noaa.gov/hdsc/pfds/orb/in_pfds.html. Click on location on map corresponding to the peak discharge. The information is shown in English measurement units only. For this example,

Indianapolis
10-year return period
Duration = t_C = 17 min.

Interpolation:
Solve for *x*,

$$\frac{(30-17)}{(1.62-x)} = \frac{(17-15)}{(x-1.16)}$$

x = 1.22 in. for 17 min, which converts to 4.31 in./h

I_{10} = 4.31 in./h from NOAA website

Step 5: Compute Peak Runoff.

$Q = CIA = (0.96)\,(4.31) = 4.14$ ft^3/s

29-9.0 USGS REGRESSION EQUATIONS

The USGS procedure cannot be used for the final design.

29-9.01 Introduction

This Section provides equations for estimating the magnitude and frequency of a flood at an ungaged site on a regulated rural stream. They are based on the USGS publication *Techniques for Estimating Magnitude and Frequency of Floods on Streams in Indiana* (Water-Resources Investigations Report 84-4134). The equations were developed by multiple-regression analysis of basin characteristics and peak-flow statistical data from 242 gaged locations in Indiana, Ohio and Illinois. The State was divided into seven areas on the basis of the regression analysis. A set of equations for estimating peak discharge with recurrence intervals of 2, 10, 25, 50, and 100 years was developed for each area. Significant basin characteristics in the equations are drainage area, channel length, channel slope, mean annual precipitation, storage, precipitation intensity, and a runoff coefficient. Standard errors of estimate for the equations range from 24 percent to 45 percent.

This Section also provides methods for estimating flood magnitude and frequency at a site on a gaged stream.

29-9.02 Hydrologic Regions

Regression analyses use stream-gage data to define hydrologic regions. These are geographic regions which have very similar flood frequency relationships, and as such commonly display similar characteristics. Because of the distance between stream gages, the regional boundaries cannot be considered as precise. Figure 29-9A, Areas for Selecting Flood-Frequency USGS Estimating Equations, shows the hydrologic regional boundaries.

Problems related to hydrologic boundaries may occur in selecting the appropriate regression equation. The watershed of interest may lie partially within two or more hydrologic regions, or it may lie totally within a hydrologic region but close to a hydrologic region boundary. A field visit is recommended to first collect all available historical flood data and to compare the project watershed characteristics with those of the abutting hydrologic regions.

29-9.03 Basin Characteristics

The basin characteristics that are required for use of the USGS equations are defined as follows:

1. Design Discharge, Q_t, (cubic feet per second). The peak discharge for the specified design flood frequency, *t*.

2. Drainage Area, *DA*, square miles. The area contributing directly to runoff at the study site. Draw an outline of the drainage areas on a topographic map and use a planimeter to determine the area.

3. Main-Channel Slope, *SL*, feet per mile. The slope of the streambed between points that are 10% and 85% of the distance from the location on the stream to the basin divide. Determine from topographic maps to nearest 0.1 ft/mi.

4. Channel Length, *L*, miles. The distance measured along the main channel from the location on the stream to the basin divide is determined from topographic maps to the nearest 0.1 mile.

5. Storage,*STOR*, percent. The percentage of drainage area covered by lakes, ponds, or wetlands.

6. Mean Annual Precipitation, *PREC*, inches. The 1941-70 average annual precipitation is determined from Figure 29-9B, Mean Annual Precipitation (1941-70), (Stewart, 1983). A constant of 30 in. is subtracted from the characteristic *PREC* for use in the estimating equations. Plot the basin centroid determined from Figure 29-9B and determine the mean annual precipitation for that point by interpolating between lines of equal precipitation.

7. Precipitation Intensity, $I_{24,2}$, inches. The maximum 24-h precipitation having a recurrence interval of 2 yr is determined from Figure 29-9C, Two-Year, 24-Hour Projection, (Hershfield, 1961).

8. Runoff Coefficient, *RC*. A coefficient that relates storm runoff to soil permeability by means of major hydrologic soil groups is determined from Figure 29-9D, Major Hydrologic Soil Groups, (Davis, 1975). Values range from 0.3 for hydrologic soil group A to 1.0 for hydrologic soil group E.

29-9.04 Regression Equations for Ungaged Site

Figure 29-9E, Production Equations, Standard Errors of the Estimate, *SEE*, and Equivalent Years of Record, *EY*, provides the equations for an ungaged site for each of the seven geographic areas (see Figure 29-9A). Figure 29-9F, Ranges of Area Basin Characteristics for USGS Regression Equations, provides the ranges for application of each basin characteristic to each of the geographic areas.

29-9.05 Procedure

Follow this procedure for the USGS Method.

Step 1: From Figure 29-9A, locate the area for the site.

Step 2: From *Techniques for Estimating Magnitude and Frequency of Floods on Streams in Indiana*, USGS Report 84-4134, Figure 1 and Table 4, determine if the study site is at a gaged site or on a gaged stream.

Step 3: If the site is on a gaged stream, go to Step 6.

Step 4: Determine the basin characteristics necessary to solve the regression equation from Figure 29-9E (Prediction Equations).

Step 5: Use the appropriate equation from Figure 29-9E to solve for the required discharge.

Step 6: If the site is at a gaged location, the weighted estimate of Q_t from the USGS Report Table 4should be used.

Step 7: If the drainage area of an ungaged site on a gaged stream is less than 50% or greater than 150% of the drainage area of a gaged site on the same stream, the discharge should be estimated from the appropriate equation in Figure 29-9E as if the site were on an ungaged stream. Go to Step 4.

Step 8: If the drainage area of an ungaged site on a gaged stream is between 50% and 150% of the drainage area of a gaged site on the same stream, the discharge should be an estimate calculated from both gaged data (USGS Report Table 4) and estimating equations (Figure 29-9E). An estimate of the process is as follows:

a. Compute the ratio as follows:

$$R = \frac{Q_{TW}}{Q_{TR}}$$

Where: Q_{TW} = weighted estimate of T-year flood at gaged site
Q_{TR} = regression equation estimate of T-year flood at gaged site
Both can be obtained from USGS Report, Table 4

b. Compute weighting factor as follows:

$$R_W = R - \left(\frac{2\Delta A}{A_G}\right)(R-1)$$

Where: R = ratio defined in Step 8.a.
ΔA = absolute value of the difference between the drainage areas (DA) of the gaged and ungaged sites

$A_G = DA$ of gaged site

c. Compute T-year peak discharge at the ungaged site as follows:

$$Q_T = Q_{TR} R_W$$

Where: Q_{TR} = regression equation estimate of T-year flood at ungaged site

* * * * * * * * * *

Example 29-9.1: Ungaged Stream

Given: Location: Brown County
$DA = 6.94$ mi^2 (ungaged site)
$L = 4.40$ mi
Elevation of channel at 10% of length (0.4 mi) = 652′
Elevation of channel at 85% of length (3.7 mi) = 824′
Distance between points = 3.7 - 0.4 = 3.3 mi
Channel Slope, $SL = \frac{824 - 652}{3.3} = 52.1\ ft/mi$
$I_{24,2} = 3.05$ in., from Figure 29-9C

Find: The 100-year discharge.

Solution:

Step 1. Determine area where site is located. From Figure 29-9A, Brown County is located in Area 3 for an ungaged stream.

Step 2. From Figure 29-9E, the regression equation for Q_{100} in Area 3 is as follows:

$$Q_{100} = 181\left(DA^{0.779}\right)\left(SL^{0.466}\right)\left(I_{24,2} - 2.5\right)^{0.831}$$

Step 3. Substitute the values of basin characteristics as follows:

$$Q_{100} = 181(6.94)^{0.779}(52.1)^{0.466}(3.05 - 2.5)^{0.831} = 3140\ cfs$$

Example 29-9.2 (Gaged Stream)

Given: Gaging Station 03366500 on the Muscatatuck River near Deputy;
Ungaged site on Muscatatuck River downstream from gaging station.
Basin characteristics (ungaged site) are as follows:

DA_U = 359 mi^2
SL = 6.2 ft/mi
L = 68.8 mi
$I_{24,2}$ = 3.00 in.

Find: Q_{100} at gaging station
Q_{100} at ungaged site downstream

Solution:

Step 1: From USGS Report, Table 4, three values are given for Q_{100} as follows:

a. 40,900 ft^3/s, from flood frequency analysis of observed station data
b. 44,600 ft^3/s, from regression equation
c. 41,200 ft^3/s, from weighting the station and area estimates

Select the weighted value as the best estimate. Therefore, Q_{100} = 41,200 ft^3/s at the gaging site.

Step 2: From USGS Report, Figure 1, and Figure 29-9A, the gaging station is located in Area 4. The regression equation for Q_{100} in Area 4 is as follows:

$$Q_{100} = 32\left(DA^{0.565}\right)\left(SL^{0.705}\right)\left(L^{0.730}\right)\left(I_{24,2} - 2.5\right)^{0.464}$$

Step 3: Substitute the values of basin characteristics as follows:

$$Q_{100} = 32(359)^{0.565}(6.2)^{0.705}(68.8)^{0.730}(3.00 - 2.5)^{0.464} = 51{,}200 \text{ ft}^3/\text{s}$$

Step 4: Compute the ratio of the ungaged drainage area to the gaged drainage area as:

$$\frac{DA_U}{DA_G} = \frac{359}{293} = 1.22$$

1.22 is greater than 0.5 and less than 1.5, therefore the ratio is acceptable.

Step 5: Compute the gaged-site discharge ratio as follows:

$$R = \frac{Q_{TW}}{Q_{TR}} = \frac{41{,}200}{44{,}600} = 0.924$$

Step 6: Compute the weighting factor as follows:

$$R_W = R - \frac{2\Delta A}{A_G}(R-1)$$

$$R_W = 0.924 - \left[\frac{2(359-293)}{293}\right](0.924-1) = 0.958$$

Step 7: Reduce regression value by weighting factor as follows:

$$Q_T = (51{,}200)(0.958) = 49{,}000 \text{ ft}^3\text{/s}$$

29-10.0 NRCS UNIT HYDROGRAPH

29-10.01 Introduction

Techniques developed by the U.S. Natural Resources Conservation Service (formerly the Soil Conservation Service) for calculating rate of runoff require the same data as for the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall intensity. The NRCS approach, however, is more sophisticated because it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. In the NRCS method, the direct runoff can be calculated for a given storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall amount to obtain the precipitation excess. Details of the methodology can be found in the *NRCS National Engineering Handbook*, Section 4.

29-10.02 Application

A unit hydrograph and a dimensionless-unit hydrograph are used in the NRCS procedure. A unit hydrograph represents the time distribution of flow resulting from 1 in. of direct runoff occurring over the watershed in a specified time. A dimensionless-unit hydrograph represents the composite of many unit hydrographs. The dimensionless-unit hydrograph is plotted in nondimensional units of time versus time to peak and discharge at given time versus peak discharge.

Characteristics of the dimensionless hydrograph vary with the size, shape, and slope of the tributary drainage area. The most significant characteristics affecting the dimensionless hydrograph shape are the basin lag and the peak discharge for a given rainfall. Basin lag is the time from the center of mass of rainfall excess to the hydrograph peak. Steep slopes, a compact shape, and an efficient drainage network tend to make lag time short and peaks high. Flat slopes, an elongated shape, and an inefficient drainage network tend to make lag time long and peaks low.

29-10.03 Equations and Concepts

The following discussion outlines the equations and basic concepts utilized in the NRCS method.

1. Drainage Area. The drainage area of a watershed is determined from topographic maps and field surveys. For a large drainage area, it may be necessary to divide the area into subdrainage areas to account for major land-use changes, obtain analysis results at different points within the drainage area, or locate stormwater drainage facilities and assess their effects on the flood flows. A field inspection of existing or proposed drainage systems should be made to determine if the natural drainage divides have been altered. These alterations could make significant changes in the size and slope of the subdrainage areas.

2. Rainfall. See Figure 29-10A, Huff Distribution of Design Rainfall (50% Probability of Design Rainfall). Quartile II should be used. The rainfall intensity for the given duration and return period should be multiplied by the duration to determine rainfall depth.

3. Rainfall-Runoff Equation. A relationship between accumulated rainfall and accumulated runoff has been derived by NRCS from experimental plots for numerous soils and vegetative-cover conditions. Data for land-treatment measures, such as contouring and terracing, from experimental watersheds have been included. Equation 29-10.1 was developed for a small watershed for which only daily rainfall and watershed data are ordinarily available. It was developed from recorded storm data that included total amount of rainfall in a calendar day but not its distribution with respect to time. The NRCS runoff equation is therefore a method of estimating direct runoff from 24-h or 1-day storm rainfall. The equation is as follows:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}$$ (Equation 29-10.1)

Where: Q = accumulated direct runoff, in.

P = accumulated rainfall (potential maximum runoff), in.

I_a = initial abstraction including surface storage, interception, and infiltration prior to runoff, in.

S = potential maximum retention, in.

The relationship between I_a and S was developed from experimental watershed data. It removes the necessity for estimating I_a for common usage. The empirical relationship used in the NRCS runoff equation is as follows:

$$I_a = 0.2S$$ (Equation 29-10.2)

Substituting $0.2S$ for I_a in Equation 29-10.1, the NRCS rainfall-runoff equation becomes the following:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$ (Equation 29-10.3)

Figure 29-10B, NCRS Relation Between Direct Runoff, Curve Number and Precipitation, shows a graphical solution of Equation 29-10.3 which enables the precipitation excess from a storm to be obtained if the total rainfall and watershed curve number are known.

<u>29-10.04 Procedure</u>

The following is a discussion of procedures that are used in the hydrograph method along with recommended tables and figures.

29-10.04(01) Runoff Factor

In a hydrograph application, runoff is referred to as rainfall excess or effective rainfall defined as the amount by which rainfall exceeds the capability of the land to infiltrate or otherwise retain the rainwater. The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope.

Land use is the watershed cover, and it includes both agricultural and nonagricultural uses. Items such as type of vegetation, water surfaces, roads, roofs, etc., are all part of the land use. Land treatment applies to agricultural land use, and it includes mechanical practices such as contouring or terracing, and management practices such as rotation of crops.

The NRCS uses a combination of soil conditions and land-use (ground cover) to assign a runoff factor to an area. The runoff factors, called runoff curve numbers, *CN*, indicate the runoff potential of an area when the soil is not frozen. The higher the *CN*, the higher the runoff potential.

Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. The NRCS has divided soils into four hydrologic soil groups, A, B, C, and D, based on infiltration rates. The groups were previously described for the Rational Method. The applicable NRCS soil classification maps may be obtained from the appropriate county agency.

The effect of urbanization on the natural hydrologic soil group should be considered. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with the antecedent soil moisture conditions, defined as the amount of rainfall occurring in a selected period preceding a given storm. The greater the antecedent rainfall, the more direct runoff there is from a given storm. A 5-day period is used as the minimum for estimating antecedent moisture conditions. Antecedent soil moisture conditions also vary during a storm. Heavy rain falling on dry soil can change the soil moisture condition from dry to average to wet during the storm period.

The following figures provide a series of runoff factors. Figures 29-10C provides runoff curve numbers for an urban area, Figure 29-10D provides those for an undeveloped area, and Figure 29-10E provides those for agricultural land. The tables are based on the average antecedent moisture condition: soils are neither very wet nor very dry once the design storm begins. Curve numbers should be selected only after a field inspection of the watershed and a review of zoning and soil maps. Figure 29-10F provides conversion from average antecedent moisture conditions. Figure 29-10G provides rainfall groups for antecedent soil moisture conditions during growing and dormant seasons.

29-10.04(02) Time of Concentration

The average slope within the watershed together with the overall length and retardance of overland flow are the factors affecting the runoff rate through the watershed. In the NRCS method, time of concentration, t_C, is defined as the time required for water to travel from the hydraulically most-distant point in a watershed to its outlet. Lag, L, can be considered as a weighted time of concentration and is related to the physical properties of a watershed, such as area, length, and slope. The NRCS derived the following empirical relationship between lag and time of concentration.

$$L = 0.6t_C \qquad \text{(Equation 29-10.4)}$$

In an urban area of less than 2000 ac, a curve-number method can be used to estimate the time of concentration from watershed lag. In this method, the lag for the runoff from an increment of excess rainfall can be considered as the time between the center of mass of the excess rainfall increment and the peak of its incremental outflow hydrograph. The equation developed by NRCS to estimate lag is as follows:

$$L = \frac{\left([L_m]^{0.8}(S+1)^{0.7}\right)}{\left(1900Y^{0.5}\right)} \qquad \text{(Equation 29-10.5)}$$

Where:
- L = lag, h
- L_m = length of mainstream to farthest divide, ft
- Y = average slope of watershed, %
- S = $\left(\left[\frac{1000}{CN}\right] - 10\right)$, in.
- CN = NRCS curve number

The lag time can be corrected for the effects of urbanization by using Figure 29-10H, Factors for Adjusting Lag if Impervious Areas Occur in Watershed, and Figure 29-10 I, Factors for Adjusting Lag if Main Channel has been Hydraulically Improved. The amount of modification to the hydraulic flow length must be determined from topographic maps or aerial photographs following a field inspection of the area. The modification to the hydraulic flow length not only includes pipes or channels but also the length of flow in streets or drives.

After the lag time is adjusted for the effects of urbanization, Equation 29-10.4 can be used to calculate the time of concentration for use in the NRCS method. Section 29-7.0 provides an alternative procedure for travel time and time of concentration estimation.

29-10.04(03) Triangular Hydrograph Equation

The triangular hydrograph is a practical representation of excess runoff with only one rise, one peak, and one recession. Its geometric configuration can be described mathematically, which makes it useful in estimating discharge rate. The NRCS developed Equation 29-10.6 to estimate the peak rate of discharge for an increment of runoff, as follows:

$$Q_p = \frac{[0.208A(q)]}{\left(\frac{d}{2}+L\right)} \qquad \text{(Equation 29-10.6)}$$

Where: Q_p = peak rate of discharge, ft^3/s
A = area, mi^2
q = storm runoff during time interval, in
d = time interval, h
L = watershed lag, h

Equation 29-10.6 can be used to estimate the peak discharge for the unit hydrograph which can then be used to estimate the peak discharge and hydrograph from the entire watershed.

The constant 0.208, or peak-rate factor, is valid for the NRCS dimensionless-unit hydrograph. A change in the dimensionless-unit hydrograph reflecting a change in the percent of volume under the rising side will cause a corresponding change in the shape factor associated with the triangular hydrograph and therefore a change in the constant 0.208. The constant has been known to vary from about 0.258 in steep terrain to 0.129 in flat swampy terrain.

29-10.05 TR-20 Example Problem, NRCS Method

The Natural Resources Conservation Service has developed the computerized program TR-20 to perform the calculations for the NRCS hydrologic methodology. The program is in English units of measurement, therefore all input and output data must be in English units. To run TR-20, the user must input the data as follows:

1. desired design frequency;
2. runoff Curve Number;
3. average watershed slope;
4. stream cross-section data;
5. reach length;
6. structure data;
7. cumulative rainfall table including desired time increment;
8. antecedent moisture condition;
9. storm characteristics;

10. routing instructions; and
11. desired output.

Figure 29-10A provides a sample output from TR-20.

29-11.0 HEC 1

HEC 1 was developed by the U.S. Army Corps of Engineers in 1967. It is also a hydrograph-oriented program with the capability to compute, combine, and route hydrographs through a system of subareas. HEC 1 can be utilized without regard to basin size. Input requirements are similar to TR-20. A detailed description of HEC 1 is not provided here because of its similarities to TR-20.

29-12.0 REFERENCES

1. AASHTO, *Highway Drainage Guidelines*, Volume II.

2. Federal Highway Administration, HYDRAIN Documentation, 1990.

3. USGS, Water-Resources Investigations Report 84-4134, *Techniques For Estimating Magnitude and Frequency of Floods on Streams in Indiana*.

4. AASHTO, *Model Drainage Manual*, 1991 and 1997.

5. Indiana DNR, *Coordinated Discharges of Selected Streams in Indiana*.

6. Federal Highway Administration, Hydraulic Engineering Circular No. 19, *Hydrology*, 1994.

7. Water Resources Council, Bulletin 17B, *Guidelines for Determining Flood Flow Frequency*, 1981.

8. Highway Extension and Research Project for Indiana Counties and Cities, H-94-6, *Storm Water Drainage Manual*, July 1994.

9. Indiana Department of Natural Resources, Division of Water, *Hydrology and Hydraulics in Indiana, Volume 1*, January 1986.

Symbol	Definition	Unit
A	Drainage area	ac, mi^2
BDF	Basin development factor	%
C	Runoff coefficient	-
C_f	Frequency factor	-
CN	NRCS-runoff curve number	-
C_t, C_p	Physiographic coefficients	-
d	Time interval	h
DH	Difference in elevation	ft
I or i	Rainfall intensity	in./h
IA	Percentage of impervious area	%
I_a	Initial abstraction from total rainfall	in.
K	Frequency factor for a particular return period and skew	-
L	Lag	h
L_m	Length of mainstream to furthest divide	ft
L_{ca}	Length along main channel to a point opposite the watershed centroid	mi
M	Rank of a flood within a long record	-
n	Manning roughness coefficient	-
N	Number of years of flood record	years
P	Accumulated rainfall	in
Q	Rate of runoff	ft^3/s
q	Storm runoff during a time interval	in.
R	Hydraulic radius	ft
RC	Regression constant	-
RQ	Equivalent rural peak runoff rate	ft^3/s
S or Y	Ground slope	in./in., ft/mi or %
S	Potential maximum retention storage	in.
NRCS	Natural Resources Conservation Service	-
SL	Main channel slope	ft/ft
S_L	Standard deviation of the logarithms of the peak annual floods	-
ST	Basin storage factor	%
T_B	Time base of unit hydrograph	h
t_C	Time of concentration	min or h
T_L	Lag time	h
T_r	Snyder's duration of excess rainfall	h
UQ	Urban peak runoff rate	ft^3/s
V	Velocity	ft/s
X	Logarithm of the annual peak	–

HYDROLOGIC SYMBOLS AND DEFINITIONS

Figure 29-3A

Highway Classification	Drainage Appurtenance							
	Roadway Serviceability	Bridge Waterway Openings		Roadway Cross Culverts		Storm Drain Systems (2)		Open Channels (4)
		Allowable Backwater	Allowable Velocity	Allowable Backwater	Allowable Velocity	Design for Inlet Spacing And And Trunk Line (Gravity)	Check for HGL on Trunk Line (Pressure Flow)	
Freeways	100	100	100	100	50	50	N/A	10
Multilane Non-Freeways	100	100	100	100	50	10	50	10
Two-Lane Facilities (3) AADT ≥ 3000	100	100	100	100	50	10	50	10
3000 > AADT ≥ 1000	25	100	100	100	25	10	50	10
AADT < 1000	10	100	100	100	10	10	50	10
Driveways (1)	10	N/A	N/A	100	10	N/A	N/A	N/A
Bridge Decks (Non-Freeways)	N/A	N/A	N/A	N/A	N/A	10	N/A	N/A
Ramps	100	100	100	100	50	10	50	10

(1) An overtopping area should be provided at the driveway entrance to accommodate floods larger than the design flood. The designer should check the capacity of the driveway culvert for the design frequency indicated for the roadway cross culvert to ensure that there will be no overtopping of the roadway. The design storm for allowable backwater for driveway pipes will match the allowable backwater for the facility the driveway is connected to.

(2) *Inlet spacing for pavement drainage is based on both the recurrence interval and the allowable spread of water in the gutter. See [Figure 36-7A](#) for the criteria for allowable spread.*

(3) Traffic volumes are for a 20-year projection.

(4) Side ditches only. Relocated streams parallel to the road will be designed for a 100-year event.

General Note: Where existing drainage appurtenances can accommodate less frequent recurrence intervals, no reduction in hydraulic capacity nor serviceability is allowable.

DESIGN FREQUENCY

(Return Period - Years)

Figure 29-5A

Facility Description(5)	Methodology				
	IDNR Coordinated Curves	TR-20	HEC I	USGS Regression Equations	Rational Method (6)
Stream Flow, Channel, Bridge, Large Culvert	(1)	(2)	(2)	(4)	
Small Culvert		(1)	(1)	(4)	(2)
Storm Drain, Roadside Ditch, Roadside Culvert, Inlet Spacing		(2)	(2)		(1)
Pumping Station		(3)	(3)		(2)
Detention Basin		(3)	(3)		(4)

Note: *TR-55 methodology is acceptable for time of concentration only.*

(1) *Preferred method.*

(2) *Alternative if (1) is not applicable.*

(3) *Preferred for complex facility or if hydrograph is required.*

(4) *Method may be used for preliminary evaluation.*

(5) *The IDNR Letter of Discharge is required for a project that requires an IDNR Permit.*

(6) *The Rational Method can only be used for a drainage area of less than 100 ac in an urban area or less than 200 ac in a rural area.*

SELECTION OF DISCHARGE COMPUTATION METHOD

Figure 29-6A

Method	Comments
NRCS Curve Number	Overland and channelized flow time for rural area (2 to 200 ac). Included in HYDRO.
Kinematic Wave	Overland flow time. Requires iterative solution and is included in HYDRO and HEC 1.
Manning's Kinematic Solution	Overland flow time. The maximum flow length is 300 ft. Included in TR-55.
Federal Aviation Administration	Overland flow time; developed for airport.
NRCS Upland Method	Flow velocity for overland and shallow concentrated flow. Included in TR-55 for shallow, concentrated flow. Grassy waterway method included in HYDRO.
Triangular Gutter Flow	Flow velocity in gutter. Included in HYDRO.
Manning's Equation	Flow velocity for non-pressure flow in pipe or open channel.
Continuity Equation	Flow velocity for pressure flow in pipe.
Reservoir and Lake	Wave velocity across water; does not account for travel time of hydrograph through storage.
Kerby	Overland flow time. Watershed < 10 ac, slopes $< 1\%$, and N value ≤ 0.8 were used to calibrate model.
Kirpich	Good estimate of overall flow time for steep, wooded watershed. Can be applied to flatter basin by dividing result by 0.6.

METHODS FOR CALCULATING TIME OF CONCENTRATION

Figure 29-7A

Surface Description		*n* [1]
Smooth Surface (concrete, asphalt, gravel, bare soil)		0.011
Fallow (no residue)		0.05
Cultivated Soil	Residue cover ≤ 20%	0.06
	Residue cover > 20%	0.17
Grass	Short-grass prairie	0.15
	Dense grass [2]	0.24
	Bermuda grass	0.41
Range (natural)		0.13
Woods [3]	Light underbrush	0.40
	Dense underbrush	0.80

[1] *The* n *value is a composite of information complied by Engman (1986).*

[2] *Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, or native grass mixtures.*

[3] *In selecting* n, *consider cover to a height of about 12 in. This is the only part of the plant cover that will obstruct sheet flow.*

ROUGHNESS COEFFICIENT (MANNING'S *n*) FOR SHEET FLOW

Figure 29-7B

Type of Surface	*n*
Smooth, impervious surface	0.02
Smooth, bare, packed soil	0.10
Poor grass, cultivated row crops, or moderately rough, bare surface	0.20
Deciduous timberland	0.60
Pasture or overage grass	0.40
Conifer timberland, deciduous timberland with deep forest litter, or dense grass	0.80

VALUES OF *n* FOR KERBY'S FORMULA

Figure 29-7C

TYPE OF SURFACE	RUNOFF COEFFICIENT
Rural Area	
Concrete or sheet asphalt pavement	0.8 - 0.9
Asphalt macadam pavement	0.6 - 0.8
Gravel roadway or shoulder	0.4 - 0.6
Bare earth	0.2 - 0.9
Steep grassed area (2:1 slope)	0.5 - 0.7
Turf meadow	0.1 - 0.4
Forested area	0.1 - 0.3
Cultivated field	0.2 - 0.4
Urban Area	
Watertight roof surface	0.75 - 0.95
Asphalt or concrete pavement	0.80 - 0.95
Traffic-bound pavement	0.70 - 0.90
Gravel roadway	0.35 - 0.70
Impervious soil, heavy	0.40 - 0.65
Impervious soil, with turf	0.30 - 0.55
Slightly pervious soil	0.15 - 0.40
Slightly pervious soil, with turf	0.10 - 0.30
Moderately pervious soil	0.05 - 0.20
Moderately pervious soil, with turf	0.00 - 0.10

Example of Weighted *C* Factor

5% watertight roof surface	5% x 0.85 = 0.04
10% asphalt or concrete pavement	10% x 0.9 = 0.09
10% traffic-bound pavement	10% x 0.8 = 0.08
50% slightly impervious soil	50% x 0.4 = 0.20
15% slightly impervious soil, with turf	15% x 0.2 = 0.03
10% moderately pervious soil	10% x 0.1 = 0.01

WEIGHTED *C* FACTOR = 0.45

RUNOFF COEFFICIENT FOR THE RATIONAL FORMULA

Figure 29-8A

URBAN AREA

Type of Drainage Area			Runoff Coefficient
Business	Downtown		0.70 - 0.95
	Neighborhood		0.50 - 0.70
Residential	Single-family unit		0.30 - 0.50
	Multi-family unit	Detached	0.40 - 0.60
		Attached	0.60 - 0.75
	Suburban		0.25 - 0.40
	Apartment dwelling		0.50 - 0.70
Industrial	Light		0.50 - 0.80
	Heavy		0.60 - 0.90
Park or Cemetery			0.10 - 0.25
Playground			0.20 - 0.35
Railroad yard			0.20 - 0.40
Unimproved			0.10 - 0.30
Lawn	Sandy soil	flat, 2%	0.05 - 0.10
		average, 2 - 7%	0.10 - 0.15
		steep, 7%	0.15 - 0.20
	Heavy soil	flat, 2%	0.13 - 0.17
		average, 2 - 7%	0.18 - 0.22
		steep, 7%	0.25 - 0.35
Street	Asphalt		0.70 - 0.95
	Concrete		0.80 - 0.95
	Brick		0.70 - 0.85
Drive or walk			0.75 - 0.85
Roof			0.75 - 0.95

RUNOFF COEFFICIENTS FOR THE RATIONAL FORMULA

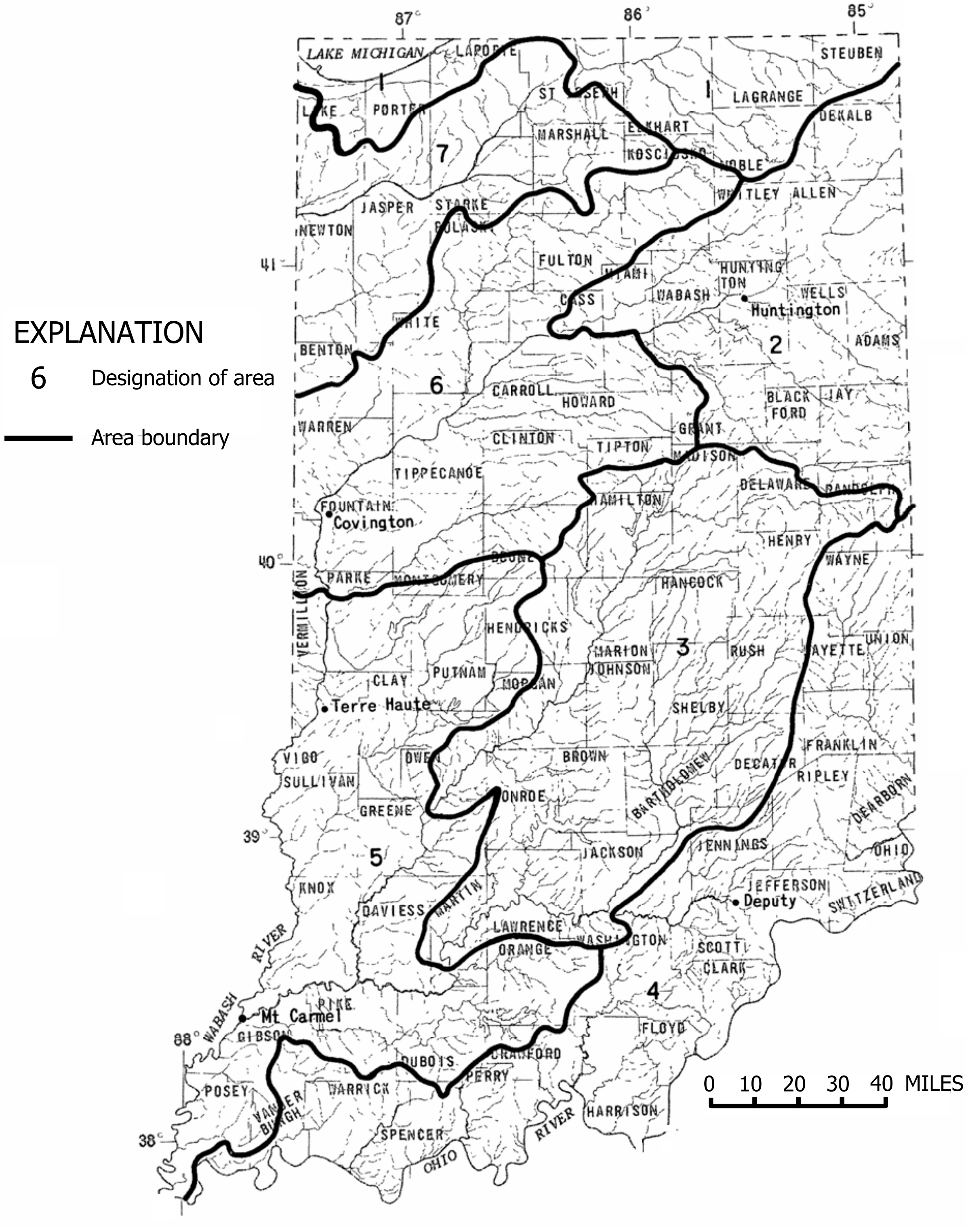
Figure 29-8B

RURAL AREA

Soil Type	Watershed Cover		
	Cultivated	Pasture	Woodlands
With above-average infiltration rate; usually sandy or gravelly.	0.20	0.15	0.10
With average infiltration rate; no clay pans, loams, or similar soils.	0.40	0.35	0.30
With below-average infiltration rate; and heavy clay soils or soils with a clay pan near the surface, and shallow soils above impervious rock	0.50	0.45	0.40

RUNOFF COEFFICIENTS FOR THE RATIONAL FORMULA

Figure 29-8B (contd.)

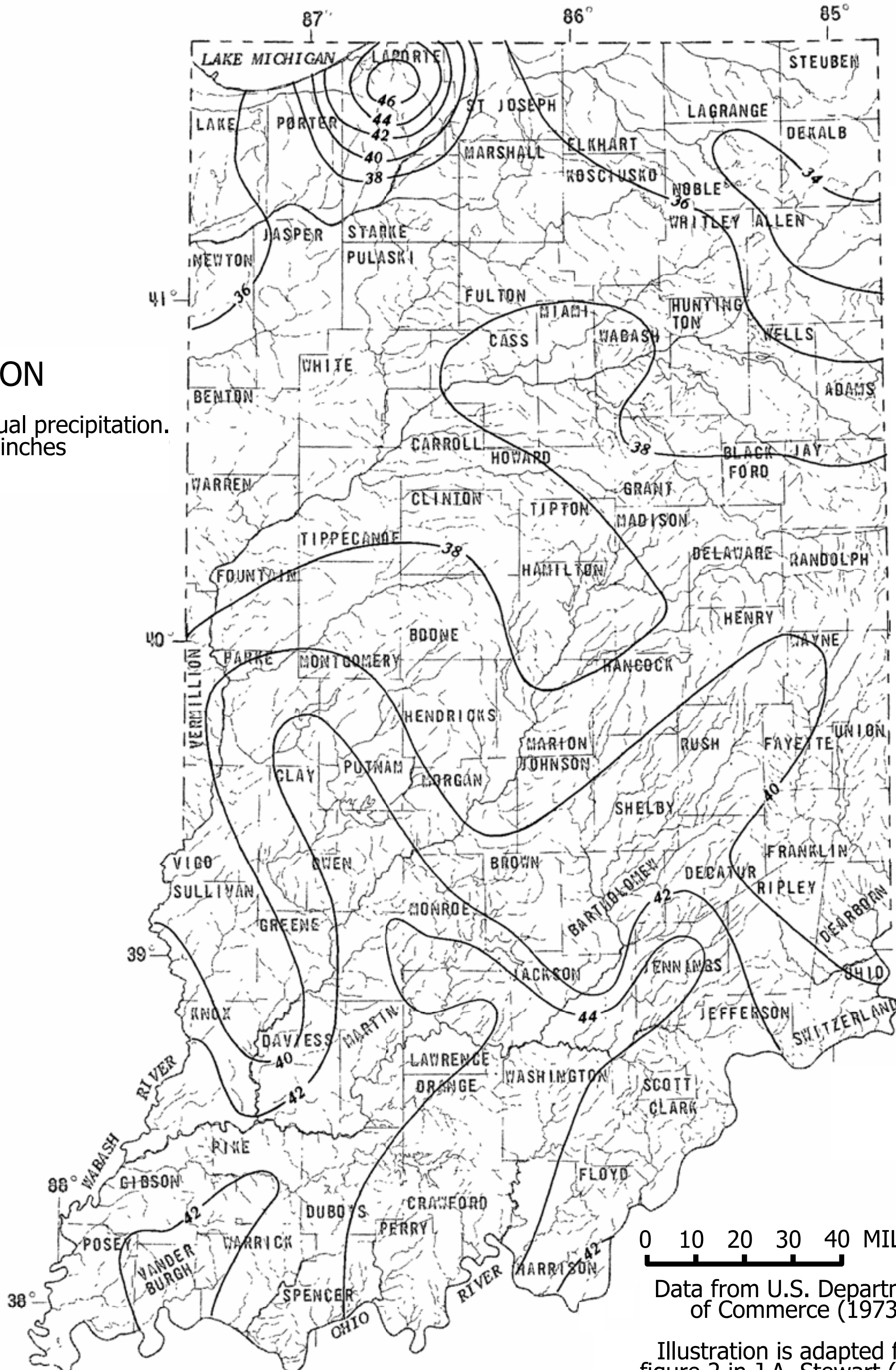


AREAS FOR SELECTING FLOOR-FREQUENCY
USGS ESTIMATING EQUATIONS

Figure 29-9A

EXPLANATION

–42– Line of equal precipitation. Interval 2 inches

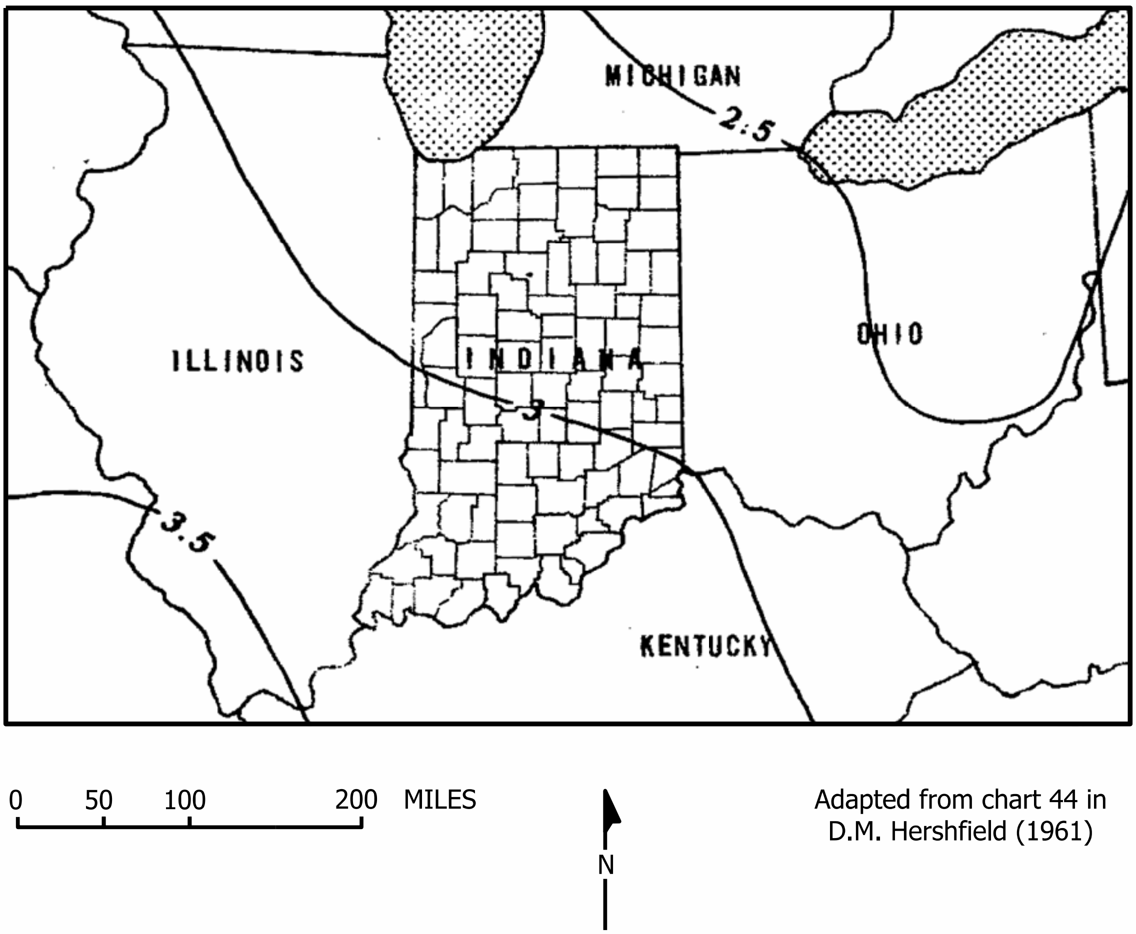


Data from U.S. Department of Commerce (1973)

Illustration is adapted from figure 2 in J.A. Stewart (1983)

MEAN ANNUAL PRECIPITATION
(1941-1970)

Figure 29-9B



EXPLANATION

—3.5— Line of equal precipitation.
Interval 0.5 inches

TWO-YEAR, 24-HOUR PRECIPITATION

Figure 29-9C

EXPLANATION

HYDROLOGIC SOIL GROUP	SOIL- RUNOFF COEFFICIENT
A	0.30
B	0.50
C	0.70
D	0.80
E	1.00

—— Hydrologic soil group boundary

(Use a runoff coefficient of 0.30 for streams in group B that are directly below lakes)

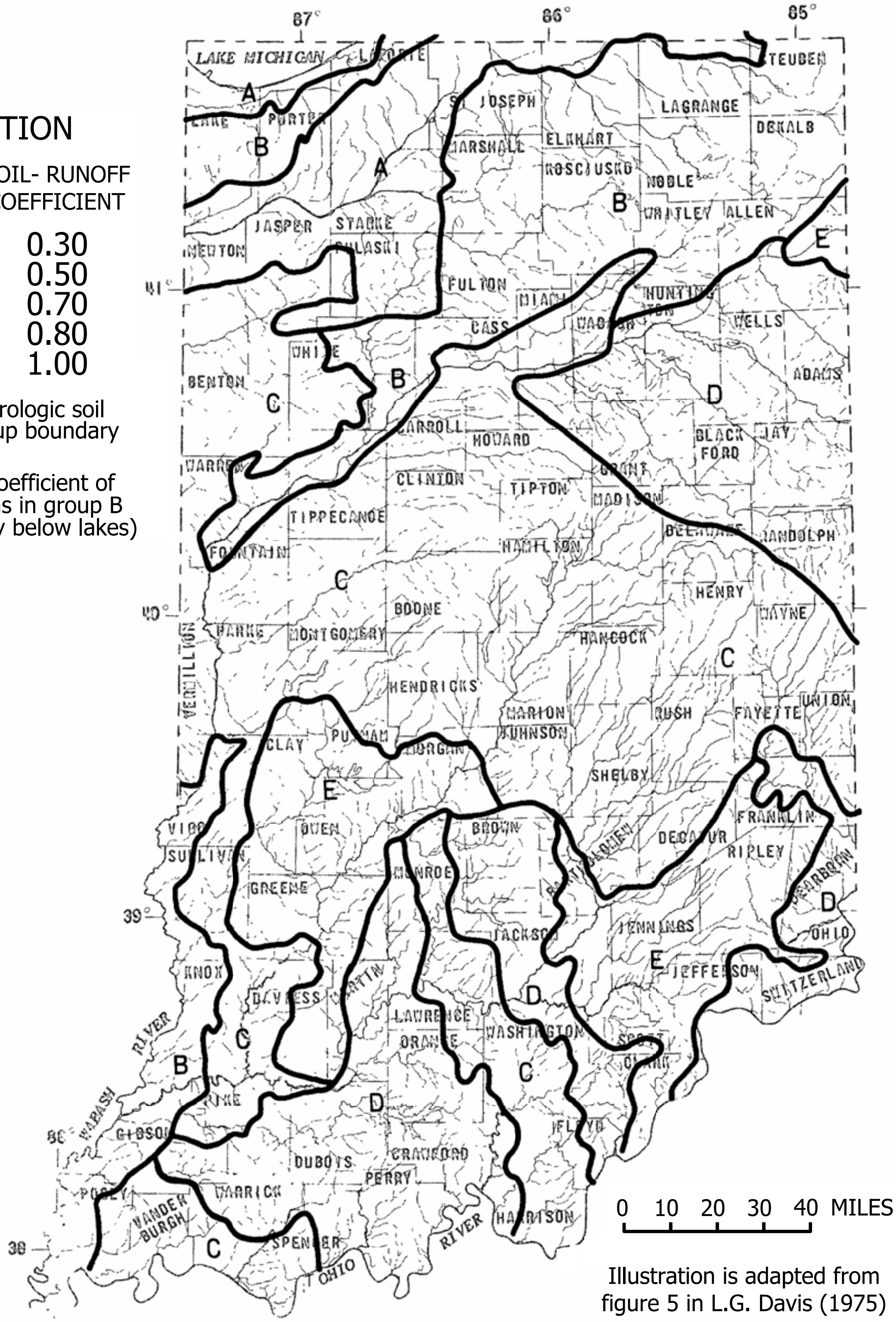


Illustration is adapted from figure 5 in L.G. Davis (1975)

MAJOR HYDROLOGIC SOIL GROUPS

Figure 29-9D

Equations	SEE	EY
AREA 1 (16 Stations)		
$Q_2 = 6.72\ DA^{0.714}\ (STOR + 1)^{-0.289}\ (PREC - 30)^{0.965}$	27%	3
$Q_{10} = 10.3\ DA^{0.701}\ (STOR + 1)^{-0.262}\ (PREC - 30)^{1.060}$	35%	3
$Q_{25} = 11.8\ DA^{0.697}\ (STOR + 1)^{-0.253}\ (PREC - 30)^{1.093}$	39%	3
$Q_{50} = 12.9\ DA^{0.696}\ (STOR + 1)^{-0.248}\ (PREC - 30)^{1.114}$	42%	4
$Q_{100} = 13.8\ DA^{0.695}\ (STOR + 1)^{-0.243}\ (PREC - 30)^{1.132}$	45%	5
AREA 2 (31 Stations)		
$Q_2 = 26.4\ DA^{0.708}\ (STOR + 1)^{-0.207}\ RC^{0.479}\ (PREC - 30)^{0.653}$	24%	4
$Q_{10} = 61.8\ DA^{0.655}\ (STOR + 1)^{-0.312}\ RC^{0.697}\ (PREC - 30)^{0.696}$	28%	4
$Q_{25} = 85.0\ DA^{0.635}\ (STOR + 1)^{-0.357}\ RC^{0.782}\ (PREC - 30)^{0.702}$	31%	5
$Q_{50} = 106\ DA^{0.619}\ (STOR + 1)^{-0.391}\ RC^{0.859}\ (PREC - 30)^{0.707}$	35%	6
$Q_{100} = 127\ DA^{0.608}\ (STOR + 1)^{-0.418}\ RC^{0.902}\ (PREC - 30)^{0.708}$	37%	7
AREA 3 (60 Stations)		
$Q_2 = 102\ DA^{0.758}\ SL^{0.273}\ (I_{24,2} - 2.5)^{0.948}$	36%	3
$Q_{10} = 141\ DA^{0.772}\ SL^{0.384}\ (I_{24,2} - 2.5)^{0.894}$	34%	4
$Q_{25} = 158\ DA^{0.776}\ SL^{0.423}\ (I_{24,2} - 2.5)^{0.868}$	36%	5
$Q_{50} = 170\ DA^{0.777}\ SL^{0.445}\ (I_{24,2} - 2.5)^{0.847}$	37%	7
$Q_{100} = 181\ DA^{0.779}\ SL^{0.466}\ (I_{24,2} - 2.5)^{0.831}$	39%	9
AREA 4 (46 Stations)		
$Q_2 = 16.8\ DA^{0.435}\ SL^{0.528}\ L^{0.860}\ (I_{24,2} - 2.5)^{0.459}$	31%	3
$Q_{10} = 24.1\ DA^{0.517}\ SL^{0.628}\ L^{0.769}\ (I_{24,2} - 2.5)^{0.445}$	30%	6
$Q_{25} = 27.4\ DA^{0.545}\ SL^{0.664}\ L^{0.741}\ (I_{24,2} - 2.5)^{0.448}$	32%	7
$Q_{50} = 29.6\ DA^{0.554}\ SL^{0.687}\ L^{0.738}\ (I_{24,2} - 2.5)^{0.458}$	34%	9
$Q_{100} = 32.0\ DA^{0.565}\ SL^{0.705}\ L^{0.730}\ (I_{24,2} - 2.5)^{0.464}$	37%	11

Q_t = Peak Discharge (ft^3/s)
DA = Drainage Area (mi^2)
SL = Channel Slope (ft/mi)
L = Channel Length (mi)
SEE = Standard Error of Estimate
STOR = Lakes, Ponds, Wetlands (%)
PREC = Average Annual Precipitation (in.)
$I_{24,2}$ = Max. 24-h, 2-yr Precipitation
RC = Runoff Coefficient
EY = Equivalent Years of Record

PREDICTION EQUATIONS, STANDARD ERRORS OF ESTIMATE, AND EQUIVALENT YEARS OF RECORD

Figure 29-9E

Equations	SEE	EY
AREA 5 (35 Stations)		
$Q_2 = 45.5\ DA^{0.760}\ SL^{0.390}$	30%	3
$Q_{10} = 67.7\ DA^{0.780}\ SL^{0.469}$	33%	5
$Q_{25} = 77.0\ DA^{0.790}\ SL^{0.499}$	36%	5
$Q_{50} = 83.8\ DA^{0.805}\ SL^{0.516}$	39%	7
$Q_{100} = 91.2\ DA^{0.811}\ SL^{0.529}$	42%	8
AREA 6 (32 Stations)		
$Q_2 = 681\ DA^{0.691}\ RC^{0.856}\ (I_{24,2} - 2.5)^{1.771}$	27%	5
$Q_{10} = 2{,}177\ DA^{0.662}\ RC^{0.865}\ (I_{24,2} - 2.5)^{1.980}$	29%	7
$Q_{25} = 3{,}165\ DA^{0.598}\ RC^{0.852}\ (I_{24,2} - 2.5)^{2.035}$	32%	7
$Q_{50} = 3{,}908\ DA^{0.584}\ RC^{0.849}\ (I_{24,2} - 2.5)^{2.049}$	34%	10
$Q_{100} = 4{,}734\ DA^{0.570}\ RC^{0.834}\ (I_{24,2} - 2.5)^{2.068}$	37%	12
AREA 7 (22 Stations)		
$Q_2 = 22.6\ DA^{0.468}\ SL^{0.414}\ L^{0.624}\ RC^{0.846}$	26%	3
$Q_{10} = 45.7\ DA^{0.350}\ SL^{0.439}\ L^{0.726}\ RC^{0.862}$	29%	4
$Q_{25} = 56.4\ DA^{0.318}\ SL^{0.458}\ L^{0.754}\ RC^{0.862}$	32%	4
$Q_{50} = 63.6\ DA^{0.300}\ SL^{0.473}\ L^{0.770}\ RC^{0.860}$	35%	5
$Q_{100} = 70.1\ DA^{0.285}\ SL^{0.488}\ L^{0.785}\ RC^{0.854}$	38%	6

Q_t =	Peak Discharge (ft^3/s)	STOR =	Lakes, Ponds, Wetlands (%)
DA =	Drainage Area (mi^2)	PREC =	Average Annual Precipitation (in.)
SL =	Channel Slope (ft/mi)	$I_{24,2}$ =	Max. 24-h, 2-yr Precipitation
L =	Channel Length (mi)	RC =	Runoff Coefficient
SEE =	Standard Error of Estimate	EY =	Equivalent Years of Record

PREDICTION EQUATIONS, STANDARD ERRORS OF ESTIMATE, AND EQUIVALENT YEARS OF RECORD

Figure 29-9E (contd.)

Area	*DA* (mi^2)	SLOPE (ft/mi)	$I_{24,2}$ (in.)	*STOR* (%)	*RC*	LENGTH (mi)	*PREC* (in.)	No. of Gaging Stations
1	0.17 - 3370			0 - 13.3				16
2	0.17 - 1967			0 - 4.1	0.5 - 0.8		34 - 46	31
3	0.31 - 4927	2.0 - 149	2.85 - 3.15				34 - 39	60
4	0.07 - 1224	2.4 - 267	2.80 - 3.30			0.3 - 77.1		46
5	0.04 - 11,125	1.2 - 236						35
6	0.10 - 856		2.70 - 3.00		0.3 - 0.8			32
7	0.39 - 1578	0.9 - 39.7			0.3 - 0.7	1.1 - 78.6		22

RANGE OF AREA-BASIN CHARACTERISTICS
FOR USGS REGRESSION EQUATION

Figure 29-9F

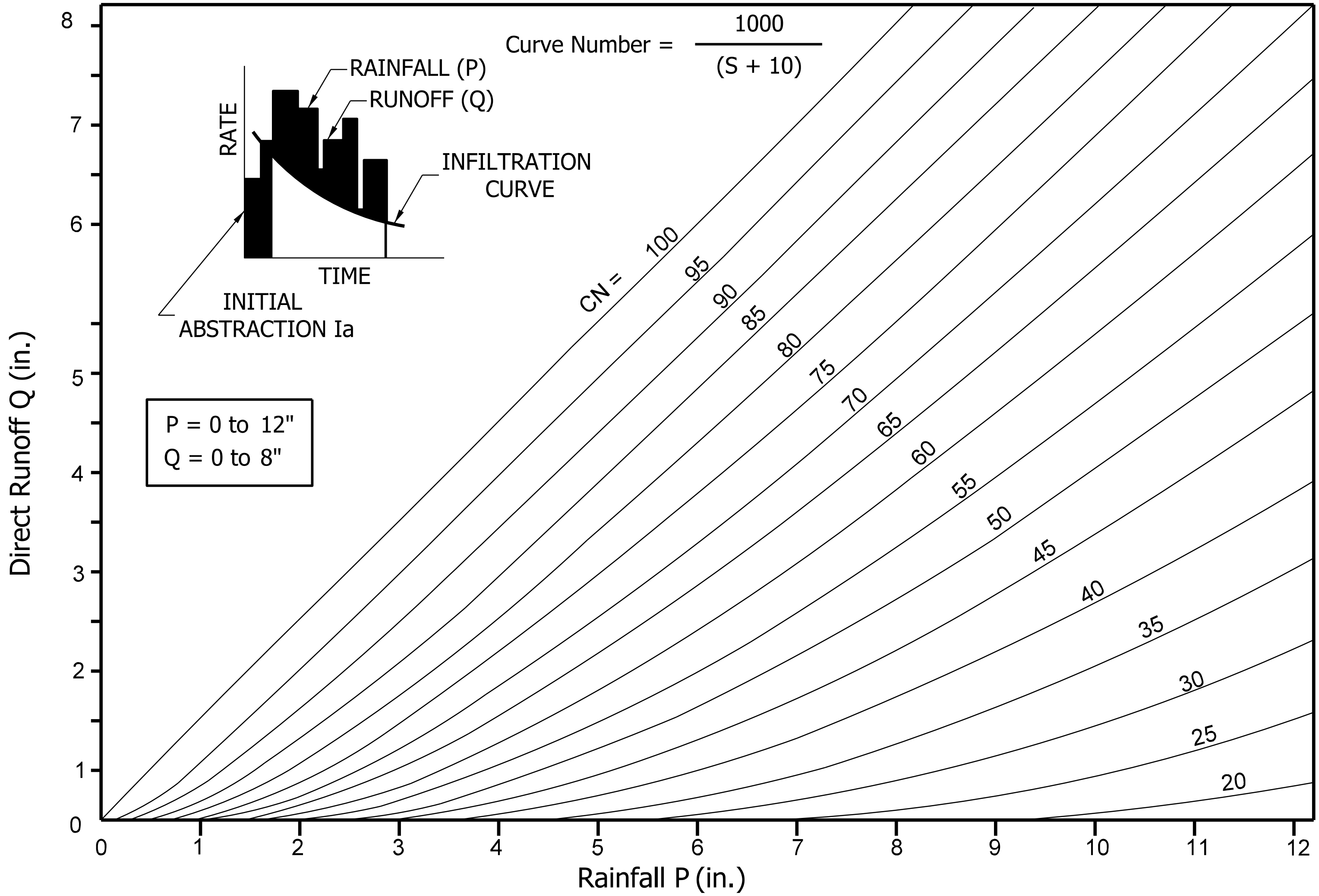
Each Indiana Station Contains Four Quartiles

% Storm Time	Indianapolis				Evansville				Fort Wayne				South Bend			
	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	20.00	6.50	5.26	6.67	22.82	6.28	5.13	6.92	20.00	6.67	6.00	7.14	20.00	7.50	7.00	8.26
20	40.80	18.13	11.55	14.25	44.69	17.33	11.11	14.04	41.11	17.14	12.23	14.23	40.00	18.57	13.33	16.35
30	54.95	35.85	17.06	20.00	57.11	33.33	16.67	20.51	54.83	34.17	18.86	20.00	51.67	34.00	20.00	22.73
40	62.50	52.94	24.24	26.09	65.33	53.09	25.44	27.06	62.00	52.18	26.15	25.71	60.89	51.43	27.50	28.50
50	68.75	67.86	37.78	33.33	71.43	69.57	37.93	34.21	68.42	66.67	38.46	33.33	67.35	66.67	39.13	34.04
60	76.67	76.52	58.33	40.00	78.15	78.57	57.39	40.91	75.00	76.36	57.23	38.00	75.00	75.17	58.46	40.20
70	83.05	83.81	78.03	50.00	84.66	85.60	77.44	50.79	81.62	84.29	76.11	48.50	80.83	82.32	75.98	50.00
80	89.70	90.67	88.68	68.57	90.00	91.72	88.54	69.70	87.50	90.00	87.69	68.24	86.67	88.89	86.79	67.50
90	95.00	95.89	95.29	88.37	95.36	96.50	95.88	89.36	93.75	95.56	95.08	87.88	92.89	94.78	94.17	87.50
100	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

Note: Quartile II is recommended for use.

HUFF DISTRIBUTION OF DESIGN RAINFALL
(50% Probability Curve Ordinates)

Figure 29-10A



NRCS RELATION BETWEEN DIRECT RUNOFF, CURVE NUMBER AND PRECIPITATION

Figure 29-10B

Cover Description	Curve Number for Hydrologic Soil Group				
Cover Type and Hydrologic Condition	Average Percent Impervious Area [2]	A	B	C	D
Fully-developed urban area (vegetation established) Open space (lawn, park, golf course, cemetery, etc.) [2] Poor condition (grass cover < 50%) Fair condition (grass cover 50% to 75%) Good condition (grass cover > 75%)		 68 49 39	 79 69 61	 86 79 74	 89 84 80
Impervious Area: Paved parking lot, roof, drive, etc. (excluding right of way)		98	98	98	98
Street or Road: Paved; curbs and storm drains (excluding right of way) Paved; open ditches (including right of way) Gravel (including right of way) Dirt (including right of way)		98 83 76 72	98 89 85 82	98 92 89 87	98 93 91 89
Urban Area: Commercial or Business Industrial	85 72	89 81	92 88	94 91	95 93
Residential Area by Average Lot Size: 0.12 ac or less (townhouse) 0.25 ac 0.33 ac 0.50 ac 1.00 ac 2.00 ac	65 38 30 25 20 12	77 61 57 54 51 46	85 75 72 70 68 65	90 83 81 80 79 77	92 87 86 85 84 82
Developing Urban Area Newly-graded area (pervious area only, no vegetation)		77	86	91	94
Idle land (CN is determined using cover types similar to those shown in Figure 29-10E).					

[1] *Average runoff condition, and $I_a = 0.2S$*

[2] *The average percent impervious area shown was used to develop the composite CN. Other assumptions are as follows: An impervious area is directly connected to the drainage system and has a CN of 98; a pervious area is considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the NRCS method has an adjustment to reduce the effect.*

[3] *CN shown is equivalent to that for a pasture. A composite CN may be computed for other combinations of open-space cover type.*

RUNOFF CURVE NUMBER FOR URBAN AREA [1]

Figure 29-10C

Cover Description Cover Type and Hydrologic Condition	Curve Number for Hydrologic Soil Group			
	A	B	C	D
Cultivated Land (Row Crops):				
With conservation treatment	62	71	78	81
Without conservation treatment	72	81	88	91
Pasture or Range Land:				
Poor condition	68	79	86	89
Good condition	39	61	74	80
Meadow:				
Good condition	30	58	71	78
Wood or Forest Land:				
Thin stand, poor cover, no mulch	45	66	77	83
Good cover	25	55	70	77

RUNOFF CURVE NUMBER FOR UNDEVELOPED AREA

Figure 29-10D

Cover Description	Curve Number for Hydrologic Soil Group			
Cover Type and Hydrologic Condition	A	B	C	D
Pasture, grassland or range with continuous forage for grazing:				
Poor	68	79	86	89
Fair	49	69	79	84
Good	39	61	74	80
Meadow with continuous grass, protected from grazing and generally mowed for hay	30	58	71	78
Brush/brush-weed-grass mixture with brush being the major element:				
Poor	48	67	77	83
Fair	35	56	70	77
Good	30	48	65	73
Woods and grass combination (orchard or tree farm):				
Poor	57	73	82	86
Fair	43	65	76	82
Good	32	58	72	79
Woods:				
Poor	45	66	77	83
Fair	36	60	73	79
Good	30	55	70	77
Farmsteads	59	74	82	86

RUNOFF CURVE NUMBER FOR AGRICULTURAL LAND

Figure 29-10E

CN for Average Condition	Corresponding CN	
	Dry	Wet
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50
25	12	43
15	6	30
5	2	13

Source: USDA Natural Resources Conservation Service TP-149 (NRCS-TP-149), A Method for Estimating Volume and Rate of Runoff in Small Watersheds, *revised April 1973.*

CONVERSION FROM AVERAGE ANTECEDENT MOISTURE CONDITIONS

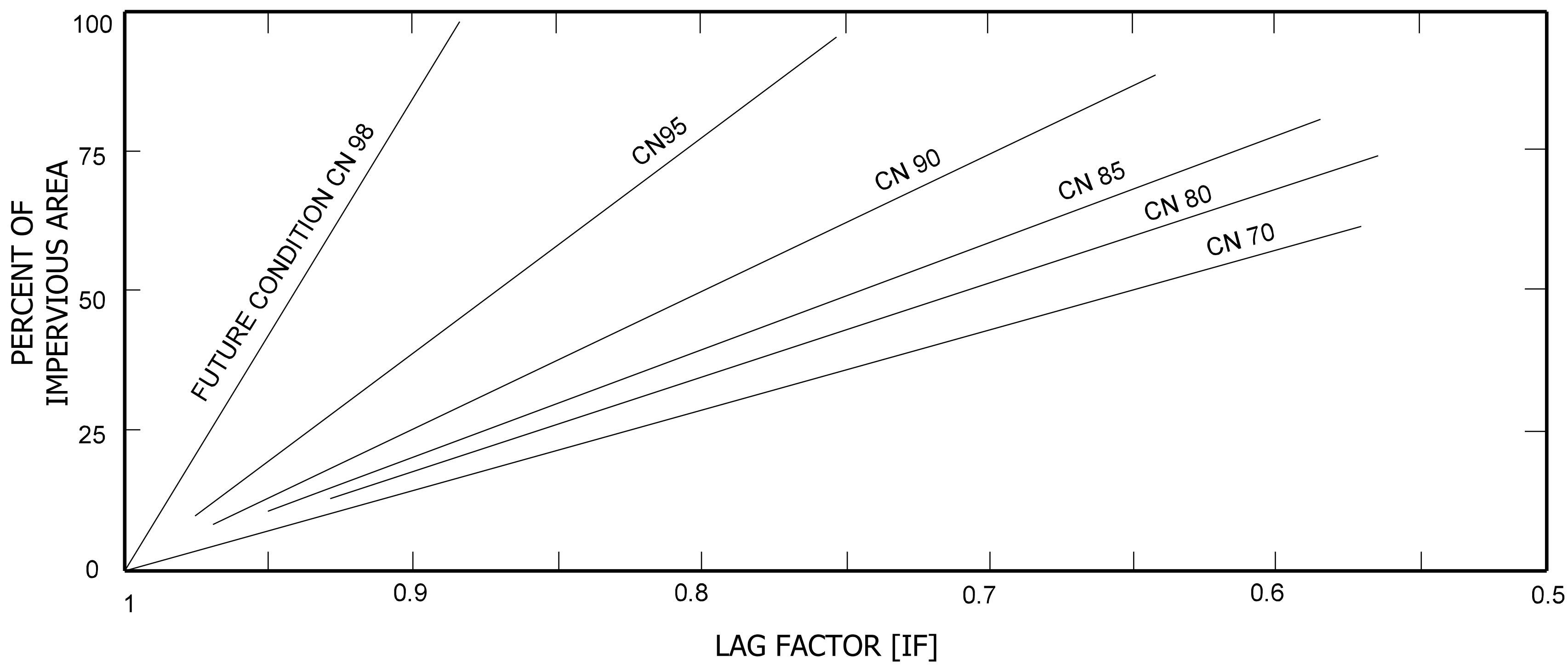
Figure 29-10F

Antecedent Condition	Condition Description	Growing Season 5-day Antecedent Rainfall	Dormant Season 5-day Antecedent Rainfall
Dry	An optimum condition of watershed soils, where soils are dry but not to the wilting point and when satisfactory plowing or cultivation takes place	Less than 1.5 in.	Less than 0.5 in.
Average	The average situation for an annual flood	1.5 in. ≤ rainfall < 2 in.	0.5 in. ≤ rainfall < 1 in.
Wet	Where a heavy rainfall, or light rainfall and low temperatures, have occurred during the five days previous to a given storm	Over 2 in.	Over 1 in.

Source: Natural Resources Conservation Service

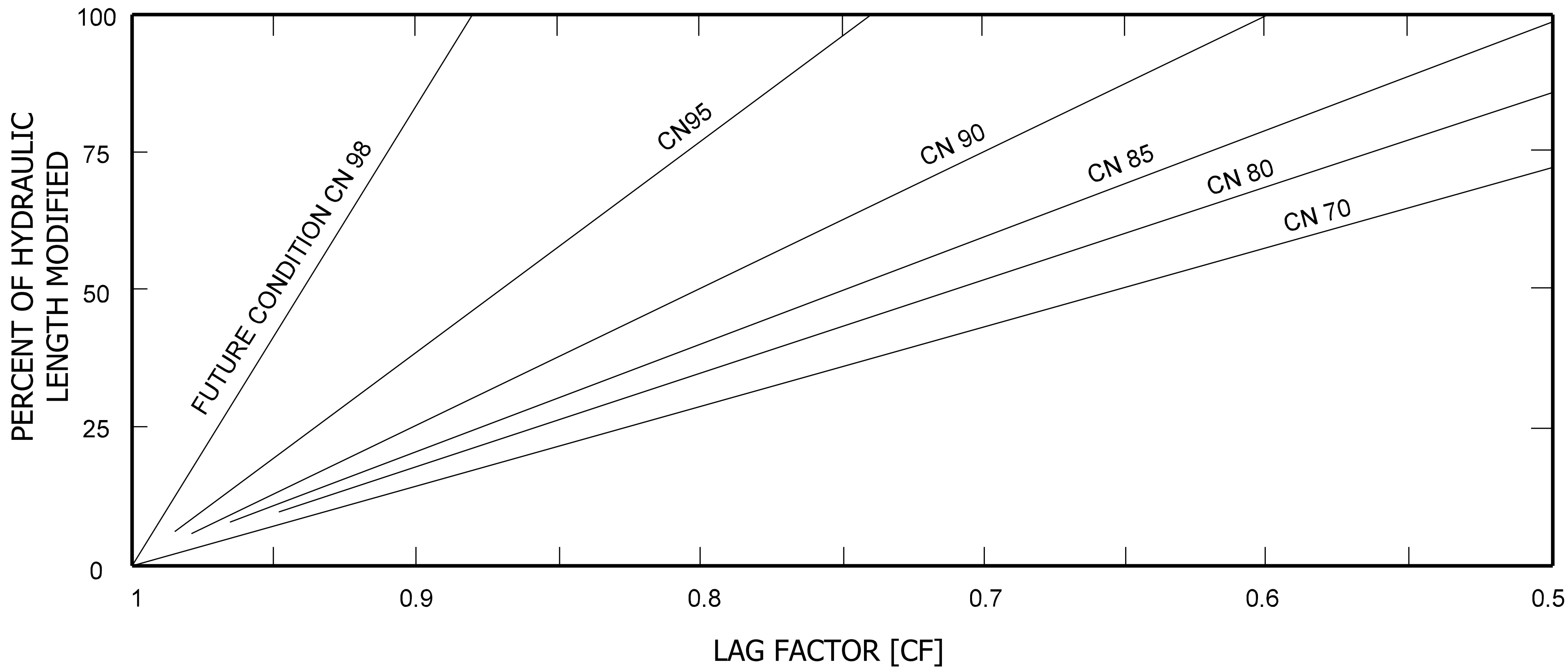
RAINFALL GROUP FOR ANTECEDENT SOIL MOISTURE CONDITION DURING GROWING OR DORMANT SEASON

Figure 29-10G



FACTORS FOR ADJUSTING LAG
(When Impervious Areas Occur in Watershed)

Figure 29-10H



FACTORS FOR ADJUSTING LAG
(When Main Channel Has Been Hydraulically Improved)

Figure 29-10I

```
******************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY*****************

JOB TR-20                    FULLPRINT         SUMMARY
TITLE 123 EXAMPLE PROBLEM  LICK CREEK AND LITTLE LICK CREEK AT US 150
TITLE 000 100 YEAR STORMS   HUFF 2ND QUARTILE FOR EVANSVILLE
 5 RAINFL 8               .05
 8          0.000        0.040        0.063        0.120        0.173
 8          0.260        0.333        0.430        0.530        0.610
 8          0.696        0.740        0.786        0.820        0.856
 8          0.880        0.917        0.950        0.965        0.980
 8          1.00         1.000        1.000        1.000        1.000
 9 ENDTBL
 6 RUNOFF 1 001       5 1.51         75.2         0.90         1 0    0 0 1
 6 RUNOFF 1 001       6 0.59         78.5         0.60         1 0    0 0 1
 6 ADDHYD 4 001    5 6 7                                       1 0    0 0 1
   ENDATA
 7 INCREM 6               .1
 7 COMPUT 7 001    001    0.         1.67         0.25         8 2   01  01
   ENDCMP 1
 7 COMPUT 7 001    001    0.         2.37         0.5          8 2   02  02
   ENDCMP 1
 7 COMPUT 7 001    001    0.         3.10         1.           8 2   03  03
   ENDCMP 1
 7 COMPUT 7 001    001    0.         3.48         2.           8 2   04  04
   ENDCMP 1
   ENDJOB 2
0*****************************END OF 80-80 LIST********************************
1


 TR20 XEQ 03-25-98  12:23        EXAMPLE PROBLEM  LICK CREEK AND LITTLE LICK CREEK AT US 150                    JOB  1    PASS   1
      REV PC 09/83(.2)           100 YEAR STORMS   HUFF 2ND QUARTILE FOR EVANSVILLE                                       PAGE   1


 EXECUTIVE CONTROL OPERATION INCREM                                                                          RECORD ID
+                                        MAIN TIME INCREMENT =    .10 HOURS


 EXECUTIVE CONTROL OPERATION COMPUT                                                                          RECORD ID
+                                        FROM XSECTION    1
+                                                            TO XSECTION    1
        STARTING TIME =    .00     RAIN DEPTH =  1.67    RAIN DURATION=    .25    RAIN TABLE NO.= 8    ANT. MOIST. COND= 2
        ALTERNATE NO.= 1           STORM NO.= 1    MAIN TIME INCREMENT =    .10 HOURS


 OPERATION RUNOFF   CROSS SECTION    1
           OUTPUT HYDROGRAPH=  5
           AREA=    1.51 SQ MI    INPUT RUNOFF CURVE= 75.    TIME OF CONCENTRATION=    .90 HOURS
           INTERNAL HYDROGRAPH TIME INCREMENT=  .0125 HOURS

           PEAK TIME(HRS)               PEAK DISCHARGE(CFS)            PEAK ELEVATION(FEET)
                .76                          285.37                         (RUNOFF)

   RUNOFF VOLUME ABOVE BASEFLOW =    .24 WATERSHED INCHES,     230.90 CFS-HRS,     19.08 ACRE-FEET;    BASEFLOW =     .00 CFS


 OPERATION RUNOFF   CROSS SECTION    1
           OUTPUT HYDROGRAPH=  6
           AREA=     .59 SQ MI    INPUT RUNOFF CURVE= 79.    TIME OF CONCENTRATION=    .60 HOURS
           INTERNAL HYDROGRAPH TIME INCREMENT=  .0125 HOURS

           PEAK TIME(HRS)               PEAK DISCHARGE(CFS)            PEAK ELEVATION(FEET)
                .55                          225.53                         (RUNOFF)

   RUNOFF VOLUME ABOVE BASEFLOW =    .33 WATERSHED INCHES,     124.14 CFS-HRS,     10.26 ACRE-FEET;    BASEFLOW =     .00 CFS


 OPERATION ADDHYD   CROSS SECTION    1
           INPUT HYDROGRAPHS= 5,6      OUTPUT HYDROGRAPH= 7

           PEAK TIME(HRS)               PEAK DISCHARGE(CFS)            PEAK ELEVATION(FEET)
                .65                          475.30                         (NULL)

   RUNOFF VOLUME ABOVE BASEFLOW =    .26 WATERSHED INCHES,     355.05 CFS-HRS,     29.34 ACRE-FEET;    BASEFLOW =     .00 CFS


 EXECUTIVE CONTROL OPERATION ENDCMP                                                                          RECORD ID
+                                        COMPUTATIONS COMPLETED FOR PASS    1


1


 TR20 XEQ 03-25-98  12:23        EXAMPLE PROBLEM  LICK CREEK AND LITTLE LICK CREEK AT US 150                    JOB  1    PASS   2
      REV PC 09/83(.2)           100 YEAR STORMS   HUFF 2ND QUARTILE FOR EVANSVILLE                                       PAGE   2
```

TR 20 EXAMPLE PROBLEM

Figure 29-10J

```
 EXECUTIVE CONTROL OPERATION COMPUT                                                                          RECORD ID
+                                    FROM XSECTION   1
+                                                          TO XSECTION   1
       STARTING TIME =    .00    RAIN DEPTH =  2.37    RAIN DURATION=    .50    RAIN TABLE NO.= 8    ANT. MOIST. COND= 2
       ALTERNATE NO.= 2          STORM NO.= 2     MAIN TIME INCREMENT =    .10 HOURS

 OPERATION RUNOFF    CROSS SECTION    1
          OUTPUT HYDROGRAPH=  5
          AREA=   1.51 SQ MI    INPUT RUNOFF CURVE= 75.    TIME OF CONCENTRATION=    .90 HOURS
          INTERNAL HYDROGRAPH TIME INCREMENT=  .0250 HOURS

          PEAK TIME(HRS)              PEAK DISCHARGE(CFS)              PEAK ELEVATION(FEET)
              .90                          679.29                          (RUNOFF)

  RUNOFF VOLUME ABOVE BASEFLOW =    .58 WATERSHED INCHES,     569.28 CFS-HRS,     47.05 ACRE-FEET;    BASEFLOW =     .00 CFS

 OPERATION RUNOFF    CROSS SECTION    1
          OUTPUT HYDROGRAPH=  6
          AREA=    .59 SQ MI    INPUT RUNOFF CURVE= 79.    TIME OF CONCENTRATION=    .60 HOURS
          INTERNAL HYDROGRAPH TIME INCREMENT=  .0250 HOURS

          PEAK TIME(HRS)              PEAK DISCHARGE(CFS)              PEAK ELEVATION(FEET)
              .70                          464.96                          (RUNOFF)

  RUNOFF VOLUME ABOVE BASEFLOW =    .73 WATERSHED INCHES,     277.23 CFS-HRS,     22.91 ACRE-FEET;    BASEFLOW =     .00 CFS

 OPERATION ADDHYD    CROSS SECTION    1
          INPUT HYDROGRAPHS= 5,6       OUTPUT HYDROGRAPH= 7

          PEAK TIME(HRS)              PEAK DISCHARGE(CFS)              PEAK ELEVATION(FEET)
              .80                         1061.99                          (NULL)

  RUNOFF VOLUME ABOVE BASEFLOW =    .62 WATERSHED INCHES,     846.52 CFS-HRS,     69.96 ACRE-FEET;    BASEFLOW =     .00 CFS

 EXECUTIVE CONTROL OPERATION ENDCMP                                                                          RECORD ID
+                                    COMPUTATIONS COMPLETED FOR PASS    2

1

 TR20 XEQ 03-25-98  12:23       EXAMPLE PROBLEM  LICK CREEK AND LITTLE LICK CREEK AT US 150                  JOB  1    PASS   3
     REV PC 09/83(.2)           100 YEAR STORMS   HUFF 2ND QUARTILE FOR EVANSVILLE                                     PAGE   3

 EXECUTIVE CONTROL OPERATION COMPUT                                                                          RECORD ID
+                                    FROM XSECTION   1
+                                                          TO XSECTION   1
       STARTING TIME =    .00    RAIN DEPTH =  3.10    RAIN DURATION=  1.00    RAIN TABLE NO.= 8    ANT. MOIST. COND= 2
       ALTERNATE NO.= 3          STORM NO.= 3     MAIN TIME INCREMENT =    .10 HOURS

 OPERATION RUNOFF    CROSS SECTION    1
          OUTPUT HYDROGRAPH=  5
          AREA=   1.51 SQ MI    INPUT RUNOFF CURVE= 75.    TIME OF CONCENTRATION=    .90 HOURS
          INTERNAL HYDROGRAPH TIME INCREMENT=  .0500 HOURS

          PEAK TIME(HRS)              PEAK DISCHARGE(CFS)              PEAK ELEVATION(FEET)
             1.18                         1037.90                          (RUNOFF)

  RUNOFF VOLUME ABOVE BASEFLOW =  1.04 WATERSHED INCHES,    1011.27 CFS-HRS,     83.57 ACRE-FEET;    BASEFLOW =     .00 CFS

 OPERATION RUNOFF    CROSS SECTION    1
          OUTPUT HYDROGRAPH=  6
          AREA=    .59 SQ MI    INPUT RUNOFF CURVE= 79.    TIME OF CONCENTRATION=    .60 HOURS
          INTERNAL HYDROGRAPH TIME INCREMENT=  .0500 HOURS

          PEAK TIME(HRS)              PEAK DISCHARGE(CFS)              PEAK ELEVATION(FEET)
              .93                          591.65                          (RUNOFF)

  RUNOFF VOLUME ABOVE BASEFLOW =  1.23 WATERSHED INCHES,     468.61 CFS-HRS,     38.73 ACRE-FEET;    BASEFLOW =     .00 CFS

 OPERATION ADDHYD    CROSS SECTION    1
          INPUT HYDROGRAPHS= 5,6       OUTPUT HYDROGRAPH= 7

          PEAK TIME(HRS)              PEAK DISCHARGE(CFS)              PEAK ELEVATION(FEET)
             1.09                         1541.21                          (NULL)

  RUNOFF VOLUME ABOVE BASEFLOW =  1.09 WATERSHED INCHES,    1479.88 CFS-HRS,    122.30 ACRE-FEET;    BASEFLOW =     .00 CFS
```

TR 20 EXAMPLE PROBLEM
(continued)
Figure 29-10J

```
EXECUTIVE CONTROL OPERATION ENDCMP                                                          RECORD ID
+                                        COMPUTATIONS COMPLETED FOR PASS    3

1

TR20 XEQ 03-25-98  12:23        EXAMPLE PROBLEM  LICK CREEK AND LITTLE LICK CREEK AT US 150            JOB  1   PASS    4
     REV PC 09/83(.2)           100 YEAR STORMS   HUFF 2ND QUARTILE FOR EVANSVILLE                                PAGE    4

EXECUTIVE CONTROL OPERATION COMPUT                                                          RECORD ID
+                                        FROM XSECTION   1
+                                                     TO XSECTION   1
        STARTING TIME =    .00    RAIN DEPTH =  3.48   RAIN DURATION=  2.00   RAIN TABLE NO.= 8   ANT. MOIST. COND= 2
        ALTERNATE NO.= 4          STORM NO.= 4    MAIN TIME INCREMENT =   .10 HOURS

OPERATION RUNOFF   CROSS SECTION   1
           OUTPUT HYDROGRAPH=  5
           AREA=   1.51 SQ MI   INPUT RUNOFF CURVE= 75.   TIME OF CONCENTRATION=   .90 HOURS
           INTERNAL HYDROGRAPH TIME INCREMENT=  .1000 HOURS

           PEAK TIME(HRS)                PEAK DISCHARGE(CFS)           PEAK ELEVATION(FEET)
               1.58                          899.88                         (RUNOFF)

  RUNOFF VOLUME ABOVE BASEFLOW =  1.30 WATERSHED INCHES,   1265.85 CFS-HRS,   104.61 ACRE-FEET;   BASEFLOW =    .00 CFS

OPERATION RUNOFF   CROSS SECTION   1
           OUTPUT HYDROGRAPH=  6
           AREA=    .59 SQ MI   INPUT RUNOFF CURVE= 79.   TIME OF CONCENTRATION=   .60 HOURS
           INTERNAL HYDROGRAPH TIME INCREMENT=  .0800 HOURS

           PEAK TIME(HRS)                PEAK DISCHARGE(CFS)           PEAK ELEVATION(FEET)
               1.29                          484.48                         (RUNOFF)

  RUNOFF VOLUME ABOVE BASEFLOW =  1.51 WATERSHED INCHES,    576.72 CFS-HRS,    47.66 ACRE-FEET;   BASEFLOW =    .00 CFS

OPERATION ADDHYD   CROSS SECTION   1
           INPUT HYDROGRAPHS= 5,6      OUTPUT HYDROGRAPH= 7

           PEAK TIME(HRS)                PEAK DISCHARGE(CFS)           PEAK ELEVATION(FEET)
               1.48                         1326.73                         (NULL)

  RUNOFF VOLUME ABOVE BASEFLOW =  1.36 WATERSHED INCHES,   1842.57 CFS-HRS,   152.27 ACRE-FEET;   BASEFLOW =    .00 CFS

EXECUTIVE CONTROL OPERATION ENDCMP                                                          RECORD ID
+                                        COMPUTATIONS COMPLETED FOR PASS    4

EXECUTIVE CONTROL OPERATION ENDJOB                                                          RECORD ID
1

TR20 XEQ 03-25-98  12:23        EXAMPLE PROBLEM  LICK CREEK AND LITTLE LICK CREEK AT US 150            JOB  1   SUMMARY
     REV PC 09/83(.2)           100 YEAR STORMS   HUFF 2ND QUARTILE FOR EVANSVILLE                                PAGE    5

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
              (A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
               A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)
```

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	AMOUNT (IN)	DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	TIME (HR)	RATE (CFS)	RATE (CSM)
ALTERNATE 1 STORM 1													
XSECTION 1	RUNOFF	1.51	8	2	.10	.0	1.67	.25	.24	---	.70	285.37	189.0
XSECTION 1	RUNOFF	.59	8	2	.10	.0	1.67	.25	.33	---	.55	225.53	382.3
XSECTION 1	ADDHYD	2.10	8	2	.10	.0	1.67	.25	.26	---	.65	475.30	226.3
ALTERNATE 2 STORM 2													
XSECTION 1	RUNOFF	1.51	8	2	.10	.0	2.37	.50	.58	---	.90	679.29	449.9
XSECTION 1	RUNOFF	.59	8	2	.10	.0	2.37	.50	.73	---	.70	464.96	788.1
XSECTION 1	ADDHYD	2.10	8	2	.10	.0	2.37	.50	.62	---	.80	1061.99	505.7
ALTERNATE 3 STORM 3													

TR 20 EXAMPLE PROBLEM
(continued)
Figure 29-10J

XSECTION	1	RUNOFF	1.51	8	2	.10	.0	3.10	1.00	1.04	---	1.18	1037.90	687.3
XSECTION	1	RUNOFF	.59	8	2	.10	.0	3.10	1.00	1.23	---	.93	591.65	1002.8
XSECTION	1	ADDHYD	2.10	8	2	.10	.0	3.10	1.00	1.09	---	1.09	1541.21	733.9

ALTERNATE 4 STORM 4
+

XSECTION	1	RUNOFF	1.51	8	2	.10	.0	3.48	2.00	1.30	---	1.58	899.88	595.9
XSECTION	1	RUNOFF	.59	8	2	.10	.0	3.48	2.00	1.51	---	1.29	484.48	821.2
XSECTION	1	ADDHYD	2.10	8	2	.10	.0	3.48	2.00	1.36	---	1.48	1326.73	8

1

TR20 XEQ 03-25-98 12:23 EXAMPLE PROBLEM LICK CREEK AND LITTLE LICK CREEK AT US 150 JOB 1 SUMMARY
REV PC 09/83(.2) 100 YEAR STORMS HUFF 2ND QUARTILE FOR EVANSVILLE PAGE 6

SUMMARY TABLE 3 - DISCHARGE (CFS) AT XSECTIONS AND STRUCTURES FOR ALL STORMS AND ALTERNATES

XSECTION/ STRUCTURE ID	DRAINAGE AREA (SQ MI)	STORM NUMBERS.......... 1	2	3	4
0 XSECTION 1	2.10				
+ ALTERNATE 1		475.30	.00	.00	.00
ALTERNATE 2		.00	1061.99	.00	.00
ALTERNATE 3		.00	.00	1541.21	.00
ALTERNATE 4		.00	.00	.00	1326.73

1END OF 1 JOBS IN THIS RUN

TR 20 EXAMPLE PROBLEM
(continued)
Figure 29-10J

CHAPTER 203

Hydraulics and Drainage Design (Pre-Rewrite Version)

Design Memorandum	**Revision Date**	**Publication Date***	**Sections Affected**
12-18	July 2012	Jan. 2013	Ch. 203

*Revisions will appear in the next published edition of the *Indiana Design Manual*.

TABLE OF CONTENTS

LIST OF FIGURES

CHAPTER THIRTY

CHANNELS

30-1.0 INTRODUCTION

30-1.01 Definitions

An open channel is a natural or constructed 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.

The types of open channels related to a transportation facility are as follows:

1. stream channels;
2. roadside channels or ditches;
3. interceptor ditches; and
4. drainage ditches.

The principles of open-channel-flow hydraulics are applicable to each drainage facility including a culvert or a storm drain.

A stream channel has the properties as follows:

1. is a natural channel with its size and shape determined by natural forces;

2. is compound in cross section with a main channel for conveying low flow and a floodplain to transport flood flow, and

3. is shaped geomorphologically by the long-term history of sediment load and water discharge which it experiences.

An artificial channel can be a roadside channel, interceptor ditch, or drainage ditch which can be a constructed channel with regular geometric cross section, and is unlined or lined with artificial or natural material to protect against erosion.

Although the principles of open-channel flow are the same regardless of the channel type, a stream channel and an artificial channel (primarily a roadside channel) will be treated separately in this Chapter as needed.

30-1.02 Significance [Rev. Jan. 2011]

Channel analysis is necessary for the design of a transportation drainage system to assess the following:

1. potential flooding caused by changes in water-surface profile;
2. disturbance of the river system upstream or downstream of the highway right of way;
3. changes in lateral flow distribution;
4. changes in velocity or direction of flow;
5. need for conveyance and disposal of excess runoff; and
6. need for channel lining to prevent erosion.
7. potential impacts to water quality.

30-1.03 Design

Hydraulic design associated with a natural channel or side ditch is a process which selects and evaluates alternatives according to established criteria. These criteria are the standards established by INDOT to ensure that a highway facility satisfies its intended purpose without endangering the structural integrity of the facility itself and without undue adverse effects on the environment or the public welfare.

30-1.04 Purpose

The purpose of this Chapter is as follows:

1. establish INDOT policy;
2. specify design criteria;
3. review design philosophy;
4. outline channel-design procedures; and
5. demonstrate design techniques by means of example problems.

30-1.05 Symbols

To provide consistency within this Chapter and throughout this *Manual*, see Figure 30-1A, Symbols for Open Channels.

30-2.0 POLICY

30-2.01 General

Policy is a set of goals that establish a definite course or method of action and that are selected to guide and determine present and future decisions. Policy is implemented through design criteria established as standards for making decisions. See Section 30-3.0.

30-2.02 Federal Policy

The following Federal policies apply.

1. Channel design or design of a highway facility that impacts a channel should satisfy the policies of the Federal Highway Administration applicable to floodplain management if Federal funding is involved.

2. Federal Emergency Management Agency floodway regulations and Corps of Engineers' wetland restrictions for permits should be satisfied.

30-2.03 INDOT Policy [Rev. Jan. 2011]

The following INDOT policies apply.

1. Coordination with other Federal, State, or local agencies concerned with water-resources planning should have high priority in the planning of a highway facility.

2. The safety of the general public should be a consideration in selection of the cross-sectional geometry of an artificial drainage channel.

3. The design of an artificial drainage channel or other facility should consider the frequency and type of maintenance expected, and should make allowance for the access of maintenance equipment.

4. A stable channel is the goal for each channel that is located on highway right of way or that impacts a highway facility.

5. The environmental impacts of channel modification, including disturbance of fish habitat, wetlands, water quality, or channel stability, should be assessed.

6. The range of design-channel discharges should be selected and approved by the designer based on roadway functional classification, the consequences of traffic interruption, flood hazard risks, economics, or local site conditions.

30-3.0 DESIGN CRITERIA

30-3.01 General

The design criteria establish the standards by which a policy is placed into action. They form the basis for the selection of the final design configuration. Listed below are examples of design criteria which should be considered for channel design.

30-3.02 Stream Channel

The following criteria apply to a natural channel.

1. The hydraulic effects of floodplain encroachment should be evaluated over a full range of frequency-based peak discharges from the mean annual or bank full flood through the 500-year flood on a major highway facility as deemed necessary by the designer.

2. If relocation of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope satisfy the existing conditions insofar as practical. A means of energy dissipation may be necessary where existing conditions cannot be duplicated. See Chapter Thirty-four.

3. Stream bank stabilization should be provided, where appropriate, as a result of a stream disturbance such as encroachment, and should include both upstream and downstream banks and the local site.

4. Incorporate provisions into the design and construction for access by maintenance personnel and equipment to maintain features such as a dike or a levee.

5. Realignment or change to a natural channel should be minimized. The conditions that warrant a channel change are as follows:

   a. the natural channel crosses the roadway at an extreme skew;

   b. the embankment encroaches on the channel;

   c. the natural channel has inadequate capacity; or

   d. the location of the natural channel endangers the highway embankment or adjacent property.

   The most important factor in channel design is the effect of scour and siltation.

30-3.03 Roadside Channel, or Side Ditch

A roadside channel is a channel, or side ditch, adjacent to the roadway which intercepts runoff and groundwater within the right of way and transports this flow to drainage structures or to a natural waterway.

> *** PRACTICE POINTER ***
>
> If a property owner has a pipe instead of an open ditch on the property, an equivalent new pipe should be provided instead of an open ditch.

30-3.03(01) General

The following criteria apply to a roadside channel.

1. Safety. Clear-zone requirements should be satisfied (see Chapter Forty-nine). Channel side slopes should not exceed the angle of repose of the soil or lining, and should be 2H:1V or flatter for rock riprap lining. See Chapter Forty-five for more information on the cross section of a roadside channel.

2. Design Discharge. The design discharge for permanent roadside channel lining should have a 10-year frequency flow. A temporary lining should be designed for a 2-year frequency flow.

3. Freeboard. If an inadequate freeboard is provided, the depth of flow may exceed d_{max} and, thereby, increase the potential for scour in the channel. In addition, freeboard provides a margin of safety against channel overtopping and its consequences. Channel freeboard should be 6 to 12 in. or two velocity heads, whichever is greater, measured vertically. This should be adequate for a small drainage channel. However, more freeboard may be appropriate, or, no freeboard may be necessary.

30-3.03(02) Channel Lining

The selection of a roadside-channel lining must reflect both initial costs and long-term maintenance costs. The channel lining should be selected based on the method of allowable tractive force. This is discussed in Section 30-6.03. The following provides the INDOT practice for roadside-channel lining. However, the use of these criteria should be confirmed using the lining-selection methodology described in Section 30-6.03:

1. <u>Seeded Channel (G < 1%)</u>. A seeded channel is protected from erosion by means of fast-growing permanent seeding. This type of channel has the advantage of being low in initial cost and maintenance, aesthetically pleasing, and compatible with the natural environment. The use of an erosion control mat (e.g., straw, coconut fiber) is encouraged to help establish seed growth.

2. <u>Sod-Lined Channel (1% ≤ G < 3%)</u>. A sod-lined channel is protected from erosion by means of a sod cover. It is used as a roadside channel in a median or at a channel change of a small watercourse. It may also be used on steeper grades where ditch flow is a minimum. A sodded channel has the advantage of being low in initial cost, aesthetically pleasing, and compatible with the natural environment. This type of channel should be selected for use wherever practical. A sodded channel should be sodded to a point 1 ft above the flow line.

3. <u>Paved Channel (G ≥ 3%)</u>. A paved concrete ditch is extremely resistant to erosion. Its principal disadvantages are its high maintenance and initial costs, susceptibility to failure if undermined by scour, and the tendency for scour to occur downstream due to an acceleration of flow.

   The INDOT *Standard Drawings* illustrate the paved channels used by the Department. Type A through H is used where the toe of the ditch is outside of the clear zone. Type J through M is used where the toe of the ditch is inside the clear zone. For Type J through M, place the 6H:1V sideslope nearest to the roadway. The INDOT *Standard Drawings* also indicate the type of paved channel that should be used based on the diameter of the pipe at the outlet and inlet.

4. <u>Riprap-Lined Channel (3% ≤ G ≤ 10%)</u>. A riprap lining is effective for this slope range, depending on the design flow of the channel. However, riprap should be used on a slope steeper than 10% at a bridge cone. It is also appropriate to use riprap in a ditch where the grade is flatter than 3%. For example, if there is a hill in the ditch watershed, riprap should be placed to dissipate energy and minimize ditch erosion. A mild slope is constructed by dumping riprap into a prepared channel lined with geotextile filter cloth and grading to the desired shape. The advantages are low construction and maintenance costs and self-healing characteristics. Riprap has limited application on a steep slope where the flow will tend to displace the lining material.

5. <u>Non-Erodible Channel (3% ≤ G ≤ 15%)</u>. A non-erodible channel has a lining of soil erosion matting that is highly resistant to erosion. This type of channel is moderately expensive to construct and, if properly designed, should have a very low maintenance cost.

The lining material should extend to the top of the channel or to at least 6 in. above the design water level measured vertically.

30-3.04 Drainage Ditch

A drainage ditch is a channel which is not immediately adjacent to the roadway but which is part of the roadway facility's overall drainage system. It includes, for example, an interceptor ditch, which is located in the natural ground near the top edge of a cut slope or down the backslope in a cut section from a small natural drainage course from outside the right of way. The criteria for a roadside channel are more stringent than those for a drainage ditch. Therefore, such criteria are listed separately. This is true for a ditch that may be located off the highway right of way.

The following criteria apply to a drainage ditch.

1. An unlined drainage ditch is considered erodible and should have a capacity equal to, or greater than, the 10-year frequency design flow with a velocity equal to or less than the maximum allowable velocity shown in Figure 30-3A, Maximum Velocity in Drainage Ditch.

2. A bend in an erodible drainage ditch should have a minimum radius to the center of the ditch of three times the bottom width. This will tend to minimize the scouring effect at the bend.

3. An erodible drainage ditch may need scour protection where it is located adjacent to a highway embankment, especially at bends in the ditch. Riprap or other suitable protection should be provided where necessary.

30-3.05 Other Criteria

The following applies to a roadside channel or other type of drainage ditch.

1. Transition. A paved-side-ditch transition is required at an intersection with an earth ditch or pipe culvert.

2. Cut-Off Wall. A cut-off wall is required at the beginning and end of each paved side ditch.

3. Lug. Lugs have been proven to prevent sliding on a steep slope. A lug should be provided at the locations as follows:

   a. 10 ft downslope from a grade change;
   b. 10 ft downslope from the intersection of different types of paved side ditches;
   c. at the downslope end of a transition between different types of paved side ditches; and
   d. as shown in Figure 30-3B, Lug Intervals.

30-4.0 OPEN-CHANNEL FLOW

30-4.01 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 can be applied to open-channel flow with the additional complication that the position of the free surface is one of the unknown variables. The determination of this unknown is one of the principal problems of open-channel flow analysis and it depends on quantification of the flow resistance. A natural channel displays a much wider range of roughness values than an artificial channel.

30-4.02 Definitions

1. Specific Energy. Specific energy, E, is defined as the energy head relative to the channel bottom. If the channel slope less than 10% and the stream lines are nearly straight and parallel (so that the hydrostatic assumption holds), E becomes the sum of the depth and velocity head, as follows:

$$E = \frac{\alpha V^2}{2g} + y \qquad \text{(Equation 30-4.1)}$$

Where: y = depth, ft
α = velocity distribution coefficient (see Equation 30-4.2)
V = mean velocity, ft/s
g = gravitational acceleration, 32.2 ft/s^2

2. Velocity Distribution Coefficient. Due to the presence of a free surface and also due to friction along the channel boundary, the velocities in a channel are not uniformly distributed in the channel section. As a result of non-uniform distribution of velocities in a channel section, the velocity head of an open channel is greater than the average velocity head computed as $(Q/A_t)/2g$. A weighted-average value of the velocity head is obtained by multiplying the average velocity head, above, by a velocity distribution coefficient, α, defined as follows:

$$\alpha = \sum_{i=1}^{n} \left(\frac{K^3{}_i}{A^2{}_i} \right) \Bigg/ \left(\frac{K^3{}_t}{A^2{}_t} \right) \qquad \text{(Equation 30-4.2)}$$

Where: K_i = conveyance in subsection (see Equation 30-4.8)
K_t = total conveyance in section (see Equation 30-4.8)

A_i = cross-sectional area of subsection, ft^2
A_t = total cross-sectional area of section, ft^2
n = number of subsections

The velocity distribution coefficient should be taken as 1 for turbulent flow in a prismatic channel, but may be different in a natural channel.

3. Total Energy Head. The total energy head is the specific energy head plus the elevation of the channel bottom with respect to a datum. The locus of the energy head from one cross section to the next defines the energy grade line. See Figure 30-4A, Specific Energy and Discharge Diagram for Rectangular Channel.

4. Steady or Unsteady Flow. A steady flow is one in which the discharge passing a given cross section is constant with respect to time. The maintenance of steady flow in a reach requires that the rates of inflow and outflow be constant and equal. Where the discharge varies with time, the flow is unsteady.

5. Uniform Flow or Non-uniform Flow. A non-uniform flow is one in which the velocity and depth vary in the direction of motion while they remain constant in uniform flow. Uniform flow can only occur in a prismatic channel, which is a channel of constant cross section, roughness, and slope in the flow direction. However, non-uniform flow can occur either in a prismatic channel or in a natural channel with variable properties.

6. Gradually-Varied or Rapidly-Varied Flow. A non-uniform flow, in which the depth and velocity change gradually enough in the flow direction that vertical accelerations can be neglected, is referred to as a gradually-varied flow. Otherwise, it is considered to be rapidly-varied.

7. Froude Number. The Froude number, Fr, is a dimensionless parameter in open-channel flow. It represents the ratio of inertial forces to gravitational forces and is defined as follows:

$$Fr = \frac{Va}{(gd)^{0.5}} \qquad \text{(Equation 30-4.3)}$$

Where: α = velocity distribution coefficient
V = mean velocity = Q/A, ft/s
g = gravitational acceleration, 32.2 ft/s^2
d = hydraulic depth = A/T, ft
A = cross-sectional area of flow, ft^2
T = channel top width at the water surface, ft
Q = total discharge, ft^3/s

This expression applies to a single-section channel. For a rectangular channel, the hydraulic depth is equal to the flow depth.

8. <u>Critical Flow</u>. Critical flow occurs if the specific energy is a minimum. 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 1. Critical depth is also the depth of maximum discharge if the specific energy is held constant. These relationships are illustrated in Figure 30-4A, Specific Energy and Discharge Diagram for Rectangular Channel. During critical flow, the velocity head is equal to half the hydraulic depth. The expression for flow at critical depth is as follows:

$$\frac{\alpha Q^2}{g} = \frac{A^3}{T} \quad \text{(Equation 30-4.4)}$$

Where: α = velocity distribution coefficient
Q = total discharge, ft^3/s
g = gravitational acceleration, 32.2 ft/s^2
A = cross-sectional area of flow, ft^2
T = channel top width at the water surface, ft

If flow is at critical depth, Equation 30-4.4 must be satisfied, regardless of the shape of the channel.

9. <u>Subcritical Flow</u>. A depth greater than the critical depth occurs in subcritical flow and the Froude number is less than 1. In this state of flow, small water surface disturbances can travel both upstream and downstream, and the control is located downstream.

10. <u>Supercritical Flow</u>. A depth less than the critical depth occurs in supercritical flow and the Froude number is greater than 1. Small water surface disturbances are swept downstream in supercritical flow, and the location of the flow control is always upstream.

11. <u>Hydraulic Jump</u>. 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 a highway-drainage structure.

30-4.03 Flow Classification

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

1. Steady Flow

   a. Uniform Flow
   b. Non-uniform Flow
      (1) Gradually-Varied Flow
      (2) Rapidly-Varied Flow

2. Unsteady Flow

   a. Unsteady Uniform Flow (rare)
   b. Unsteady Non-uniform Flow
      (1) Gradually-Varied Unsteady Flow
      (2) Rapidly-Varied Unsteady Flow

Steady uniform flow and the steady non-uniform flow are the most fundamental types of flow treated in highway-engineering hydraulics.

30-4.04 Equations

The following equations are those used to analyze open-channel flow. The use of these equations in analyzing open-channel hydraulics is discussed in Section 30-5.0.

1. Continuity Equation. The continuity equation is the statement of conservation of mass in fluid mechanics. For one-dimensional, steady flow of an incompressible fluid, it assumes the simple form as follows:

$$Q = A_1 V_1 = A_2 V_2 \qquad \text{(Equation 30-4.5)}$$

Where: Q = discharge, ft^3/s
A = cross-sectional area of flow, ft^2
V = mean cross-sectional velocity perpendicular to the cross section, ft/s

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

2. Manning's Equation. For a given depth of flow in an open channel with a steady, uniform flow, the mean velocity, V, can be computed as follows:

$$V = \frac{R^{0.67} S^{0.5}}{n} \qquad \text{(Equation 30-4.6)}$$

Where: V = velocity, ft/s

n = Manning's roughness coefficient
R = hydraulic radius = A/P, ft
P = wetted perimeter, ft
S = slope of the energy gradeline, ft/ft. For steady uniform flow, S = channel slope
A = cross-sectional area of flow, ft^2

The selection of Manning's *n* is based on observation. However, considerable experience is essential in selecting appropriate *n* value. The selection of Manning's *n* is discussed in Section 30-5.02(01). The range of *n* values for each type of channel and floodplain is shown in Figure 30-4B, Uniform Flow (Value of Manning's *n*).

The continuity equation can be combined with Manning's equation to obtain the steady, uniform flow discharge as follows:

$$Q = [(1.486/n)AR^{2/3}S^{1/2}] \quad \text{(Equation 30-4.7)}$$

For a given channel geometry, slope, and roughness, and a specified value of *Q*, a unique value of depth occurs in steady, uniform flow. It is called normal depth and is computed from Equation 30-4.7 by expressing the area and hydraulic radius in terms of depth. The resulting equation may require a trial-and-error solution. See Section 30-5.03 for a more detailed discussion of the computation of normal depth.

If the normal depth is greater than critical depth, the slope is classified as a mild slope. 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.

3. Conveyance. In channel analysis, it is convenient to group the channel properties into a single term called the channel conveyance, *K*.

$$K = (1.486/n)AR^{2/3} \quad \text{(Equation 30-4.8)}$$

Equation 30-4.7 can then be written as follows:

$$Q = KS^{0.5} \quad \text{(Equation 30-4.9)}$$

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.

Channel conveyance is useful in computing the distribution of overbank flood flows in the stream cross-section, or the flow distribution through the opening in a proposed stream crossing. It is also used to determine the velocity distribution coefficient, ά (see Equation 30-4.2).

4. Energy Equation. The energy equation expresses conservation of energy stated as energy per unit weight of fluid, which has the dimension of length, and is therefore called energy head. The energy head is composed of potential energy head (elevation head), pressure head, and kinetic energy head (velocity head). These energy heads are scalar quantities which give the total energy head at a given cross section once added. Between an upstream open channel cross section designated 1 and a downstream cross section designated 2, the energy equation is as follows:

$$h_1 + \frac{\alpha_1 V_1^2}{2g} = h2 + \frac{\alpha_2 V_2^2}{2g} + h_L \qquad \text{(Equation 30-4.10)}$$

Where: h_1 and h_2 = the upstream and downstream stages, respectively, ft
α = velocity distribution coefficient
V = mean velocity, ft/s
h_L = head loss due to local cross-sectional changes (minor loss) and 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, therefore, $h = z + y$. See Figure 30-4C, Terms in the Energy Equation. 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.

30-5.0 HYDRAULIC ANALYSIS

30-5.01 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 a channel lining or highway-drainage structure.

Two methods are commonly used. The single-section method is a simple application of Manning's equation to determine tailwater rating curves for a culvert or to analyze other situations in which uniform or nearly-uniform flow conditions exist. Manning's equation can be used to estimate the high-water elevation for a bridge that does not constrict the flow. The step-backwater method is used to compute the complete water surface profile in a stream reach to evaluate the unrestricted water-surface elevations for bridge hydraulic design or to analyze other gradually-varied flow problems in a stream.

The single-section method will yield less-reliable results because it requires more judgment and assumptions than the step-backwater method. However, the single-section method is all that is justified (e.g., standard roadway ditch, culvert, storm drain outfall).

30-5.02 Cross Sections

Cross-sectional geometry of a stream is defined by coordinates of lateral distance and ground elevation which locate individual ground points. The cross section is taken normal to the flow direction along a single straight line where possible. In a wide floodplain or bend, it may be necessary to use a section along intersecting straight lines; i.e., a dog-leg section. The cross section should be plotted to reveal inconsistencies or errors.

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

Cross sections should be subdivided with vertical boundaries where there are abrupt lateral changes in geometry or roughness, such as in overbank flow. The conveyances of each subsection are computed separately to determine the flow distribution, and ά and are then added to determine the total flow conveyance. The subsection divisions must be chosen 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 30-5A, Hypothetical Cross Section Showing Reaches, Segments and Subsections Used in Determining *n* Value.

30-5.02(01) Manning's *n* Value Selection

Manning's *n* is affected by many factors, and its selection for a natural channel depends on engineering experience. Pictures of channels and floodplains for which the discharge has been measured and Manning's *n* has been calculated are useful (see Arcement and Schneider, 1984; Barnes, 1978). A more-regimented approach is provided in Arcement and Schneider, 1984. Once the Manning's *n* value has been selected, it should be verified or calibrated with historical high-water marks or gaged streamflow data.

Manning's *n* value for an artificial channel is more easily defined than for a natural stream channel. See Figure 30-4B, Uniform Flow (Value of Manning's *n*), for the *n* value for an artificial channel or a natural stream channel.

30-5.02(02) Calibration

The equations should be calibrated with historical high-water marks or gaged streamflow data 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, cross section. Proper calibration is essential if accurate results are to be obtained.

30-5.02(03) Switchback Phenomenon

If the cross section is improperly subdivided, the mathematics of Manning's equation causes a switchback. A switchback results if the calculated discharge decreases with an associated increase in elevation. This occurs where, 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 which 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 to avoid the switchback.

See Figure 30-5A(1) for the switchback phenomenon.

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

<u>30-5.03 Single-Section Analysis</u>

The single-section analysis method (slope-area method) is 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 an artificial or natural stream channel. Nevertheless, the single-section method is used to design an artificial channel 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. This relationship should include a range of discharges up to at least the base 100-year flood. The stage-discharge curve can be determined as follows.

1. Select the typical cross section at or near the location where the stage-discharge curve is needed.

2. Subdivide the cross section and assign *n* values to subsections as described in Section 30-5.02(01).

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

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

5. Calculate the discharge using Manning's equation for each incremental elevation. 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.

6. After the discharge has been calculated at several incremental elevations, a plot of stage versus discharge should 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 discharges of interest.

An example application of the stage-discharge curve procedure is provided in Section 30-8.0.

Alternatively, a graphical technique such as that shown in Figure 30-5B, Trapezoidal Channel Capacity Chart, or a nomograph as in Figure 30-5C, Nomograph for Normal Depth, can be used for a trapezoidal or prismatic channel. The best approach for a stream channel is to use a computer program such as WSPRO, HEC-RAS or HEC-2 to obtain the normal depth.

In a stream channel, the transverse variation of velocity in a cross section is a function of subsection geometry and roughness, and may vary considerably from one stage and discharge to another. This variation should be considered for the purpose of designing erosion-control measures and locating relief openings in a highway fill, for example. The best method of establishing transverse velocity variations is by means of current foot 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. The energy-grade-line slope is assumed to be 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 $K_tS^{1/2}$, and the discharge in each subsection is proportional to its conveyance. The velocity in each subsection is obtained from the continuity equation, $V = Q/A$.

An alluvial channel provides for 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, the methods outlined in Vanoni, 1977 have been developed for this situation (Einstein-Barbarossa; Kennedy-Alam-Lovera; and Engelund), and should be followed unless it is possible to obtain a measured stage-discharge relation.

There may be a location where a stage-discharge relationship has already been measured in a channel. This usually exists at a gaging station on a stream monitored by the USGS. Measured stage-discharge curves will yield more accurate estimates of water-surface elevation, and should take precedence over the analytical methods described above.

30-5.04 Step-Backwater Analysis

Step-backwater analysis is useful for determining unrestricted water surface profiles where a highway crossing is planned, or for analyzing how far upstream the water-surface elevation is affected by a culvert or bridge. Because the calculations involved in this analysis are tedious and repetitive, a computer program such as the FHWA/USGS program WSPRO or Corps of Engineers HEC-2 should be used.

30-5.04(01) Step-Backwater Models

The WSPRO program has been designed to provide a water-surface profile for the types of open-channel-flow situations as follows:

1. unconstricted flow;
2. single-opening bridge;
3. bridge opening with spur dikes;
4. single-opening embankment overflow;
5. multiple alternatives for a single site; and
6. multiple openings.

The HEC-2 or HEC-RAS programs developed by the Corps of Engineers are used for calculating water surface profile for steady gradually-varied flow in a natural or constructed channel. Both subcritical and supercritical flow profiles can be calculated. The effect of a bridge, culvert, weir, or other structure in the floodplain may be also considered in the computations. This program is also designed for application in a floodplain-management or flood-insurance study.

30-5.04(02) Step-Backwater Methodology

The computation of water-surface profiles by WSPRO, HEC-RAS, or HEC-2 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 30-5.01. Manning's *n* values can vary both horizontally and vertically across the section. 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 clarify the methodology, the energy equation from Section 30-4.04 is repeated below.

$$h_1 + \frac{\alpha_1 V_1^{\,2}}{2g} = h2 + \frac{\alpha_2 V_2^{\,2}}{2g} + h_L \qquad \text{(Equation 30-4.10)}$$

The total head loss is calculated as follows:

$$h_L = K_m \left(\frac{\alpha 1 V_1^{\,2} - \alpha 2 V_2^{\,2}}{2g} \right) + S_f L \qquad \text{(Equation 30-5.2)}$$

Where:
- K_m = expansion or contraction loss coefficient
- $\overline{S}_f$ = the mean slope of the energy grade line evaluated from Manning's equation and a selected averaging technique, ft/ft
- L = discharge-weighted or conveyance-weighted reach length, 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 and 0.1 for a contraction, or 0.5 and 0.3 for an expansion, in WSPRO and HEC-2, respectively. HEC-RAS requires that the user input the value for K_m. The range of these coefficients, from an ideal transition to an abrupt change, is 0.0 to 1.0 for an expansion, or 0.0 to 0.5 for a contraction.

WSPRO calculates a conveyance-weighted reach length, L, as follows:

$$L = \frac{L_{lob}K_{lob} + L_{ch}K_{ch} + L_{rob}K_{rob}}{K_{lob} + K_{ch} + K_{rob}} \qquad \text{(Equation 30-5.3)}$$

Where: L_{lob}, L_{ch}, L_{rob} = flow distance between cross sections in the left overbank, main channel, and right overbank, respectively, ft

K_{lob}, K_{ch}, K_{rob} = conveyance in the left overbank, main channel, and right overbank, respectively, of the cross section with the unknown water-surface elevation

HEC-2 or HEC-RAS calculates a discharge-weighted reach length, L, as follows:

$$L = \frac{L_{lob}Q_{lob} + L_{ch}Q_{ch} + L_{rob}Q_{rob}}{Q_{lob} + Q_{ch} + Q_{rob}} \qquad \text{(Equation 30-5.4)}$$

Where: L_{lob}, L_{ch}, L_{rob} = flow distance between cross sections in the left overbank, main channel, and right overbank, respectively, ft

Q_{lob}, Q_{ch}, Q_{rob} = arithmetic average of flows between cross sections for the left overbank, main channel, and right overbank, respectively, ft^3/s

WSPRO, HEC-RAS, or HEC-2 allows the user the following options for determining the friction slope, S_f.

1. Average conveyance equation:

$$S_f = \left[\frac{(Q_u + Q_d)}{K_u + K_d}\right]^2 \qquad \text{(Equation 30-5.5)}$$

2. Average friction slope equation:

$$S_f = \frac{S_{fu} + S_{fd}}{2} \qquad \text{(Equation 30-5.6)}$$

3. Geometric mean friction slope equation:

$$S_f = \sqrt{S_{fu} S_{fd}} \qquad \text{(Equation 30-5.7)}$$

4. Harmonic mean friction slope equation:

$$S_f = \frac{2 S_{fu} S_{fd}}{S_{fu} + S_{fd}} \qquad \text{(Equation 30-5.8)}$$

Where: Q_u , Q_d = discharge at the upstream and downstream cross sections, respectively, ft^3/s

K_u , K_d = conveyance at the upstream and downstream cross sections, respectively

S_{fu} , S_{fd} = friction slope at the upstream and downstream cross sections, respectively, ft/ft

The default option is the geometric mean friction slope equation in WSPRO and the average conveyance equation in HEC-2 and HEC-RAS.

30-5.04(03) Profile Computation

Water surface profile computation requires a beginning value of elevation or depth (boundary condition) and proceeds upstream for subcritical flow, or downstream for supercritical flow. For supercritical flow, critical depth is often the boundary condition at the control section. For subcritical flow, uniform flow and normal depth may be the boundary condition. The starting depth can 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. The 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, the stream reach may need to be extended downstream, a shorter cross section interval should be used, or the range of starting water-surface elevations should be adjusted. A plot of the convergence profiles can be a useful tool in such an analysis (see Figure 30-5D, Profile Convergence Pattern Backwater Computations).

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 ensure accurate results at the structure site, and continued a sufficient distance upstream to accurately determine the impact of the structure on upstream water-surface profiles (see Figure 30-5E, Profile Study Limits).

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

$$L_d = \frac{1.2(HD)^{0.8}}{S} \qquad \text{(Equation 30-5.9)}$$

$$L_u = 1000[(HD^{0.6})(HL^{0.5})]/S \qquad \text{(Equation 30-5.10)}$$

Where:

L_d = downstream study length along main channel for normal-depth starting condition, ft

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

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

S = average reach slope, ft/ft

HL = head loss ranging between 0.50 ft and 5.0 ft at the channel crossing structure for the 1-percent-chance flood, ft

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

The computation procedure is as follows.

A sample procedure 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 30-5F. An editable version of this form may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/. 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 include calculations for the velocity distribution across the section; Columns 15 through 17 include the average kinetic energy; Column 18 includes calculations for other losses (expansion and contraction losses due to interchanges between kinetic and potential energies as the water flows); and Column 19 includes 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. Distance in miles should be shown upstream from the mouth.

Column 2. ASSUMED is the assumed water-surface elevation which must agree with the resulting computed water-surface elevation within ± 0.05 ft, or an 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-sectional area. If the section is complex and has been subdivided into parts (e.g., left overbank, channel, and right overbank) use one line of the form for each subsection, then sum them to get the total area of cross section, A_t.

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

Column 6. $R^{2/3}$ is two-thirds power of hydraulic radius.

Column 7. n is Manning roughness coefficient.

Column 8. K is conveyance and is defined as follows:

$$K = \frac{C_m A R^{0.67}}{n}$$

where C_m should be taken as 1.486. If the cross section is complex, sum the subsection K values to get K_t.

Column 9. $\overline{K}_t$ is average conveyance for the reach and is calculated as $0.5(K_{td} + K_{tu})$ where subscripts d and u refer to downstream and upstream ends of the reach, respectively.

Column 10. $\overline{S}_f$ is the average slope through the reach and is determined as follows:

$$S_f = \left(\frac{Q}{K_t}\right)^2$$

Column 11. L is the discharge-weighted or conveyance-weighted reach length.

Column 12. h_f is energy loss due to friction through the reach and is calculated as follows:

$$h_f = L\left(\frac{Q}{K_t}\right)^2 = LS_f$$

Column 13. $\Sigma(K^3/A^2)$ 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 α from Column 14 below should be taken as 1.

Column 14. α is the velocity distribution coefficient and is calculated as follows:

$$\alpha = \Sigma\left(\frac{K^3 A_t^{\,2}}{A^2 K_t^{\,2}}\right)$$

Column 15. V is the average velocity and is calculated as Q/A_t.

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

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

Column 18. h_o is the other losses, and is calculated by multiplying either the expansion or contraction coefficient, K_m, times the absolute value of that in Column 17.

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

30-5.05 Water and Sediment Routing

The BRI-STARS (Bridge Stream Tube Model for Sediment Routing Alluvial River Simulation) Model was developed by the National Cooperative Highway Research Program and FHWA. It is based on utilizing the stream-tube method of calculation which allows the lateral and longitudinal variation of hydraulic conditions and sediment activity at various cross sections along the study reach. Both energy and momentum functions are used in the BRI-STARS model so that the water-surface profile computation can be carried out through combinations of sub critical and supercritical flows without interruption. The stream-tube concept is used for hydraulic computations in a semi-two-dimensional way. Once the hydraulic parameters in each stream tube are computed, the scour or deposition in each stream tube, determined by sediment routing, will provide the variation of channel geometry in the vertical direction.

The BRI-STARS model includes a rule-based expert system program for classifying a stream by size, bed and bank-material stability, planform geometry, and other hydrologic and morphological features. Due to the complexities of a single classification system that utilizes all parameters, no universally-acceptable stream-classification method presently exists. Consequently, this model does not include a single methodology for classifying every stream. Instead, methodologies were first classified according to the channel-sediment sizes they were derived for then. Within each size group, one or more classification schemes have been included to cover a wider range of environments. The stream-classification information can be used to assist in the selection of model parameters and algorithms.

Applications of the BRI-STARS model can be summarized as follows.

1. Fixed-bed model to compute water-surface profile for subcritical, supercritical, or a combination of both flow conditions involving a hydraulic jump.

2. Movable-bed model to route water and sediment through alluvial channels.

3. Use of stream tubes to allow the model to compute the variation of hydraulic conditions and sediment activity in the longitudinal and the lateral directions.

4. The armoring option allows simulation of longer-term riverbed changes.

5. The minimization procedure option allows the model to simulate channel widening and narrowing processes.

6. The local bridge-scour option allows for the computation of pier or abutment scour.

7. The bridge routines for the fixed-geometry mode from WSPRO are available as an option in the program.

30-6.0 DESIGN PROCEDURE

30-6.01 General

The design procedure for each type of channel has some common elements and some substantial differences. This Section will outline a process for assessing a natural stream channel and a more-specific design procedure for a roadside channel.

30-6.02 Stream Channel

The analysis of a stream channel is in conjunction with the design of a highway hydraulic structure such as a culvert or bridge. The objective is to convey the water along or under the highway such that it will not cause damage to the highway, stream, or adjacent property. An assessment of the existing channel is necessary to determine the potential for problems that can 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. Additional information on stream morphology and factors that affect stream stability is provided in Section 30-7.0.

Although the following step-by-step procedure may not be appropriate for each application, it does outline a process which will usually apply.

1. Step 1: Assemble Site Data and Project File.

    a. Data Collection
        (1) Topographic, site, and location maps
        (2) Roadway profile
        (3) Photographs
        (4) Field reviews
        (5) Design data at nearby structures
        (6) Gaging records
        (7) Historic flood data and local knowledge
        (8) pH readings
    b. Studies by Other Agencies
        (1) Flood-insurance studies
        (2) Floodplain studies
        (3) Watershed studies
    c. Environmental Constraints
        (1) Floodplain encroachment
        (2) Floodway designation
        (3) Fish and wildlife habitat
        (4) Commitments in review documents
    d. Design Criteria. See Section 30-3.0.

2. Step 2: Determine the Project Scope.

   a. Determine Level of Assessment
      (1) Stability of existing channel
      (2) Potential for damage
      (3) Sensitivity of the stream
   b. Determine Type of Hydraulic Analysis
      (1) Qualitative Assessment
      (2) Single-Section Analysis
      (3) Step-Backwater Analysis
   c. Determine Additional Survey Information
      (1) Extent of streambed profiles
      (2) Locations of cross sections
      (3) Elevations of flood-prone property
      (4) Details of existing structures
      (5) Properties of bed and bank materials

3. Step 3: Evaluate Hydrologic Variables.

   a. Compute Discharges for Selected Frequencies
   b. See Chapter Twenty-nine

4. Step 4: Perform Hydraulic Analysis.

   a. Single-Section Analysis (Section 30-5.03)
      (1) Select representative cross section (Section 30-5.02)
      (2) Select appropriate *n* values from Figure 30-4B, Uniform Flow (Values of Manning's *n*)
      (3) Compute stage-discharge relationship
   b. Step-Backwater Analysis (Section 30-5.04)
   c. Calibrate with Known High Water

5. Step 5: Perform Stability Analysis.

   a. Geomorphic Factors
   b. Hydraulic Factors
   c. Stream Response to Change

6. Step 6: Design Countermeasures.

   a. Criteria for Selection

(1) Erosion mechanism
(2) Stream characteristics
(3) Construction and maintenance requirements
(4) Vandalism considerations
(5) Cost

b. Types of Countermeasures
(1) Meander migration countermeasures
(2) Bank stabilization (Chapter Thirty-eight)
(3) Bend control countermeasures
(4) Channel braiding countermeasures
(5) Degradation countermeasures
(6) Aggradation countermeasures

c. For Additional Information
(1) HEC 20 *Stream Stability*
(2) Highways in the River Environment
(3) See Reference List

7. Step 7: Documentation.

a. Prepare Report and File with Background Information
b. See Chapter Twenty-eight

30-6.03 Roadside Channel

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

The primary function of a roadside channel is to collect surface runoff from the highway pavement and areas which drain to the right of way, and convey the accumulated runoff to acceptable outlet points.

A secondary function is 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 a subsurface drainage system such as pipe underdrains.

The alignment, cross section, and grade of a roadside channel are constrained by the geometric and safety criteria applicable to the project. Such a channel should accommodate the design runoff in a manner which ensures the safety of the motorist and minimizes future maintenance, damage to adjacent properties, and adverse environmental or aesthetic effects.

The procedure described below will assist the designer in stable channel design and in determining the type of lining if necessary. Section 30-6.03(02) provides information on the computer program HYCHL.

30-6.03(01) Step-By-Step Procedure

Each project is unique, but the following basic design steps are applicable.

1. Step 1: Establish a Roadside Plan.

   a. Collect available site data.
   b. Obtain or prepare existing and proposed plan-and-profile layouts including highway, culverts, bridges, etc.
   c. Determine and plot on the plan the locations of natural basin divides and roadside channel outlets. An example of a roadside channel plan and profile is shown in Figure 30-6A, Sample Roadside Channel.
   d. Perform the layout of the proposed roadside channels to minimize diversion flow lengths.

2. 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 which may restrict cross section design
      (1) right-of-way limits
      (2) trees or environmentally-sensitive areas
      (3) utilities
      (4) existing drainage facilities

3. Step 3: Determine Initial Channel Grades.

   a. Plot initial grades on the plan-and-profile layout. Slopes in a roadside ditch in a cut are controlled by the highway grades.
   b. Provide a 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 which may influence or restrict grade, such as utility locations.

4. Step 4: Check Flow Capacities and Adjust As Necessary.

   a. Compute the design discharge at the downstream end of a channel segment (see Chapter Twenty-nine).
   b. Set preliminary values of channel size, roughness coefficient, and slope.
   c. Determine maximum allowable depth of channel including freeboard.
   d. Check flow capacity using Manning's equation and single-section analysis.
   e. If capacity is inadequate, possible adjustments are as follows:
      (1) increase bottom width;
      (2) make channel side slopes flatter;
      (3) make channel slope steeper;
      (4) provide smoother channel lining; or
      (5) 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 or to reduce peak discharge.

5. Step 5: Determine Channel Lining or Protection Needed (HEC 15).

   a. Select a lining and determine the permissible shear stress τ_p in lb/ft^2 from Figure 30-6B, Classification of Vegetal Covers as to Degrees of Retardancy, or Figure 30-6C, Summary of Permissible Shear Stress for Variable Protection Measures.

   b. Estimate the flow depth and choose an initial Manning's *n* value from Figure 30-6D, Manning's Roughness Coefficients and Roughness Element Height, $k_{S,}$ or from the figures as follows:

      30-6E Manning's *n* Versus Relative Roughness for Selected Lining Types (HEC 15)
      30-6F Manning's *n* Versus Hydraulic Radius, *R*, for Class A Vegetation (HEC 15)
      30-6G Manning's *n* Versus Hydraulic Radius, *R*, for Class B Vegetation (HEC 15)
      30-6H Manning's *n* Versus Hydraulic Radius, *R*, for Class C Vegetation (HEC 15)
      30-6 I Manning's *n* Versus Hydraulic Radius, *R*, for Class D Vegetation (HEC 15)
      30-6J Manning's *n* Versus Hydraulic Radius, *R*, for Class E Vegetation (HEC 15)

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 5.b. and 5.c.

d. Compute maximum shear stress at normal depth as follows:

$$\tau_d = 2990 y_o S, \text{ where } S = \text{channel slope, ft/ft}$$

e. If $\tau_d < \tau_p$, the lining is acceptable. Otherwise, consider the following:
(1) choose a more resistant lining;
(2) use concrete, gabions, or other more-rigid lining either as full lining or composite;
(3) decrease channel slope;
(4) decrease slope in combination with drop structures; or
(5) increase channel width or flatten side slopes.

6. Step 6: Analyze outlet points and downstream effects.

a. Identify adverse impacts such as increased flooding or erosion to downstream properties which, at the channel outlet, may result from one of the following:
(1) increase or decrease in discharge;
(2) increase in velocity of flow;
(3) concentration of sheet flow;
(4) change in outlet water quality; or
(5) diversion of flow from another watershed.

b. Mitigate adverse impacts identified in Step 6.a. The possibilities include the following:
(1) enlarge outlet channel or install control structures to provide detention of increased runoff in channel;
(2) install velocity control structures (energy dissipators);
(3) increase capacity or improve lining of downstream channel;
(4) install sedimentation or infiltration basins;
(5) install weirs or other outlet devices to redistribute concentrated channel flow; or
(6) eliminate diversions which result in downstream damage and which cannot be mitigated in a less-expensive manner.

30-6.03(02) Design Considerations

To obtain the optimum roadside-channel-system design, it may be necessary to conduct more than one trial of the previous procedure before a final design is achieved.

More details on channel lining design are provided in HEC 15 including consideration of channel bends, steep slopes, and composite linings. The riprap design procedures described in HEC 15 are for a channel having a design discharge of 50 ft^3/s or less. Where the design discharge exceeds 50 ft^3/s, the design procedure provided in Chapter Thirty-eight on bank protection should be followed.

Stable channel design can be accomplished with the assistance of the HYCHL computer program. HYCHL is a module of the integrated system HYDRAIN. The basis for the program's algorithms is provided in FHWA publications HEC-11 and HEC-15. Although both documents address the analysis of lining stability, each document addresses different classes of problems. HEC 15 focuses on linings in a roadside channel which are characterized by relatively uniform cross sections on a constant slope. Alternatively, HEC 11 addresses a natural channel with irregular cross sections, varying bottom slopes, and carrying a larger flow. HEC 11 focuses on the design of riprap lining. Together, HEC 15 and HEC 11 provide a series of analysis and design tools that are present in HYCHL.

The computational elements of the program are all based in English-measurement units because these are the common units for all of the reference materials. HYCHL performs the necessary input and output conversions for an English-units design.

HEC 15 outlines procedures for analyzing channel linings based on tractive-force theory. The procedure involves comparing an estimated shear stress resulting from flow in a channel to the maximum permissible shear stress determined for a given lining type. If the shear from flowing water increases to where it is greater than the permissible shear of the lining, failure may occur. This concept allows for calculation of the maximum discharge that a channel can convey, if the calculated shear is assumed to equal permissible shear.

The analysis of a rigid, vegetative, gabion, or temporary lining in HYCHL is applicable to a channel of uniform cross section and constant bottom slope. A roadside channel exhibits such characteristics. HYCHL offers design and analysis options including the following:

1. rigid or flexible linings;
2. temporary or permanent linings;
3. single or composite linings;
4. straight or bending channel section;
5. alternative regular channel shape; and
6. constant or variable channel flow.

HEC 15 and HEC 11 both outline procedures for analyzing riprap-lined channels. These procedures are based on the tractive-force theory but include additional considerations not necessary for analyzing rigid, vegetative, gabion, or temporary lining types. A channel lined with riprap can be analyzed for stability given the riprap size, or the riprap size can be determined based on a user-supplied stability factor. A composite channel which has riprap for the low-flow lining and another lining type for the high-flow channel can be analyzed. HYCHL can also analyze an irregular channel shape which is lined with riprap only.

30-7.0 STREAM MORPHOLOGY

30-7.01 Introduction

The form assumed by a natural stream, which includes its cross-sectional shape and its plan-form, is a function of many variables for which cause-and-effect relationships are difficult to establish. The stream may be graded or in equilibrium with respect to long time periods, which means that on average it discharges the same amount of sediment that it receives although there may be short-term adjustments in its bed forms in response to flood flows. 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. 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 a proposed highway facility is possible through a thorough knowledge of river mechanics and the 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 or degradation problem. The BRI-STARS model (see Section 30-5.0) is recommended as a tool to quantify the expected scour or sedimentation of potential problem locations. References (FHWA, 1990, and Molinas, 1994) should be consulted to evaluate the problem and propose mitigation measures. The proposed methodology is subject to approval by the Hydraulics Team leader.

The natural stream channel will assume a geomorphological form which will be compatible with the sediment load and discharge history which it has experienced. 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.

30-7.02 Levels of Assessment

The analysis and design of a stream channel will require an assessment of the existing channel and the potential for problems as a result of the proposed action. The necessary detail of studies should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream. Observation 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. These analytical tools should only be used to substantiate the erosion potential indicated through observation. Descriptions of the three levels of assessment are as follows.

1. 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, or interviews with long-time residents.

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

3. Level 3. Complex quantitative analysis based on detailed mathematical modeling and possibly physical hydraulic modeling. This is necessary only for a high-risk location, an extraordinarily complex problem, or an 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 (see Section 30-5.0) or physical modeling. Required data will include Level 1 and 2 data, 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, or angle of repose.

30-7.03 Factors That Affect Stream Stability

Factors that affect stream stability and, potentially, bridge and highway stability at a stream crossing can be classified as geomorphic factors and hydraulic factors.

1. Geomorphic Factors.

    a. Stream size
    b. Flow variability

c. Valley setting
d. Floodplains
e. Natural levees
f. Apparent incision
g. Sinuosity
h. Channel boundaries
i. Width variability
j. Degree of braiding
k. Bar development
l. Degree of anabranching

Figure 30-7A, Geomorphic Factors That Affect Stream Stability, depicts examples of the geomorphic factors.

2. Hydraulic Factors.

a. Magnitude, frequency, and duration of flood
b. Bed configuration
c. Resistance to flow
d. Water-surface profiles

Figure 30-7B, Channel Classification and Relative Stability as Hydraulic Factors Are Varied, depicts the changes in channel classification and relative stability as related to the hydraulic factors.

Rapid or unexpected changes may occur in a stream in response to man's activities in the watershed such as alteration of vegetative cover. 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, 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.

A natural disturbance such as a flood, drought, earthquake, landslide, volcano, or forest fire can also cause large changes in sediment load and thus major changes in the stream channel. Although it is difficult to plan for such a disturbance, if one does occur it is likely that changes will also occur to the stream channel.

30-7.04 Stream Response To Change

The complicating factors in river mechanics are the large number of interrelated variables that can simultaneously respond to natural or imposed changes in a stream system; and the continual evolution of stream channel patterns, channel geometry, bars, and forms of bed roughness with changing water and sediment discharge. To better understand the responses of a stream to the actions of man and nature, hydraulic and geomorphic concepts are described below.

The dependence of stream form on slope, which may be imposed independently of other stream characteristics, is illustrated schematically in Figure 30-7C, Sinuosity Versus Slope with Constant Discharge.

A natural or artificial change which 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. In Figure 30-7C, 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.

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 wave length, 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.

Figure 30-7D, Slope-Discharge for Braiding- or Meandering-Bed Stream, illustrates the dependence of river form on channel slope and discharge, showing that if $SQ^{0.25} \leq 0.00070$ in a sand bed channel, the stream will meander. Similarly, if $SQ^{0.25} \geq 0.0041$, the stream is braided.

In these equations, S is the channel slope in feet per foot and Q is the mean discharge in cubic feet per second. The transitional range of $SQ^{0.25}$ is between these values.

Many rivers plot in the zone between the limiting curves defining a meandering or braided stream. 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.

30-7.05 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. It may be installed at the time of highway construction or retrofitted to resolve stability problems at an existing crossing.

Retrofitting is good economics and good engineering practice because the magnitude, location, and nature of potential stability problems are not always discernible at the design stage and, 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.

Below is a discussion of possible countermeasures for some common river-stability problems. Consult the references listed at the end of this Chapter for detailed information on the design and construction of the countermeasures.

30-7.05(01) Meander Migration

The best countermeasure against meander migration is a crossing location on a relatively straight reach of stream between bends. Other countermeasures include the protection of an existing bank line, the establishment of a new flow line or alignment, or the control and constriction of channel flow. Countermeasures identified for bank stabilization and bend control include bank revetment, spur, retardance structure, longitudinal dike, vane dike, bulkhead, or channel relocation. Measures may be used individually or in combination to combat meander migration at a site (FHWA, 1990; HEC-20, 1991).

30-7.05(02) Channel Braiding

Countermeasures used at a braided stream are 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 consist of dikes constructed from the limits of the multiple channels to the channel over which the bridge is constructed. A spur dike at a bridge end used in combination with revetment on a highway fill slope, riprap on highway fill slope only, or a spurs arranged in the stream channels to constrict flow to one channel have also been used successfully.

30-7.05(03) Degradation

Degradation can cause the loss of a bridge pier in a stream channel, or a pier or abutment in a caving bank. 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 a small to medium stream.

A longitudinal stone dike placed at the toe of a channel bank can be an effective countermeasure for bank caving in a degrading stream. Precautions to prevent outflanking, such as tiebacks to the banks, may be necessary where installation is limited to the vicinity of the highway stream crossing. Channel lining alone is not a successful countermeasure against degradation problems (HEC-20).

30-7.05(04) Aggradation

Current measures in use to alleviate aggradation problems at a highway-stream crossing include channelization, bridge modification, continued maintenance, or combinations of these.

Channelization may include excavating and cleaning a channel, constructing cutoffs to increase the local slope, constructing flow control structures to reduce and control the local channel width, or 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.

Another technique is the submerged vane technique developed by the University of Iowa. The studies suggest that the submerged vane structure may be an effective, economic, low-maintenance, and environmentally-acceptable sediment-control structure with a wide range of applications (HEC-20, Odgaard and others, 1986).

30-8.0 EXAMPLE PROBLEMS

30-8.01 Example 30-8.1

Given: Q_{25} = 175 ft^3/s and Q_{100} = 220 ft^3/s. Cross-section information is shown in the following table of surveyed data points for a typical cross section.

Table of Cross-Section Data (Section A, Sta. 1 + 36)

Distance	Elevation	*n* Value
0.00	745.23	0.06
8.13	742.90	0.06
40.63	742.50	0.035
45.73	738.63	0.035
50.80	738.63	0.035

53.83	742.30	0.05
77.98	741.07	0.05
79.23	742.70	0.05
109.73	745.73	0.05

Subsection 1 consists of an overbank area with light brush and trees. Subsection 2 is in the main channel of this stream and comprises a clean, straight stream with a few weeds and rocks. Subsection 3 is in the right-hand floodplain and includes some scattered brush with considerable weeds.

See Figure 30-8.1 for site data.

Find: Use the single-section method to develop a stage-discharge curve for the channel cross-section at Station 1 + 36 which is located downstream from a highway culvert. Determine the tailwater elevation at the outlet of the culvert (assume a channel station of 0 + 30 for this location) for the 25-year and 100-year floods.

See Figure 34-8.2 for stream cross section "A."

Solution: The slope of the stream can be determined by examining the reach from stream Station -0 + 91 to the typical section at Station 1 + 36. The flow line differential for this reach is 2 ft (in 757 ft of stream reach). Therefore, the slope, *S*, is 0.0027 ft/ft.

Figure 30-8A can be used to assist in the development of a stage-discharge curve for this typical section. Assuming water-surface elevations beginning at 739.63, calculate pairs of water surface elevation/discharge for plotting on a stage-discharge curve. Illustrative calculations in which arbitrary increments of water-surface elevation of 1 ft were used are shown in Figure 30-8A, Channel Computation Form. A plotted stage-discharge curve is shown in Figure 30-8B. The water elevation for Q_{25} is 175 ft^3/s and for Q_{100} is 220 ft^3/s.

Because the calculation section for the stream is downstream of the culvert site, it will be necessary to project the water-surface elevation as determined from the typical section at stream Station 1 + 36 to represent the tailwater elevation at stream Station 0 + 30. Therefore, the projected tailwater levels are calculated as follows:

TW_{25} = 742.60 + (453 – 100) (0.0027) = 743.55 ft

TW_{100} = 742.80 + 0.953 = 743.75 ft

30-8.02 Example 30-8.2

Given: A series of five cross sections are available for a creek flowing at a discharge of 10,000 ft^3/s. The ground data points, *GR*, roughness, *n*, and subsection data, *SA*, are included in the WSPRO data input shown in Figure 30-8C, WSPRO Input Data File, for cross sections A, B, C, D and E.

Find: For tailwater elevations of 607.57 ft, 608.60 ft, and 609.60 ft at cross section A, compute the water surface profiles using the FHWA/USGS program WSPRO.

Solution: The computer results for tailwater, *TW*, values of 607.75 ft, 608.60 ft, and 609.60 ft are shown in Figure 30-8D, WSPRO Results. The message no. 135 is ignored because the ratio of conveyances of cross sections A and B is only slightly outside the recommended limit of 1.4. If this ratio becomes too large (>1.4) or too small (< 0.6), additional cross sections may be required. The water-surface profiles are shown in Figure 30-8E, Water Surface Profiles for Q – 10,000 ft^3/s and Variable Tailwater Elevations. The critical water-surface elevations, CRWS, at each cross section are shown to emphasize that these profiles are all sub critical. The profiles tend to converge in the upstream direction, but the distance required for convergence at the desired cross section should be determined as discussed in Section 30-5.04(03).

<u>Example 30-8.3</u>

Given: A roadside drainage channel is trapezoidal with a bottom width of 4 ft and 3H:1V side slopes. The bed slope is 0.005 ft/ft and the design flow rate is 21 ft^3/s.

Find: Calculate the required diameter, D_{50}, of a gravel riprap that is to be used as a permanent channel lining and the design depth of flow.

Solution: The solution follows the procedure outlined in HEC 15 which is based on the tractive-force method.

1. Choose a rounded gravel with D_{50} = 1 in.

   Then τ_p = 0.40 lb/ft^2 (Figure 30-6C, Summary of Permissible Shear Stress for Variable Protection Measures)

2. Estimate n = 0.033 from Figure 30-6D, Manning's Roughness Coefficients, and Roughness Element Height, k_S, for d = 0.5 – 2 ft

3. Calculate y from Manning's equation (Figure 30-5B, Trapezoidal Channel Capacity Chart), as follows:

$$\frac{Qn}{b^{2.67}s^{0.5}} = \frac{(0.033)(21)}{(4)^{2.67}(0.005)^{0.5}} = 0.242$$

Then from Figure 30-5B with $Z = 3$: $y/b = 0.28$ and $y = (4.0)(0.28) = 1.12$ ft

4. Check n value as follows:

$$R = \frac{y(b+3y)}{b+(10)^{0.5}(2y)} = 0.74 \text{ ft, and } \frac{R}{k_s} = \frac{R}{D_{50}} = 8.9$$

From Figure 30-6E, Manning's n Versus Relative Roughness for Selected Lining Types (HEC 15), $n = 0.034 \approx 0.033$, therefore, acceptable.

5. Calculate maximum bed shear stress, τ_d:

$\tau_d = 9800Sy = (62.4)(1.12)(0.005) = 0.35 \text{ lb/ft}^2$

Since $\tau_d < \tau_p$, accept D_{50} of approximately 1 in. Otherwise, repeat with another riprap diameter.

6. Sideslopes will be stable because they are not steeper than 3H:1V. If the sideslopes are steeper than 3H:1V or if the channel slope is steep, consult HEC 15 for additional computations.

<u>Example 30-8.4</u>

(From HEC 15)

Given: A median ditch is lined with a stand of Kentucky bluegrass of approximately 0.677 ft height. The ditch is trapezoidal with a bottom width of 4 ft and side slopes of 4H:1V. The ditch slope is 0.01 ft/ft.

Find: Compute the maximum discharge for which the lining will be stable, and the corresponding flow depth.

Solution: From Figure 30-6B, Classification of Vegetal Covers as to Degree of Retardancy, Kentucky bluegrass has a retardance class of C and, from Figure 30-6C, Summary of Permissible Shear Stress for Variable Protection Measures, the permissible shear stress is $\tau_p = 1 \text{ lb/ft}^2$

The allowable depth can be determined by assuming $\tau_p = \tau_d$, as follows:

$$y = \tau_p/(62.4\ S) = 1/(62.4)\ (0.01) = 1.60 \text{ ft}$$

Determine the flow area A and hydraulic radius R, as follows:

$$A = y(b + yz) = 1.60[4 + (4)(1.60)] = 16.6 \text{ ft}^2$$

$$P = b + 2y(1 + z^2)^{1/2} = 4 + 2(1.60)(1 + 16)^{1/2} = 17.19 \text{ ft}$$

$$R = \frac{A}{P} = \frac{16.6}{17.19} = 0.97\ ft$$

Finally, determine the Manning's n value from Figure 30-6H, Manning's n Versus Hydraulic Radius, R, for Class C Vegetation (HEC 15), and solve for Q from Manning's equation.

From Figure 30-6H, n = 0.072, and

$$Q = \frac{1.486 A R^{0.67} S^{0.5}}{n}$$

$$Q = \frac{1.486(16.6)(0.97)^{2.67}(0.01)^{0.5}}{0.072} = 33.6 \text{ ft}^3\text{/s}$$

This method is called the maximum discharge method and is useful for determining the stable channel capacity for a variety of different linings for purposes of comparison.

<u>Example 30-8.5</u>

Given: A rectangular channel on a slope S = 0.001 with a width of 6 ft expanding to a width b = 10 ft in a straight walled transition, and Z = 0. The design discharge is 300 ft^3/s and Manning's n = 0.02.

Find: Calculate the depth of flow in the upstream 6 ft wide channel if normal depth is the downstream control.

Solution: 1. Compute the downstream normal depth, y_2, as follows:

$$\frac{Qn}{S^{0.5}b^{2.67}} = \frac{(300)(0.02)}{(0.001)^{0.5}(10)2.67} = 0.41$$

From Figure 30-5B, Trapezoidal Channel Capacity Chart, with $z = 0$, $y_2 = 6.50$ ft. Therefore $y_2/b = 0.65$. For a rectangular channel,

$$y_c = \left[\frac{1}{g}\left(\frac{Q}{b}\right)^2\right]^{0.33} = 3.0\ ft \text{, therefore subcritical.}$$

2. The downstream specific energy is determined as follows:

$$E_2 = y_2 + \frac{Q^2}{2gA_2{}^2} = 6.5 + \frac{(300)^2}{2(32.2)[(10)(6.5)]^2} = 6.83\ ft$$

3. Choose a straight-walled transition with a divergence angle of 12.5 deg with an expansion-loss coefficient of 0.5 (HEC 14). The length of the transition is determined as follows:

$$L = \frac{\left[\frac{(10-6)}{2}\right]}{\tan 12.5} = 8.9\ ft$$

4. Check if sub critical flow is possible by assuming critical depth in the upstream channel as follows:

$$E_{1c} - y_{1c} + \frac{V_{1c}{}^2}{2g},\ and\ E_1 = E_2 + z_2 - z_1 + h_L$$

where: $z_2 - z_1 = 0.001(2.71) = 0.009$, say 0

$$y_{1c} = \left[\frac{\left(\frac{300}{6}\right)^2}{32.2}\right]^{1/3} = 4.27\ ft$$

$$V_{1c} = \frac{Q}{A_{1c}} = \frac{300}{(6)(4.27)} = 11.7\ ft/s$$

$$V_2 = \frac{Q}{A_2} = \frac{300}{(6.5)(10)} = 4.6\ ft/s$$

$$h_L = 0.5(11.7^2 - 4.6^2)/(2)(32.2) = 0.90$$

$then,\ E1c = 4.27 + 11.7^2/(2)(32.2) = 6.40\ \ ft,$
and, $E_1 = 6.83 + 0 + 0.90 = 7.73$ ft.

Since $E_1 > E_{1c}$, a sub critical solution exists. If this were not true, the width of 6.0 ft would have to be increased.

5. Solve the energy equation, Equation 30-4.10, by trial as follows:

$$z_1 + y_1 + \frac{Q^2}{2gA_1^{\ 2}} = z_2 + E_2 + h_L$$

$$y_1 + \frac{(1-0.5)(300)^2}{2(32.2)(6.0)^2\left(y_1^{\ 2}\right)} = 6.83 - \frac{0.5(300)^2}{2(32.2)[(10)(6.5)]^2}$$

where: $z_1 - z_2 = 0$

$$h_L = 0.5\left(\frac{Q^2}{\left(2gA_1^{\ 2}\right)} - \frac{Q^2}{\left(2gA_2^{\ 2}\right)}\right)$$

$E_2 = 6.66$ ft
$A_2 = (10)(6.5) = 65$ ft

with the result $y_1 = 6.13$ ft, and $V_1 = 8.2$ ft/s.

6. Calculate the water surface profile using the Standard Step Method if boundary-resistance losses are of concern.

<u>Example 30-8.6</u>

Given: A rectangular transition contracts from a width of 10 ft to a width of 5 ft. The approach flow rate is 300 ft^3/s with a depth of 1 ft.

Find: Calculate the depth in the contracted section and the angle and length of the contraction so that the transmission of downstream standing waves is minimized.

Solution: 1. Calculate the approach Froude number for a rectangular channel as follows:

$$F = \frac{V}{(gd)^{0.5}} = \frac{\left(\frac{300}{(10)(1)}\right)^{1/2}}{[(32.2)(1)]} = 5.3 \text{, therefore supercritical.}$$

2. Determine the contraction ratio as follows:

$$\tau = \frac{b_3}{b_1} = \frac{5.0}{10.0} = 0.5$$

3. Use Figures 30-8F and 30-8G to determine the following:

θ = 5 deg and $y_3/y_1 = 2.1$, or $y_3 = 2.1$ ft

$$\frac{F_3}{F_1} = \tau^{-1}\left(\frac{y_3}{y_1}\right)^{-3/2} = 0.66,\ or\ F_3 = 3.6$$

$$L = \frac{\left(\frac{(b_3 - b_1)}{2}\right)}{\tan 5} = 28.6\ ft$$

4. This design satisfies the criterion $F_3 > 2$ and also is just to the left of curve A which means choking is not possible.

For the complete equations, see HEC 14 and Sturm, 1985.

30-9.0 REFERENCES

The references shown are recommended additions to the designer's library of references.

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SYMBOL	DEFINITION	UNIT
A	Cross-sectional area of flow	ft^2
d	Hydraulic depth	ft
E	Energy of a system	ft
Fr	Froude number	--
n	Manning's roughness coefficient	--
P	Wetted perimeter	ft
q	Discharge per unit width of channel	$ft^3/s/ft$
Q	Total discharge	ft^3/s
R	Hydraulic radius	ft
S	Slope of the energy gradeline	ft/ft
T	Channel top width at the water surface	ft
V	Mean velocity	ft/s
V_c	Critical velocity	ft/s
y_c	Critical depth	ft
α	Velocity distribution coefficient	--
h	Elevation of channel bottom, with respect to a datum, plus y, depth of water	ft
h_L	Head loss in the reach under study	ft
y	Depth of water	ft
Z	Elevation above a datum	ft

SYMBOLS FOR OPEN-CHANNEL HYDRAULICS

Figure 30-1A

Material	Manning's *n*	Maximum Allowable Velocity (ft/s)
Fine Sand	0.20	2.5
Sandy Loam	0.20	2.5
Silty Loam	0.20	3.0
Clay Loam	0.20	3.6
Clay	0.20	5.0
Silty Clay	0.20	5.0
Shale	0.20	6.0
Fine Gravel	0.20	5.0
Coarse Gravel	0.25	6.0

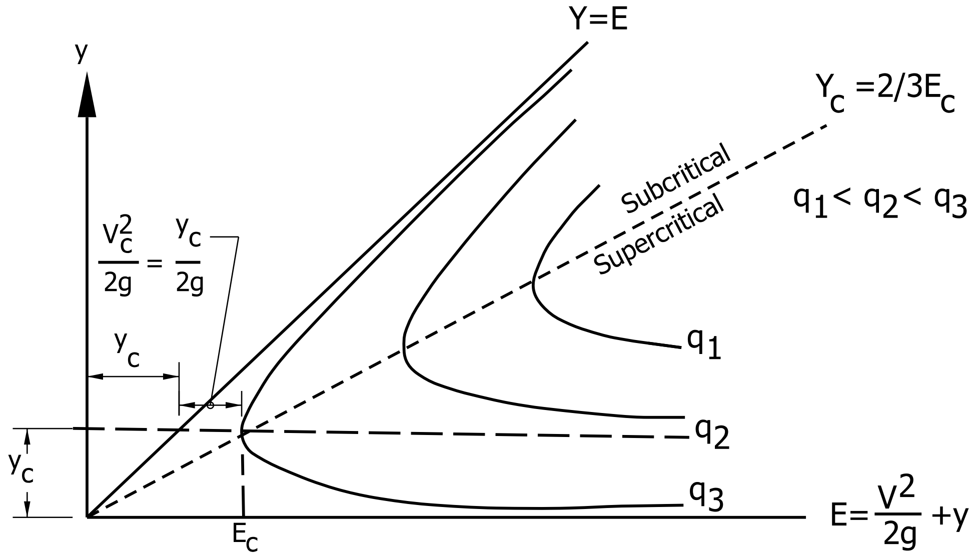
MAXIMUM VELOCITY IN A DRAINAGE DITCH

Figure 30-3A

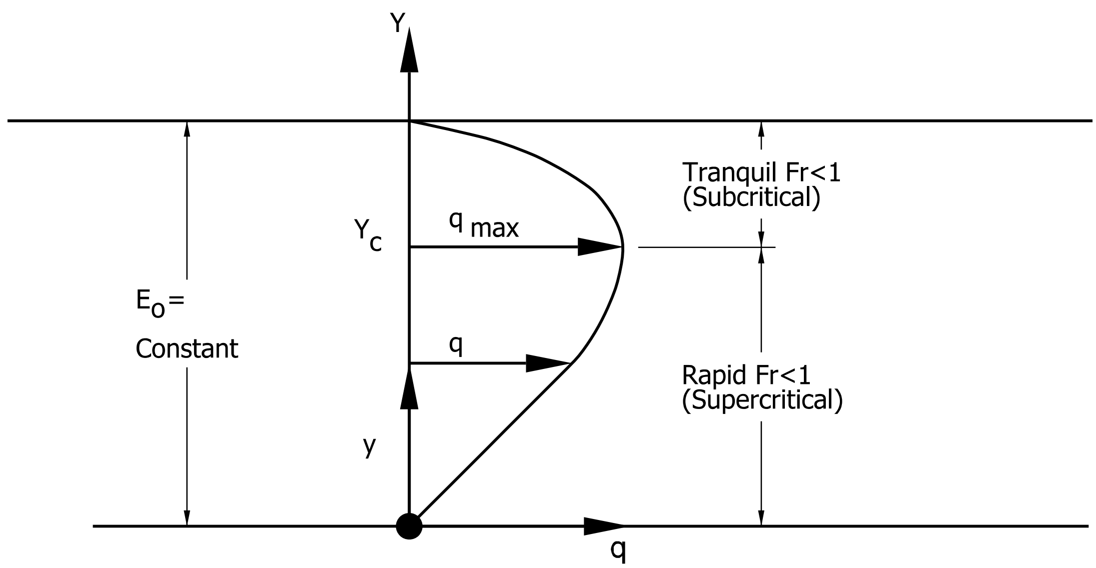
Grade, G	Interval
$3\% \leq G < 5\%$	200 ft
$5\% \leq G < 8\%$	150 ft
$8\% \leq G < 10\%$	100 ft
$\geq 10\%$	50 ft

LUG INTERVAL

Figure 30-3B



(a) Specific Energy Diagram



(b) Discharge Diagram

Note: See Figure 30-1A for a definition of terms.

SPECIFIC ENERGY AND DISCHARGE DIAGRAM
FOR RECTANGULAR CHANNELS

Figure 30-4A

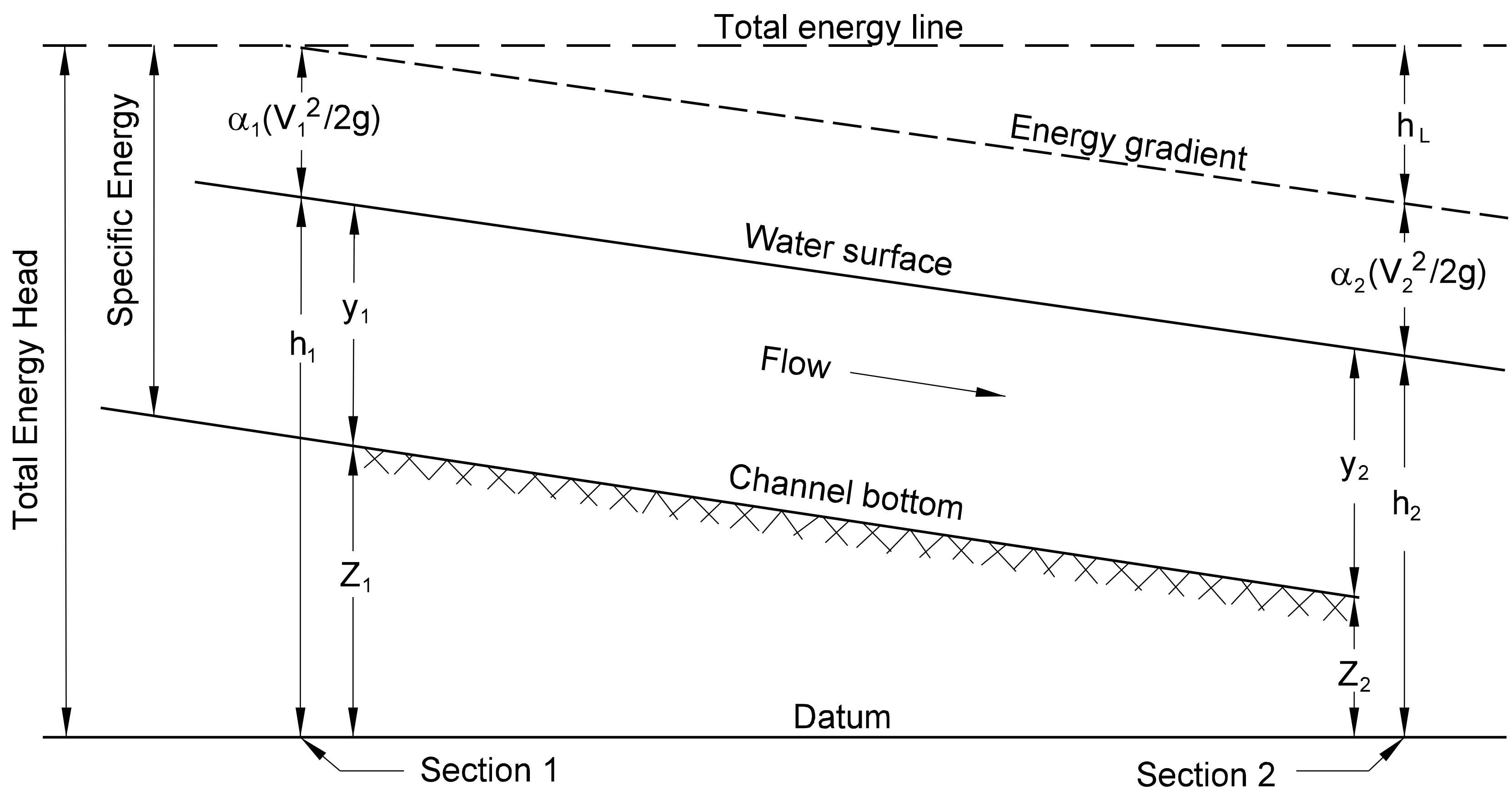
Type of Channel and Description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
1. Earth, Straight and Uniform	0.016	0.018	0.020
a. Clean, recently completed	0.018	0.022	0.025
b. Clean, after weathering	0.022	0.025	0.030
c. Gravel, uniform section, clean	0.022	0.027	0.033
2. Earth, Winding and Sluggish			
a. No vegetation	0.023	0.025	0.030
b. Grass, some weeds	0.025	0.030	0.033
c. Dense weeds or aquatic plants in deep channel	0.030	0.035	0.040
d. Earth bottom and rubble sides	0.025	0.030	0.035
e. Stony bottom and weedy sides	0.025	0.035	0.045
f. Cobble bottom and clean sides	0.030	0.040	0.050
3. Dragline, Excavated or Dredged			
a. No vegetation	0.025	0.028	0.033
b. Light brush on banks	0.035	0.050	0.060
4. Rock Cut			
a. Smooth and uniform	0.025	0.035	0.040
b. Jagged and irregular	0.035	0.040	0.050
5. Channel Not Maintained, Weeds and Brush Uncut			
a. Dense weeds, high as flow depth	0.050	0.080	0.120
b. Clean bottom, brush on sides	0.040	0.050	0.080
c. Clean bottom, highest stage of flow	0.045	0.070	0.110
d. Dense brush, high stage	0.080	0.100	0.140
NATURAL STREAM			
1. Minor Stream (top width at flood stage < 100 ft)			
a. Stream 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 or weeds	0.030	0.035	0.040
(3) Clean, winding, some pools or shoals	0.033	0.040	0.045
(4) Same as above, but some weeds or 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 floodway with heavy stand of timber and underbrush	0.075	0.100	0.150

NATURAL STREAM (contd.)			
Type of Channel and Description	**Minimum**	**Normal**	**Maximum**
1. Minor Stream (contd.)			
b. Mountain stream, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
(1) Bottom: gravel, cobbles, and few boulders	0.030	0.040	0.050
(2) Bottom: cobbles with large boulders	0.040	0.050	0.07
2. Floodplain			
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
d. Trees			
(1) Dense willows, in summer, straight	0.110	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 sprouts	0.050	0.060	0.080
(4) Heavy stand of timber, a few downed trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
(5) Same as above, but with flood stage reaching branches	0.100	0.120	0.160
3. Major Stream (top width at flood stage > 100 ft). The *n* value is less than that for a minor stream of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	n/a	0.060
b. Irregular and rough section	0.035	n/a	0.100

Source: Chow, V.T.

VALUES OF MANNING'S *n* FOR UNIFORM FLOW

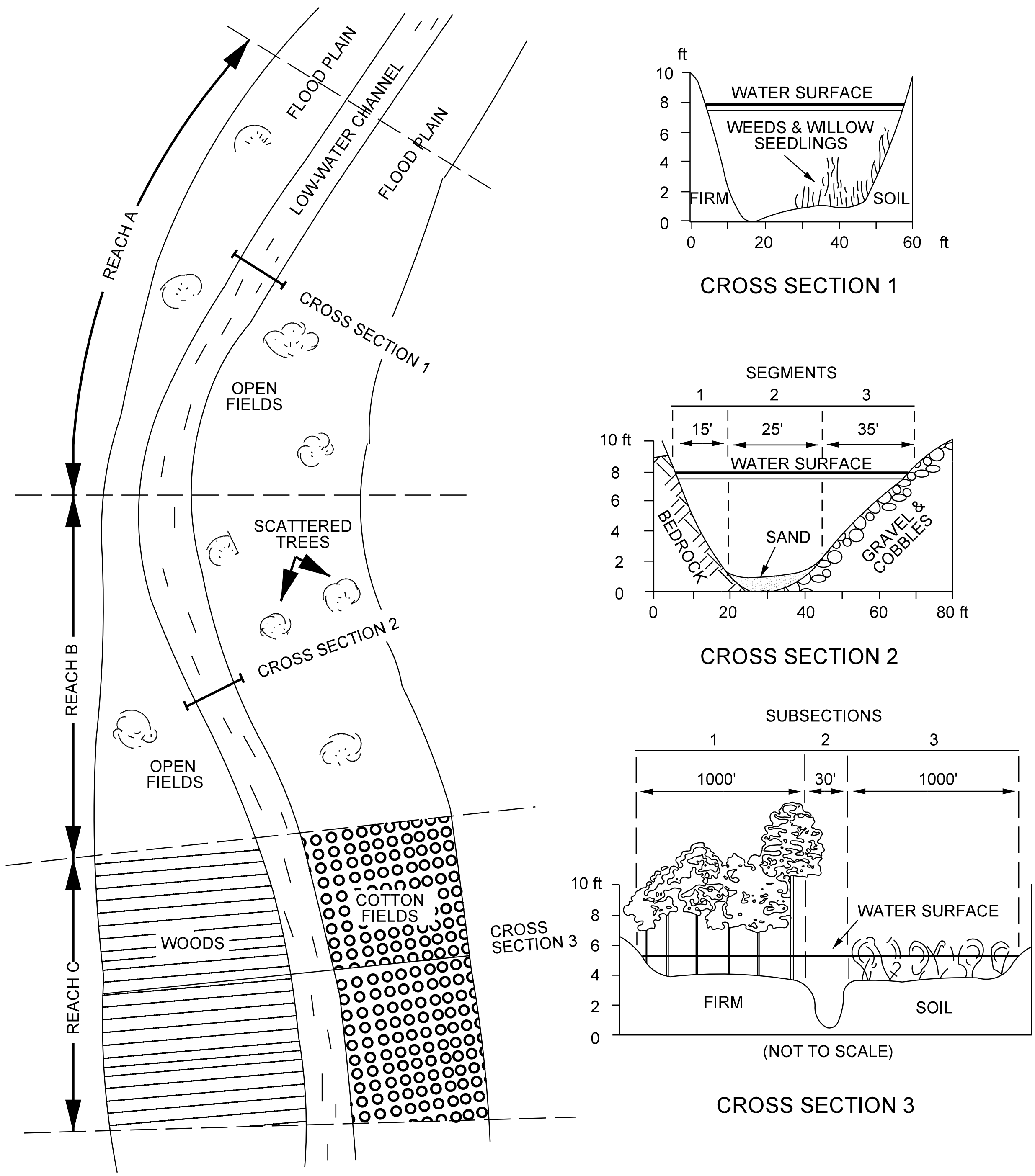
Figure 30-4B



Note: See Figure 30-1A for a definition of terms.

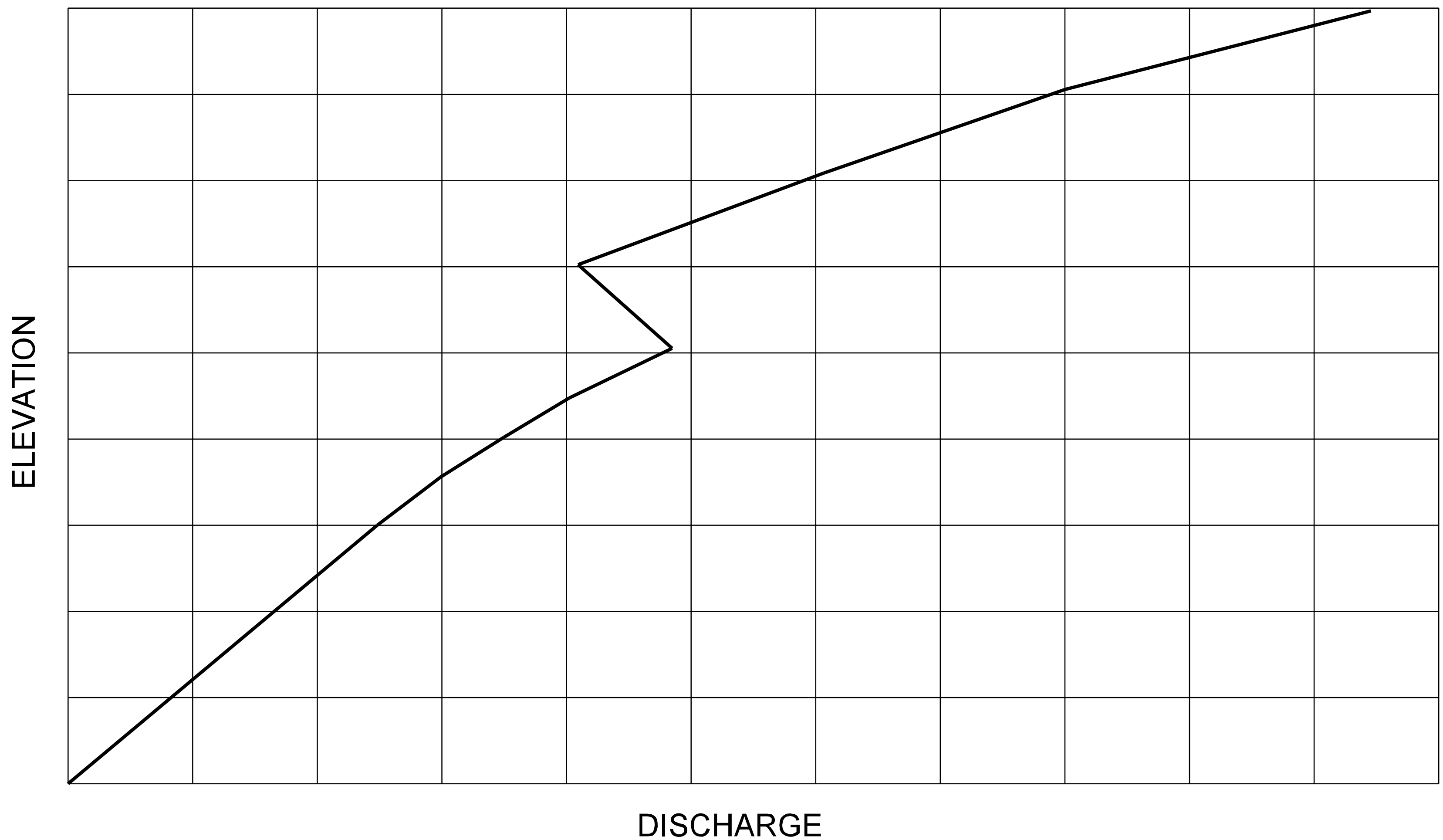
TERMS IN THE ENERGY EQUATION

Figure 30-4C



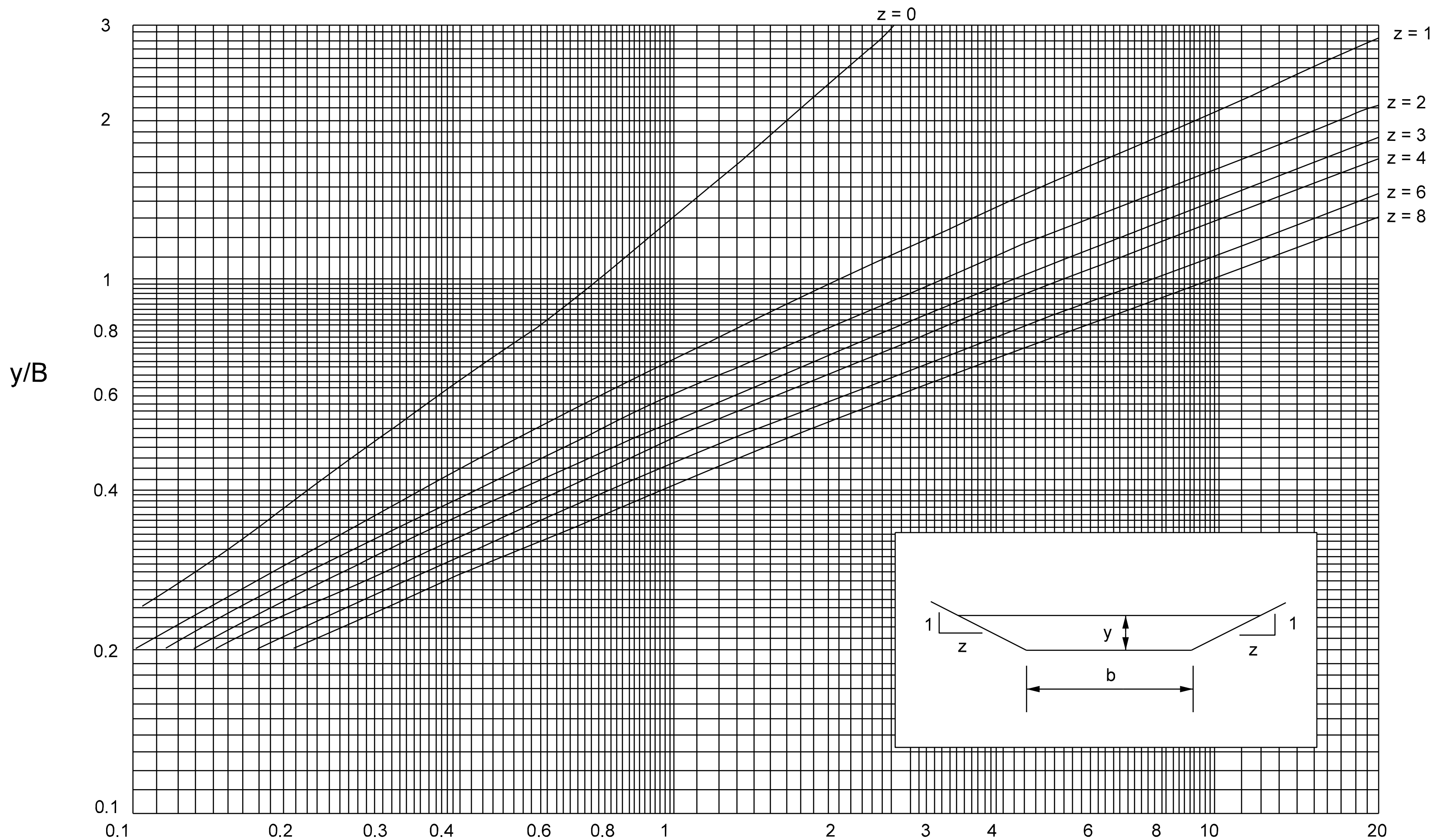
HYPOTHETICAL CROSS SECTION SHOWING REACHES, SEGMENTS AND SUBSECTIONS USED IN ASSIGNING n VALUES

Figure 30-5A



SWITCHBACK

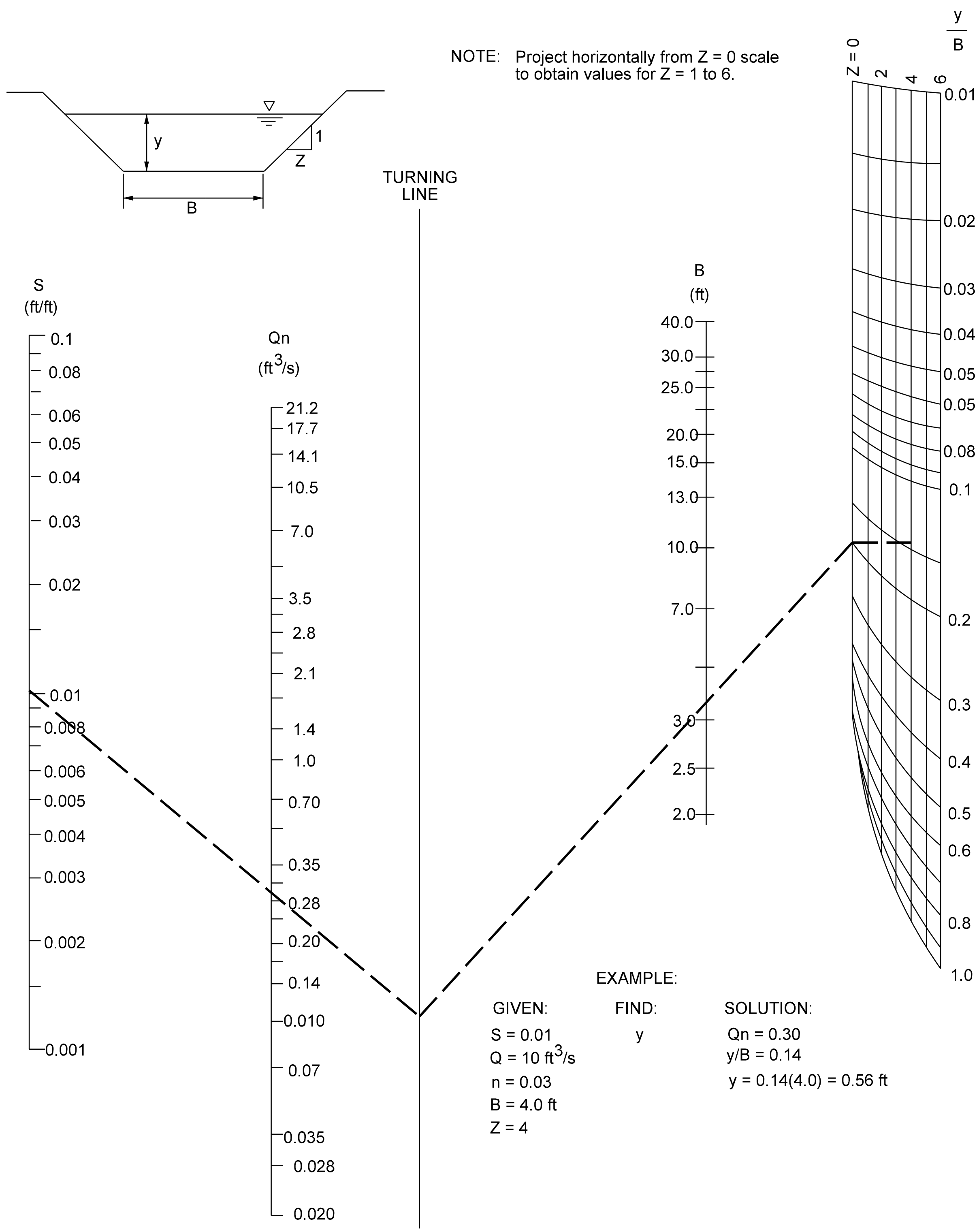
Figure 30-5A.1



$$\frac{Qn}{b^{8/3}S^{1/2}}$$

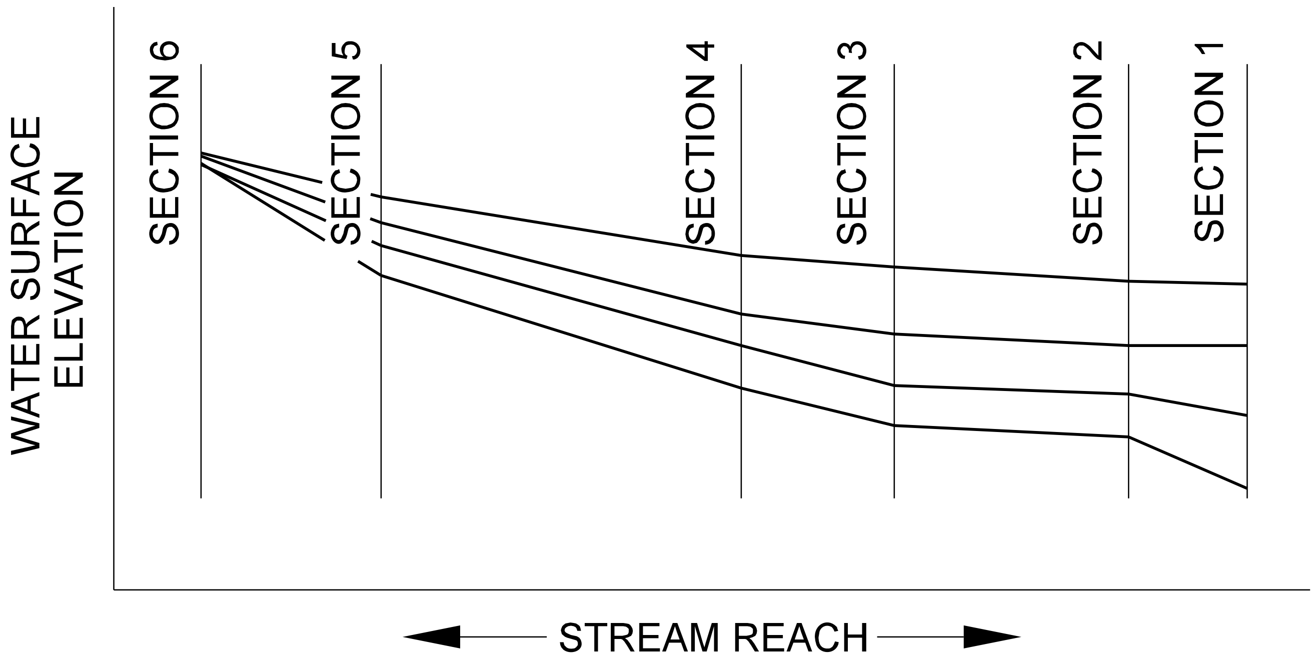
TRAPEZOIDAL CHANNEL CAPACITY CHART

Figure 30-5B



NOMOGRAPH FOR NORMAL DEPTH

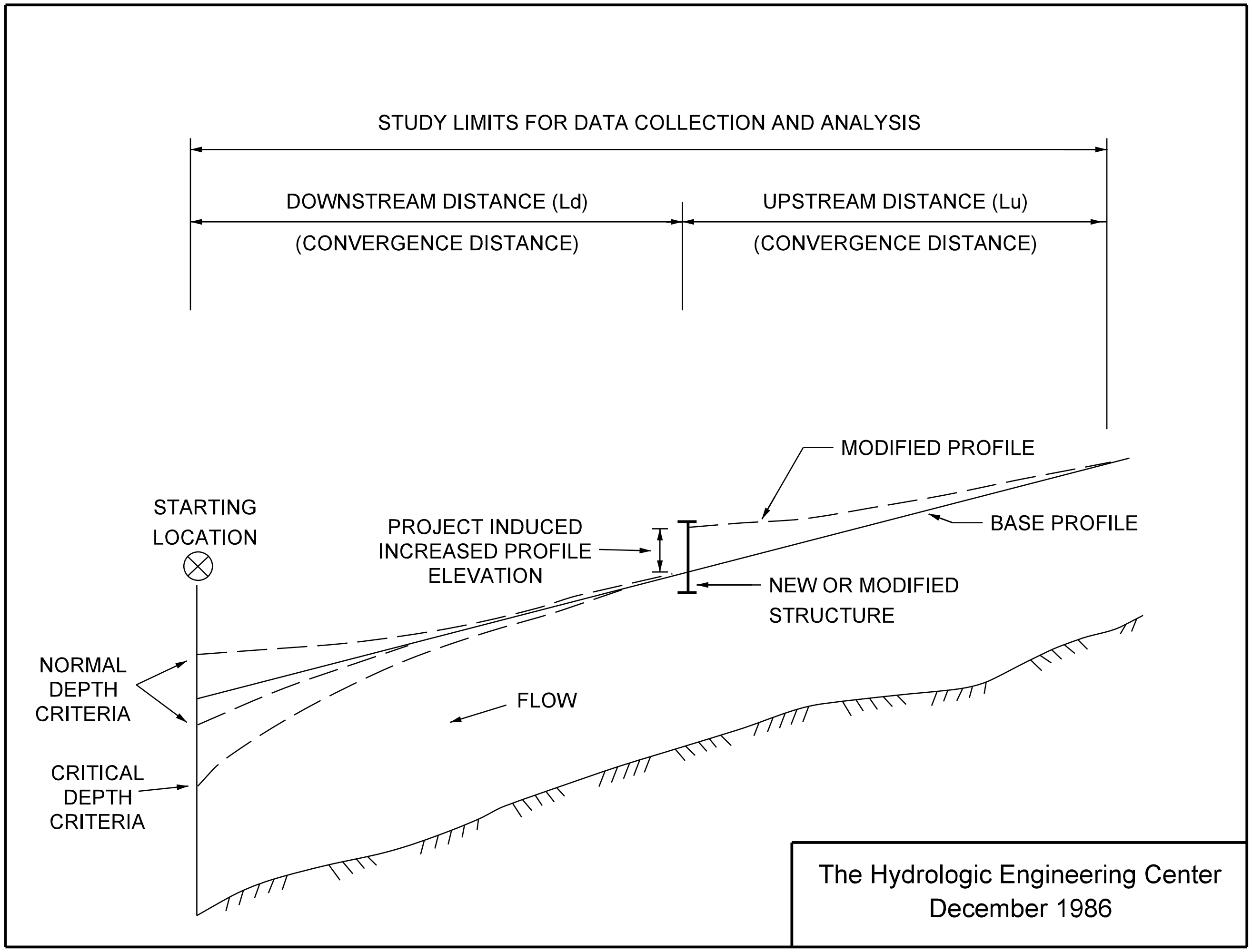
Figure 30-5C



PROFILE CONVERGENCE PATTERN
BACKWATER COMPUTATIONS

PROFILE CONVERGENCE PATTERN BACKWATER COMPUTATION

Figure 30-5D

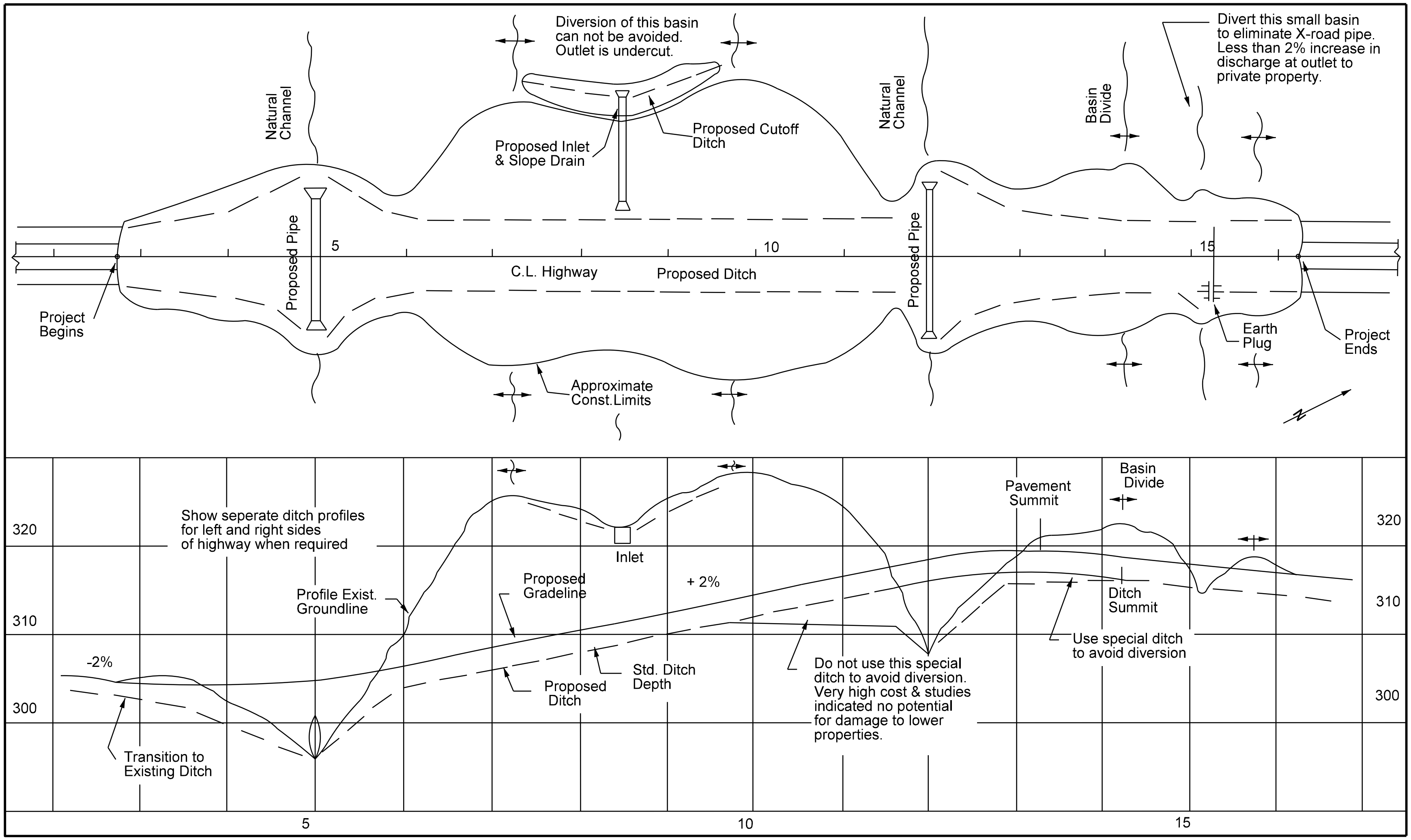


PROFILE STUDY LILMITS

Figure 30-5E

Cross Sec. No.	Water Surface Elevation		Area	Hydr. Rad. R	$R^{2/3}$	N	K	K_r	$\frac{1000}{S_r}$	L	h_f	$\frac{K^3}{A^2}$	ά	V	$\frac{\alpha V^2}{2g}$	$\Delta\left(\frac{\alpha V^2}{2g}\right)$	h_0	Δ Wtr. Surface Elev.
	Asmd.	Comp.																
col.1	col. 2	col. 3	col.4	col.5	cl.6	cl.7	cl.8	cl.9	col.10	cl.11	cl.12	cl.13	cl.14	cl.15	cl. 16	col. 17	cl.18	col. 19

WATER SURFACE PROFILE



SAMPLE ROADSIDE CHANNEL

Figure 30-6A

RETARD-ANCE	COVER	CONDITION
A	Ischaemum	Excellent stand, tall, average 30 in.
	Weeping lovegrass	Excellent stand, tall, average 36 in.
	Yellow bluestem	Excellent stand, tall, average 36 in.
B	Alfalfa	Good stand, unmowed, average 19 in.
	Bermuda grass	Good stand, tall, average 12 in.
	Blue gamma	Good stand, unmowed, average 13 in.
	Kudzu	Very close growth, uncut
	Lespedeza sericea	Good stand, not woody, tall, average 24 in.
	Native grass mixture: blue gamma, bluestem, little bluestem, other short- and long-stem Midwest grasses	Good stand, unmowed
	Weeping lovegrass	Good stand, unmowed, average 13 in.
C	Bermuda grass	Good stand, mowed, average 6 in.
	Centipedegrass	Very dense cover, average 6 in.
	Common lespedeza	Good stand, unmowed, average 11 in.
	Crabgrass	Fair stand, unmowed, 10 to 48 in.
	Grass-legume mixture, summer: common lespedeza, Italian ryegrass, orchard grass, redtop	Good stand, unmowed, 6 to 8 in.
	Kentucky bluegrass	Good stand, headed, 6 to 12 in.
D	Bermuda grass	Good stand, mowed to 2-½ in.
	Buffalo grass	Good stand, unmowed, 3 to 6 in.
	Common lespedeza	Excellent stand, unmowed, average 4-½ in.
	Grass-legume mixture, autumn: common lespedeza, Italian ryegrass, orchard grass, redtop	Good stand, unmowed, 4 to 5 in.
	Lespedeza sericea	Good stand, mowed to 2 in.
E	Bermuda grass	Good stand, mowed to 1-½ in.
	Bermuda grass	Burned stubble

Note: Covers classified as shown have been tested in an experimental channel. Covers were green and generally uniform. Source of table is HEC 15.

CLASSIFICATION OF VEGETAL COVERS AS TO DEGREE OF RETARDANCY

Figure 30-6B

PROTECTIVE COVER	UNDERLYING SOIL	τ_p (lb/ft^2)
Class A Vegetation	Erosion Resistant	3.70
	Erodible	3.70
Class B Vegetation	Erosion Resistant	2.10
	Erodible	2.10
Class C Vegetation	Erosion Resistant	1.00
	Erodible	1.00
Class D Vegetation	Erosion Resistant	0.60
	Erodible	0.60
Class E Vegetation	Erosion Resistant	0.35
	Erodible	0.35
Woven Paper	n/a	0.15
Jute Net	n/a	0.45
Single Fiberglass	n/a	0.60
Double Fiberglass	n/a	0.85
Straw with Net	n/a	1.45
Curled Wood Mat	n/a	1.55
Synthetic Mat	n/a	2.00
Plain Grass, Good Cover	Clay	n/a
Plain Grass, Average Cover	Clay	n/a
Plain Grass, Poor Cover	Clay	n/a
Grass, Reinforced with Nylon	Clay	n/a
Dycel with Grass	Clay	n/a
Petraflex with Grass	Clay	n/a
Armorflex with Grass	Clay	n/a
Dymex with Grass	Clay	n/a
Grasscrete	Clay	n/a
Gravel		
D_{50} = 1 in.	n/a	0.40
D_{50} = 2 in.	n/a	0.80
Rock		
D_{50} = 6 in.	n/a	2.50
D_{50} = 12 in.	n/a	5.00
6 in. Gabions	Type I	35
4 in. Geoweb	Type I	10
Soil Cement (8% cement)	Type I	> 45.00
Dycel w/o Grass	Type I	> 32.00
Petraflex w/o Grass	Type I	> 1532
Armorflex w/o Grass	Type I	12.00 – 20.00
Enkamat with 3 in. in Asphalt	Type I	13.00 – 16.00
Enkamat with 1 in. in Asphalt	Type I	< 5.00
Armorflex Class 30 with longitudinal and lateral cables, no grass	Type I	> 34.00
Dycell 100, longitudinal cables, cell filled with mortar	Type I	< 12.00
Concrete construction blocks, granular filter underlayer	Type I	> 20.00
Wedge-shaped blocks with drainage slot	Type I	> 25.00

Type I soil is a silty clay to silty sand (SC-SM) with AASHTO classification A-4(0). *Source: FHWA-RD-89-199*

SUMMARY OF PERMISSIBLE SHEAR STRESS FOR VARIABLE PROTECTION MEASURES

Figure 30-6C

Lining Category	Lining Type	k_s (ft)	*n* value		
			Depth Range		
			0 < 0.5 ft	0.5 ≤ depth ≤ 2.0 ft	> 2.0 ft
Rigid	Concrete	--	0.15	0.13	0.13
	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
Unlined	Bare Soil	--	0.023	0.020	0.020
	Rock Cut	--	0.045	0.035	0.025
Temporary *	Woven Paper Net	0.003	0.016	0.015	0.015
	Jute Net	0.039	0.028	0.022	0.019
	Fiberglass Roving	0.036	0.028	0.022	0.019
	Straw with Net	0.121	0.065	0.033	0.025
	Curled Wood Mat	0.112	0.066	0.035	0.028
	Synthetic Mat	0.066	0.036	0.025	0.021
Gravel Riprap	1 in. D_{50}	0.082	0.044	0.033	0.030
	2 in. D_{50}	0.164	0.066	0.041	0.034
Rock Riprap	6 in. D_{50}	0.492	0.104	0.069	0.035
	12 in. D_{50}	0.984	--	0.078	0.040

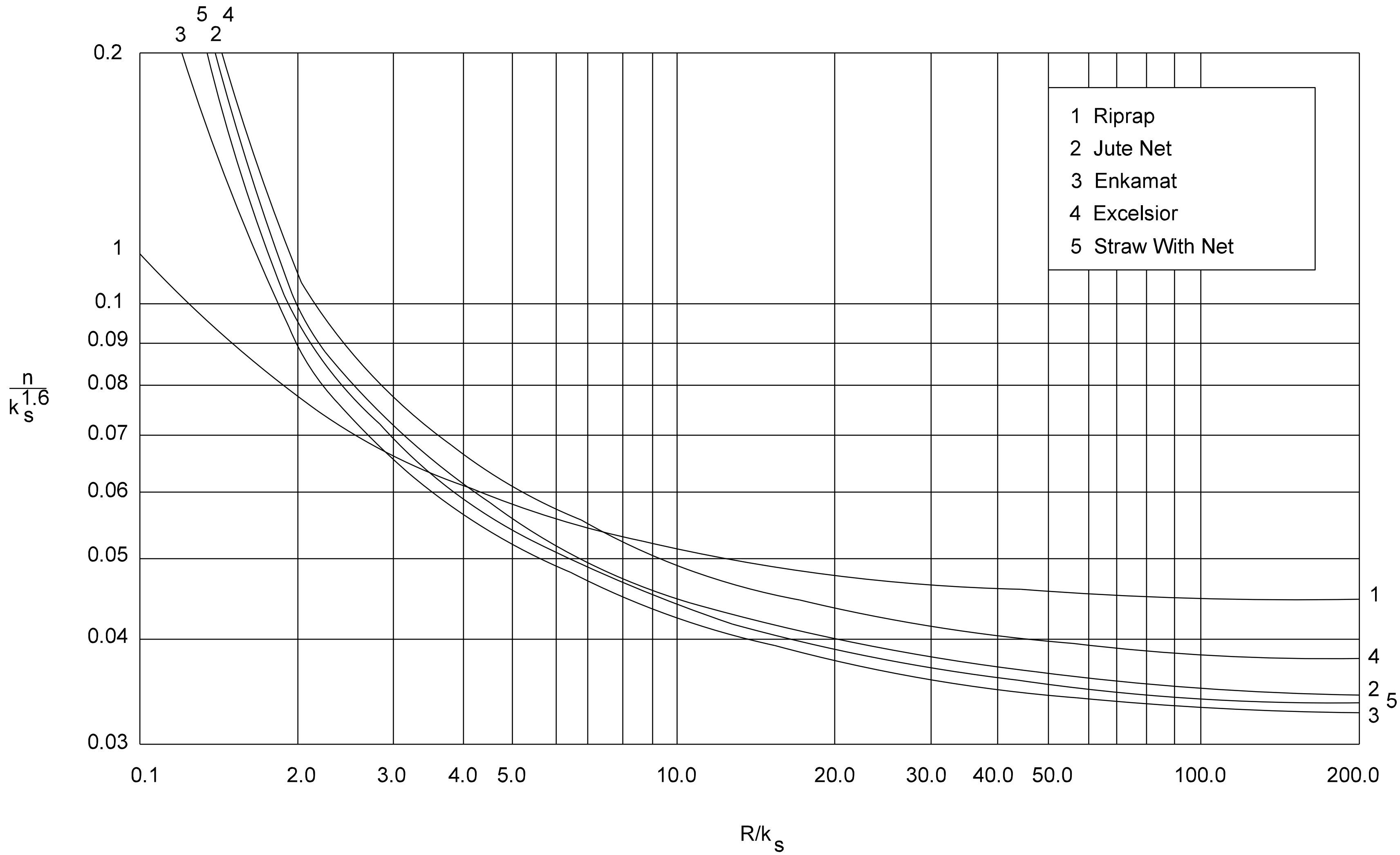
Note: The n *value listed is the representative value for the respective depth range, and varies with the flow depth. For riprap,* $k_s = D_{50}$.

** Some temporary linings become permanent if buried.*

Source: HEC 15

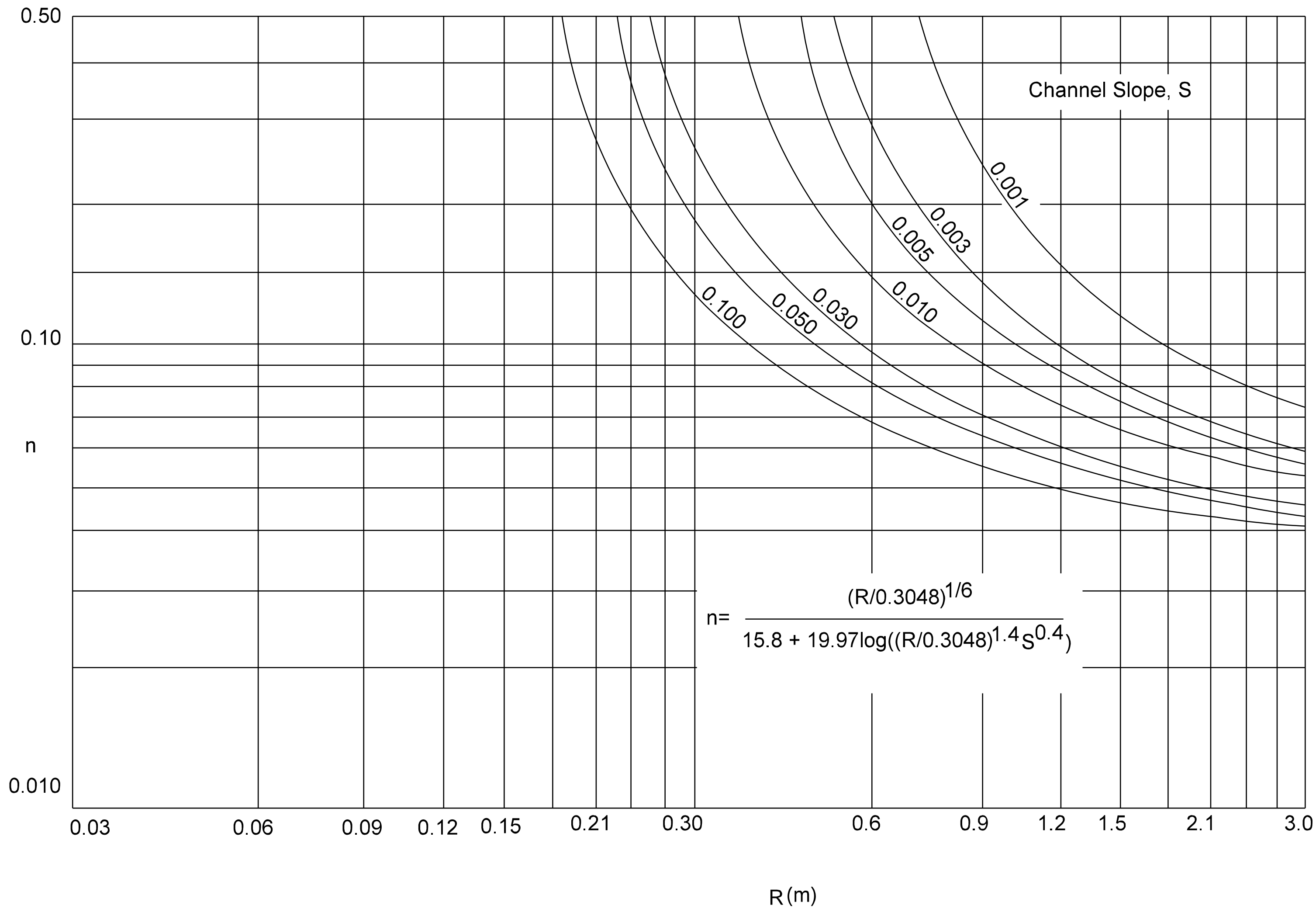
MANNING'S ROUGHNESS COEFFICIENT, *n*, AND ROUGHNESS-ELEMENT HEIGHT, k_S

Figure 30-6D



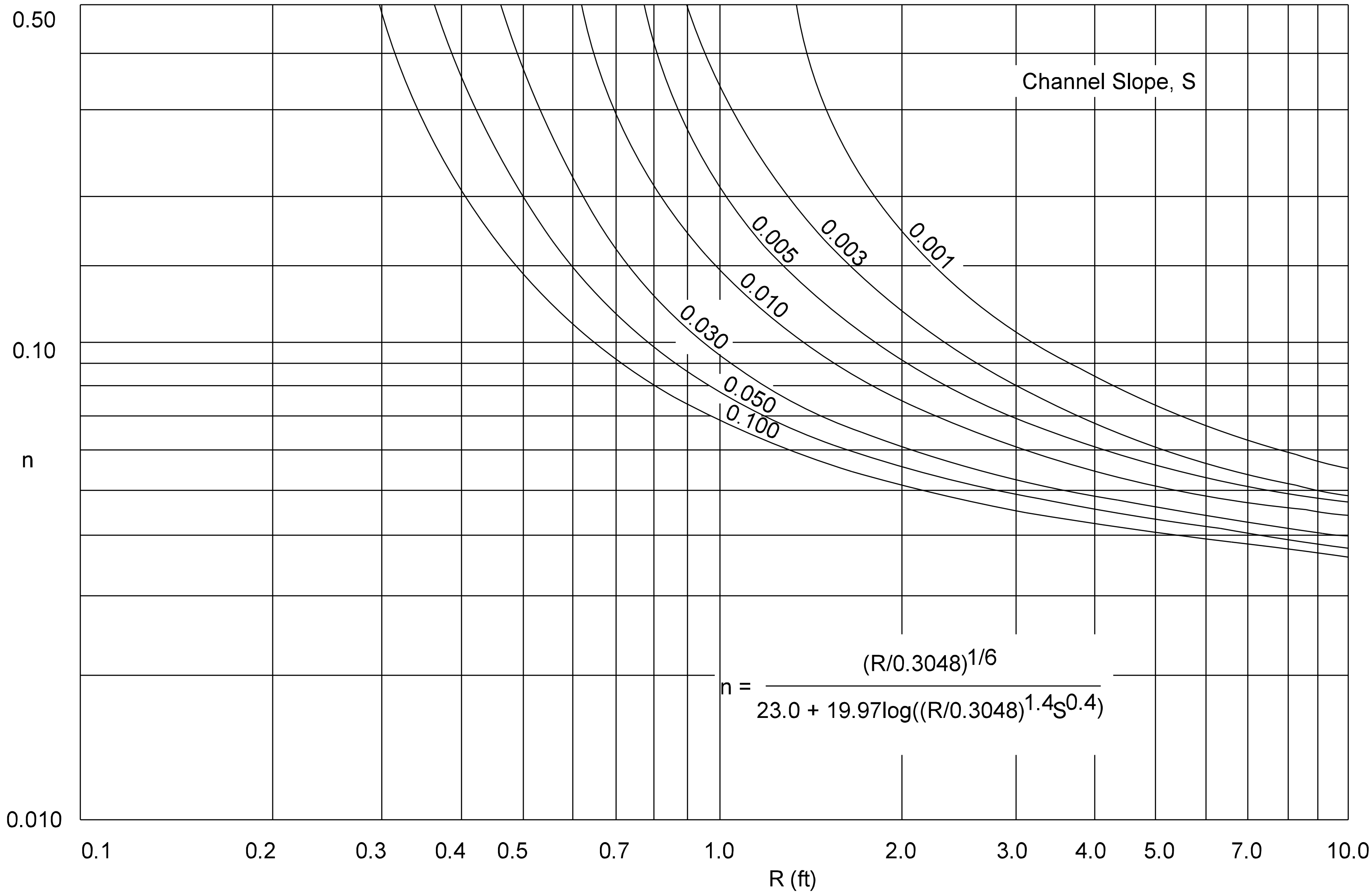
MANNING'S n VERSUS RELATIVE ROUGHNESS FOR
SELECTED LINING TYPES (HEC 15)

Figure 30-6E



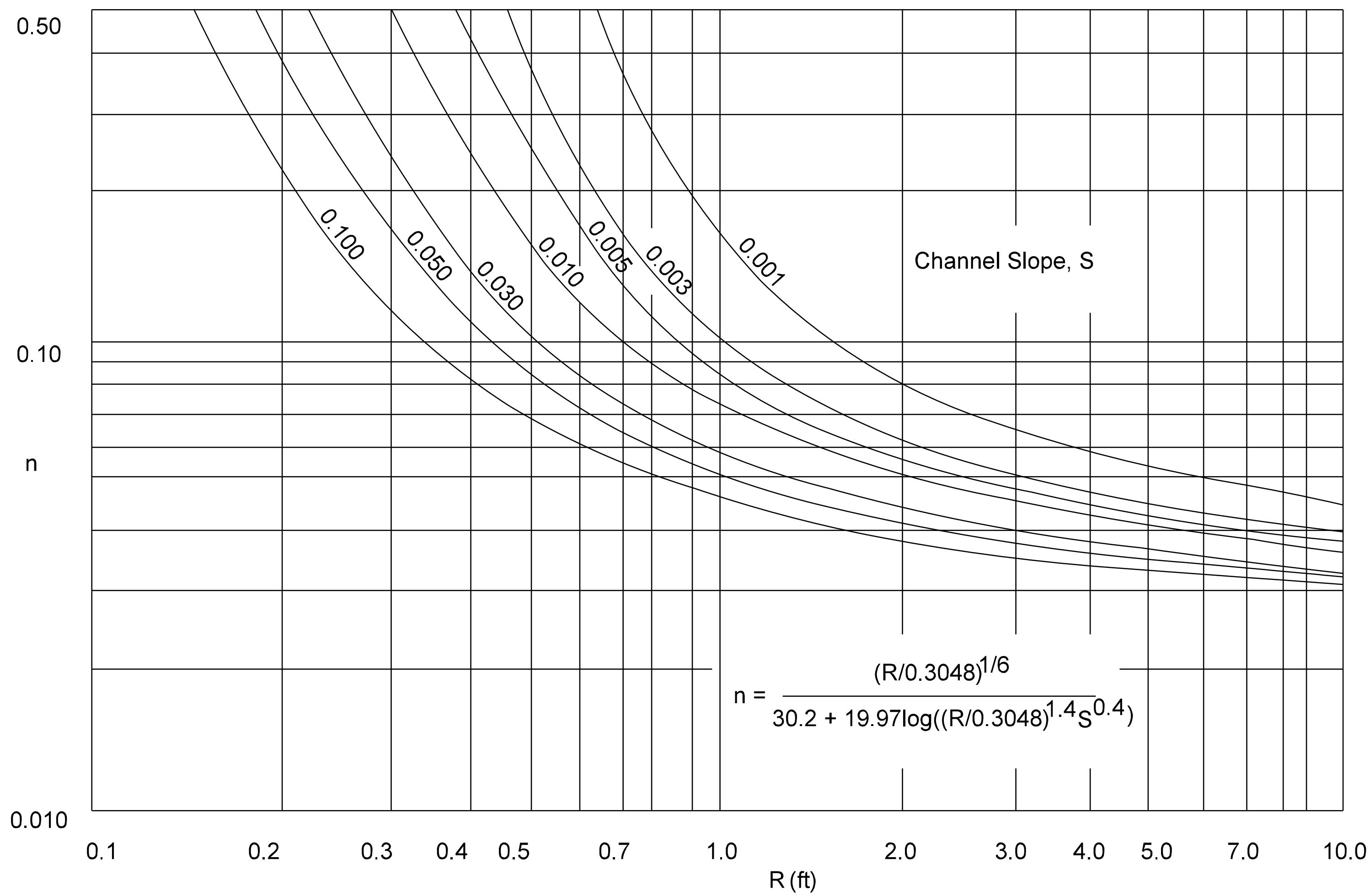
MANNING'S n VERSUS HYDRAULIC RADIUS, R, FOR
CLASS A VEGETATION (HEC 15)

Figure 30-6F



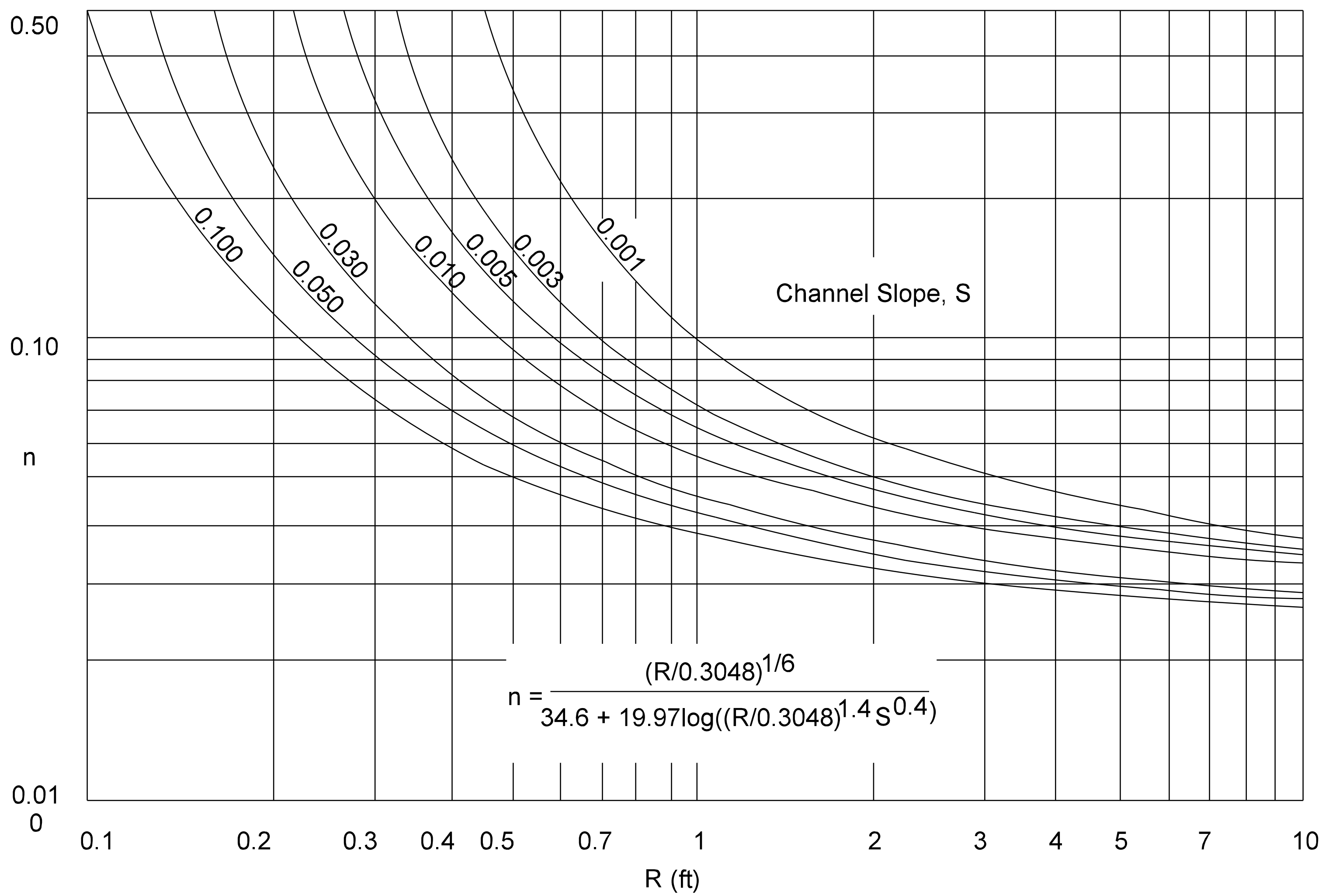
MANNING'S n VERSUS HYDRAULIC RADIUS, R, FOR
CLASS B VEGETATION (HEC 15)

Figure 30-6G



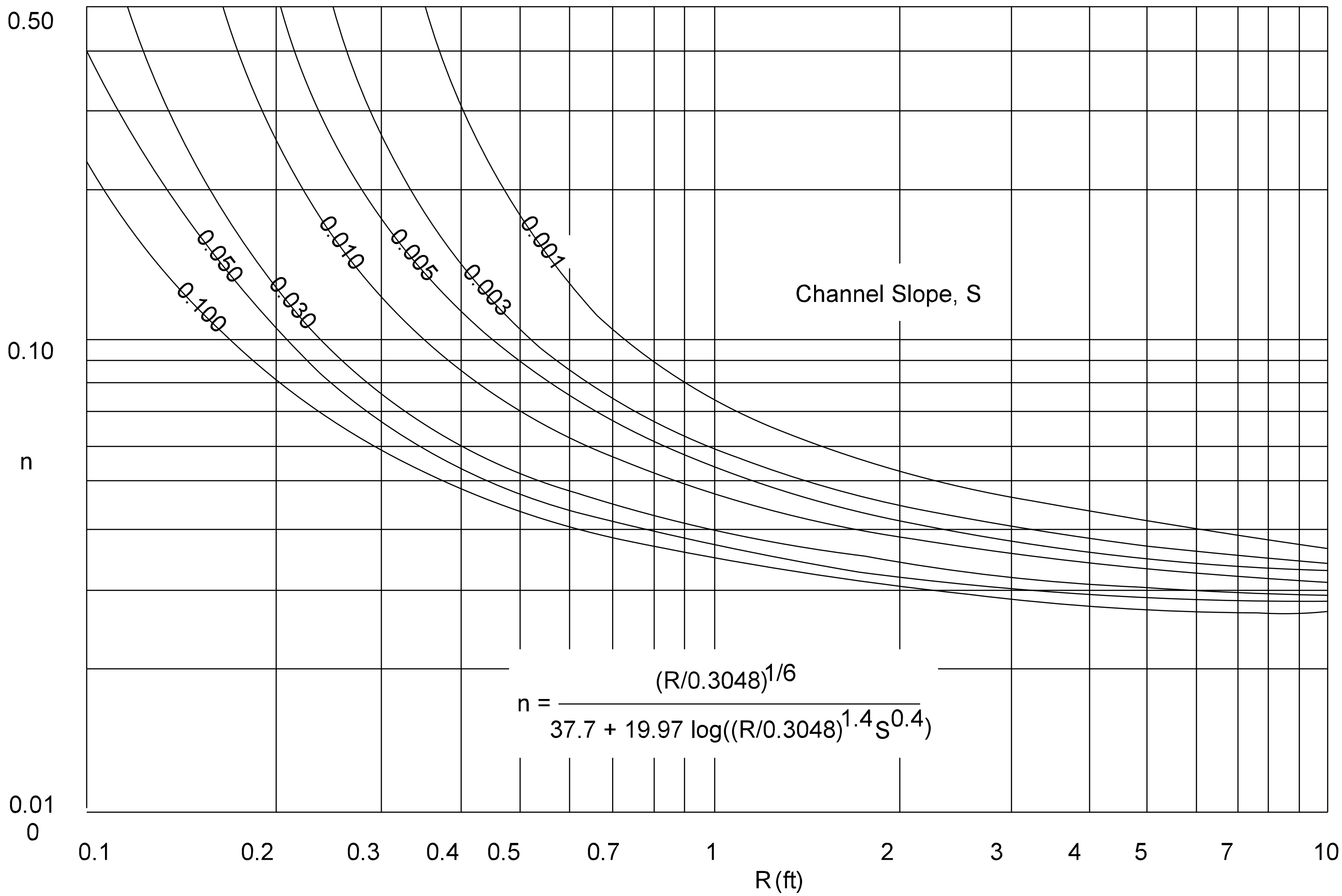
MANNING'S n VERSUS HYDRAULIC RADIUS, R, FOR CLASS C VEGETATION (HEC 15)

Figure 30-6H



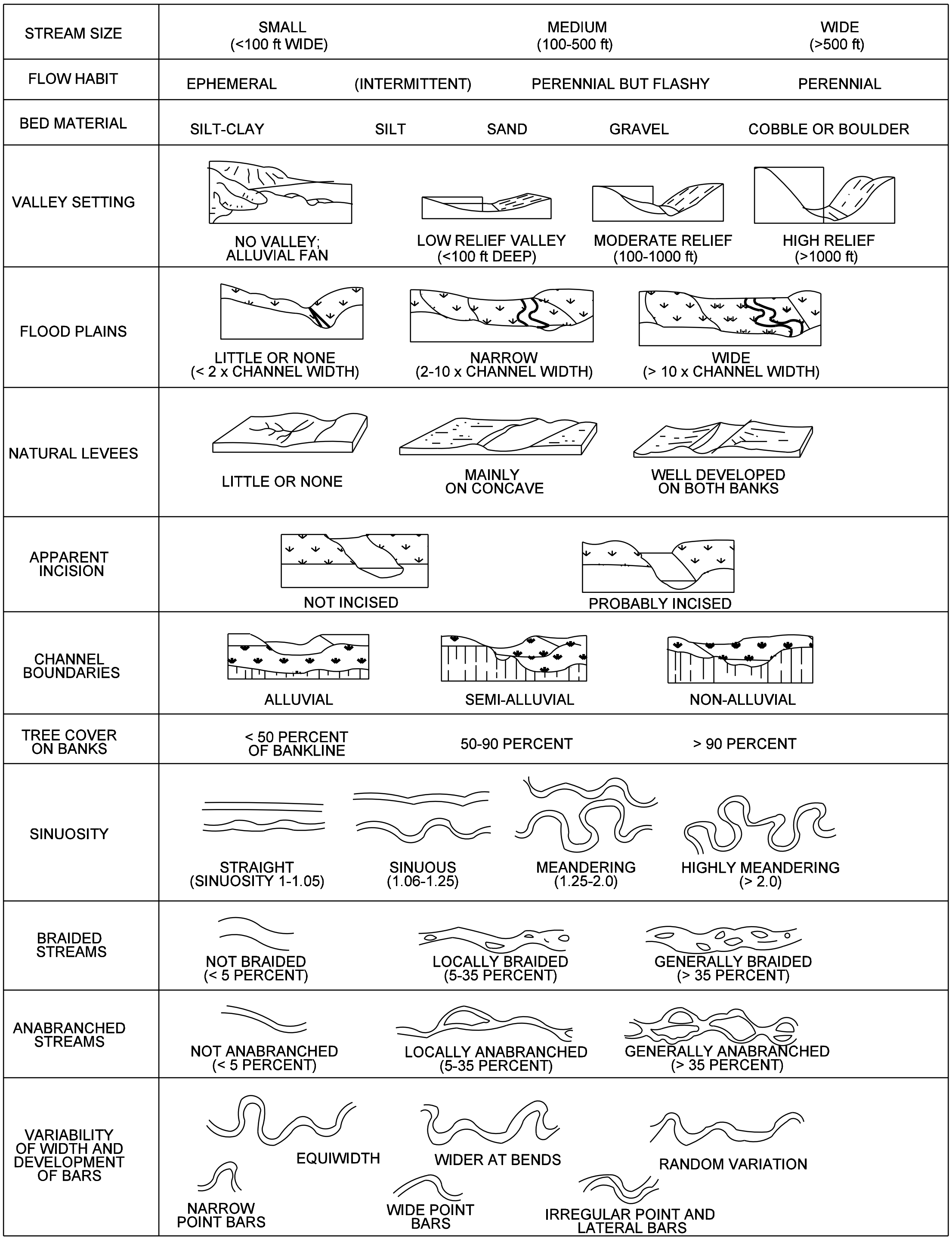
MANNING'S n VERSUS HYDRAULIC RADIUS, R, FOR CLASS D VEGETATION (HEC 15)

Figure 30-6I



MANNING'S n VERSUS HYDRAULIC RADIUS, R, FOR
CLASS E VEGETATION (HEC 15)

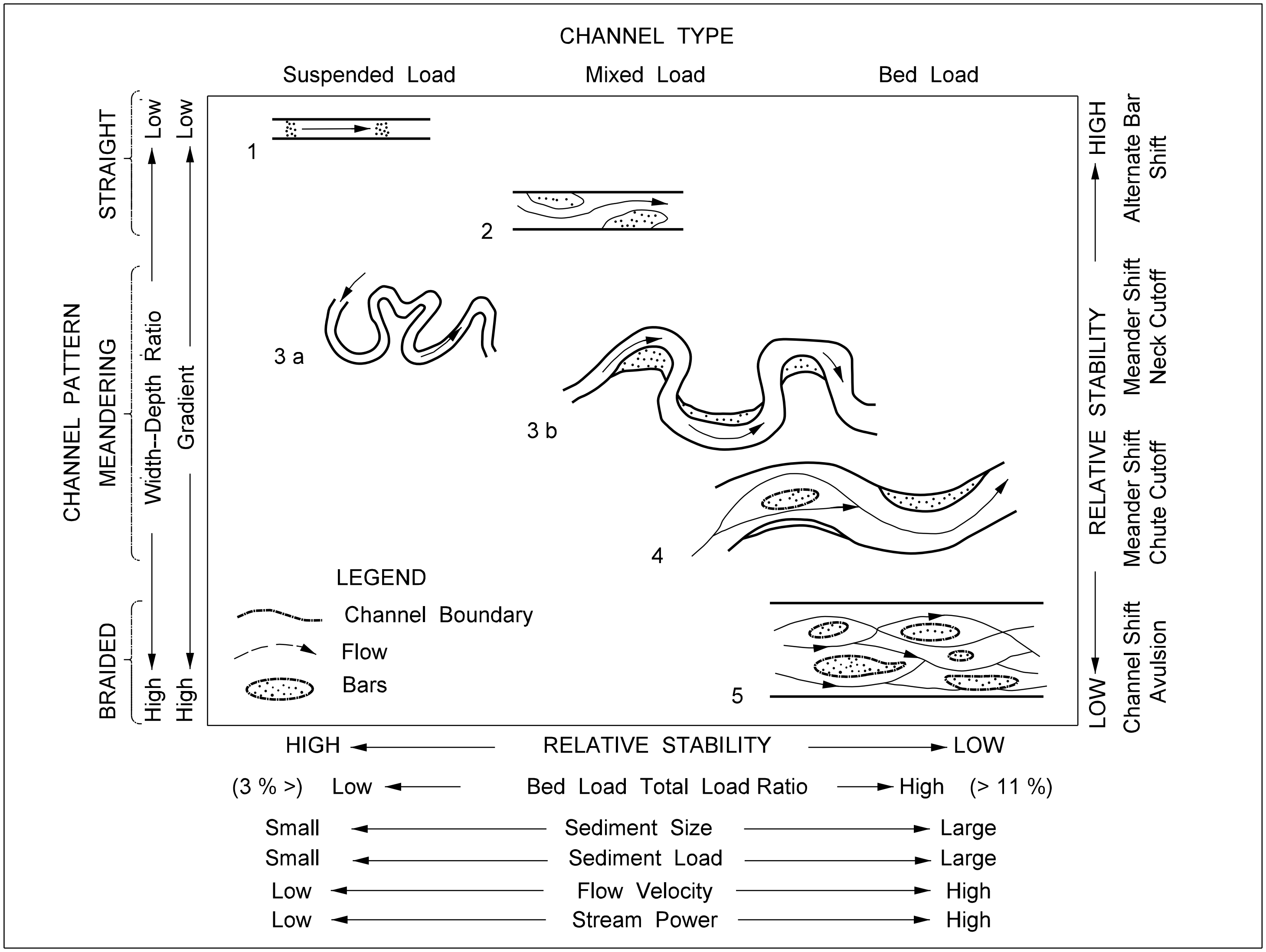
Figure 30-6J



Source: Adapted From Brice and Blodgett, 1978

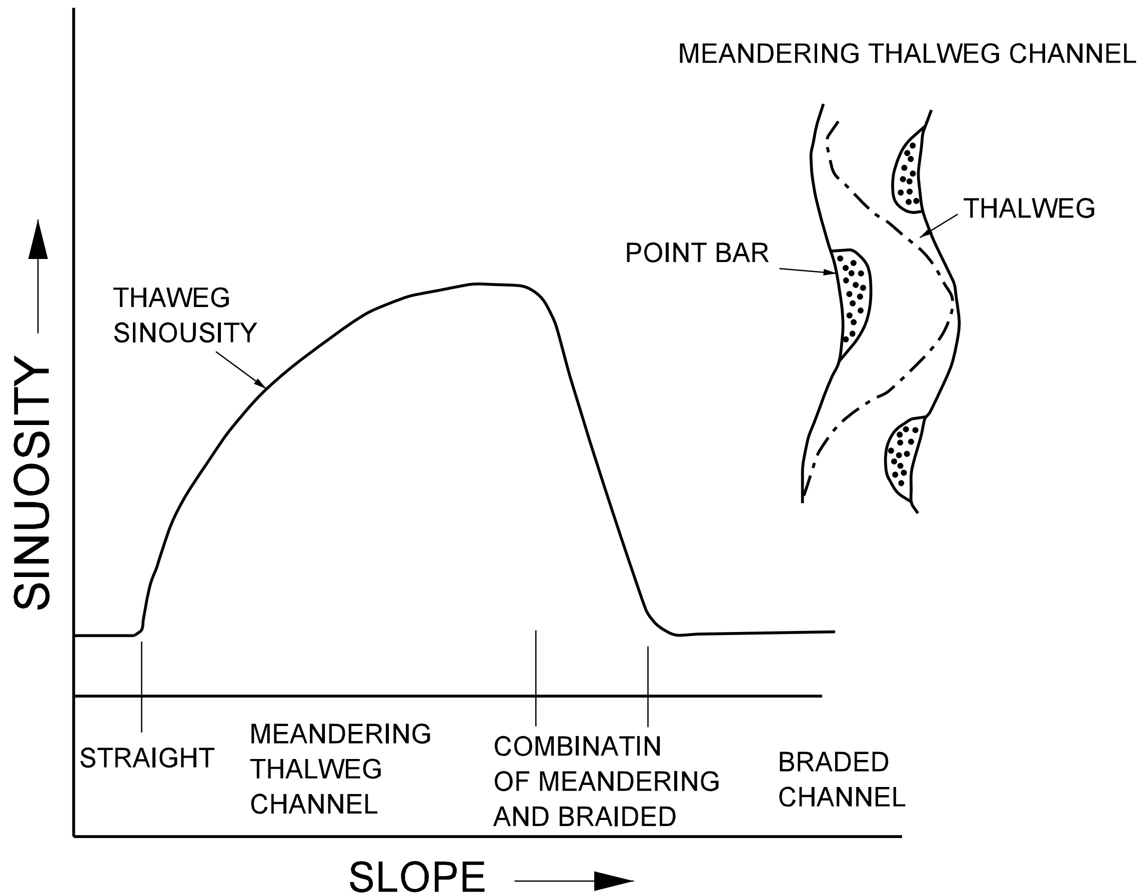
GEOMORPHIC FACTORS THAT AFFECT STREAM STABILITY

Figure 30-7A



CHANNEL CLASSIFICATION AND RELATIVE STABILITY AS HYDRAULIC FACTORS ARE VARIED

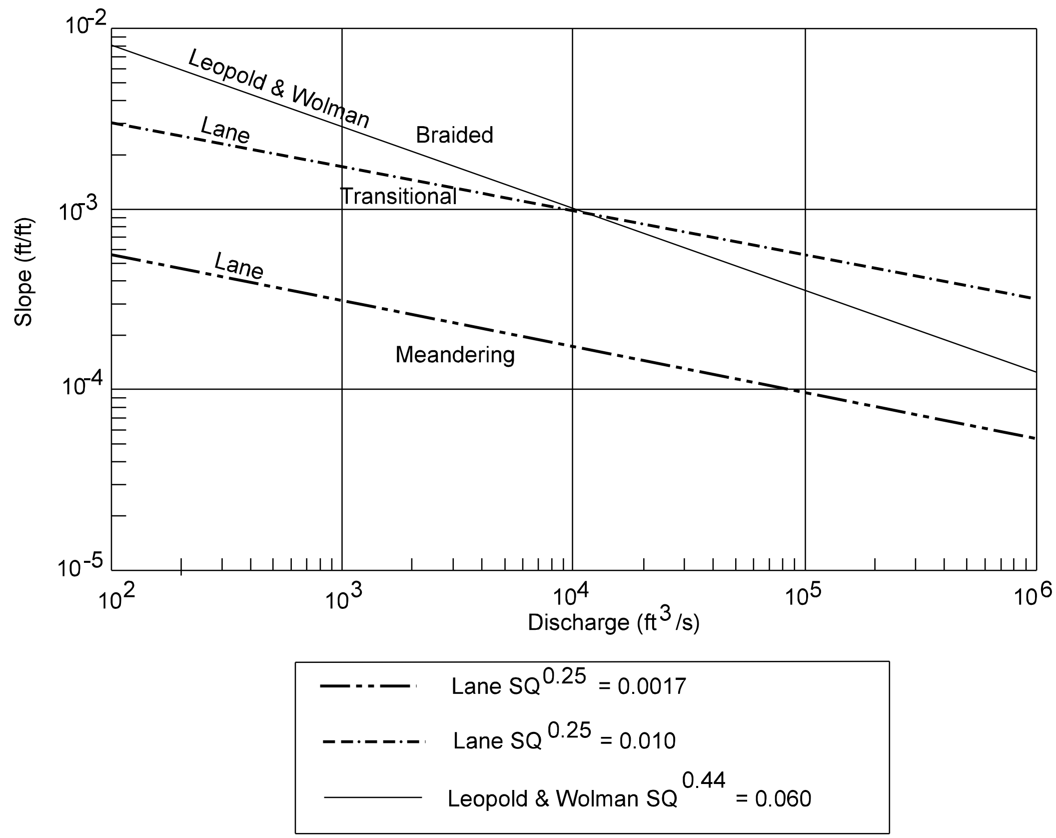
Figure 30-7B



Source: After Richardson et. al., 1988

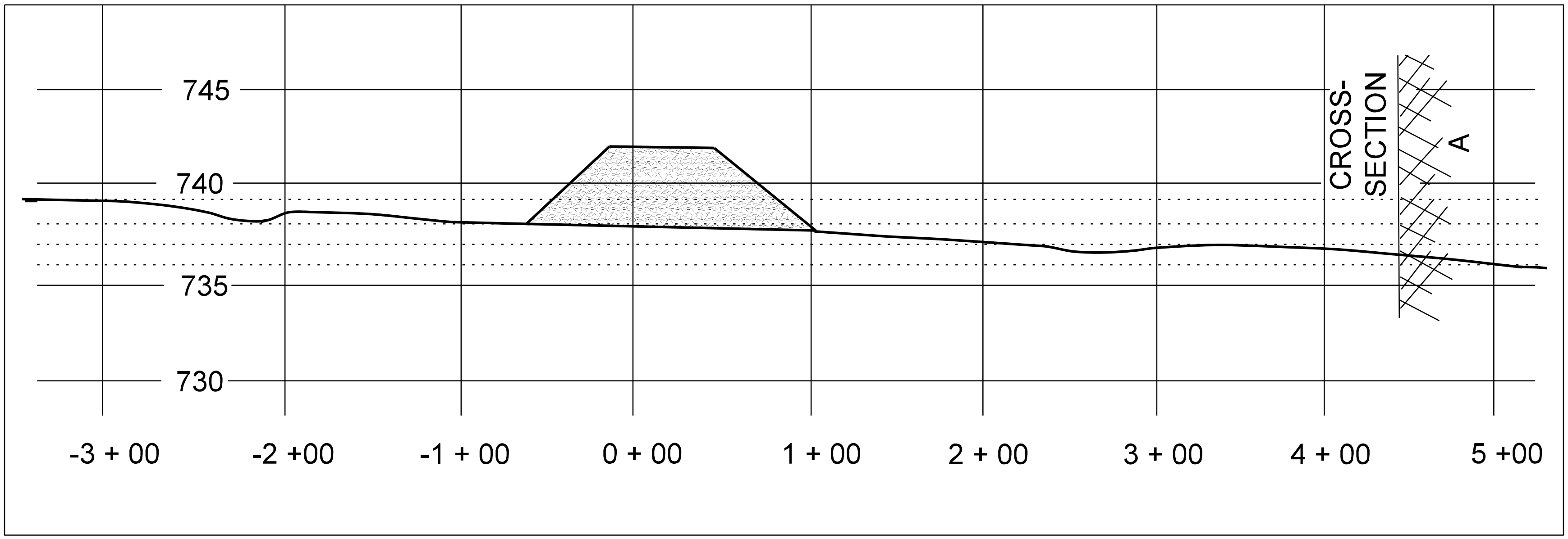
SINUOSITY VERSUS SLOPE WITH CONSTANT DISCHARGE

Figure 30-7C



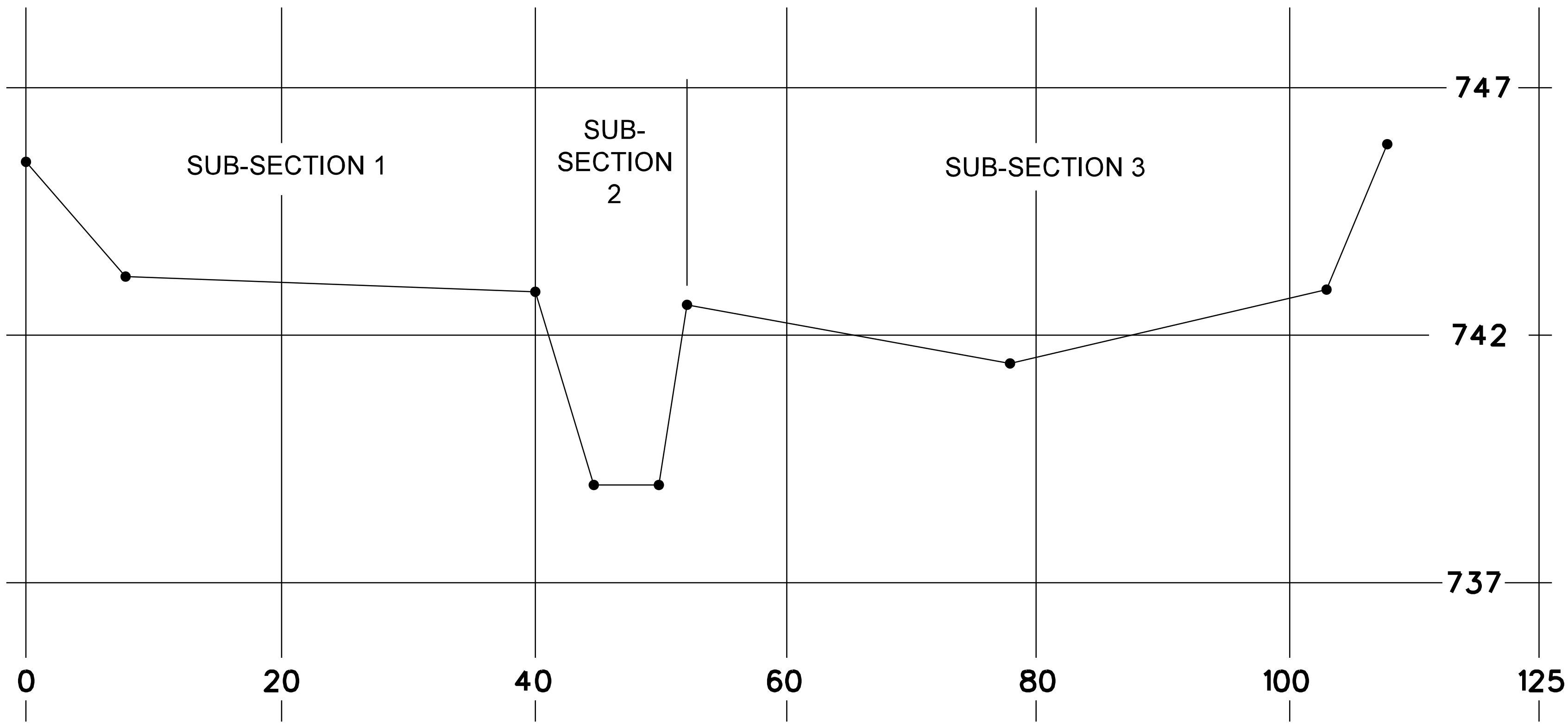
SLOPE-DISCHARGE FOR BRAIDING OR MEANDERING BED STREAMS

Figure 30-7D



SITE DATA EXAMPLE 30-8.1

Figure 30-8.1



STREAM CROSS-SECTION "A" FOR EXAMPLE 30-8.1

Figure 30-8.2

Elevation = 739.63, Slope = 0.0027							
Subsection ID	I	II	III	IV	V	VI	Totals/ Average
Area (ft^2)		6.22					6.22
Wetted Perimeter (ft)		8.03					
Hydraulic Radius (ft)		0.77					
$R^{2/3}$		0.84					
N	0.060	0.035	0.050				
ΔQ (ft^3/s)		11.53					11.53
Subsection Vel. (ft/s)		1.85					1.85

Elevation = 734.00							
Subsection ID	I	II	III	IV	V	VI	Totals/ Average
Area (ft^2)		14.78					14.78
Wetted Perimeter (ft)		11.07					
Hydraulic Radius (ft)		1.33					
$R^{2/3}$		1.21					
N	0.060	0.035	0.050				
ΔQ (ft^3/s)		39.45					39.45
Subsection Vel. (ft/s)		2.67					2.67

Elevation = 741.67							
Subsection ID	I	II	III	IV	V	VI	Totals/ Average
Area (ft^2)		25.44	6.78				32.2
Wetted Perimeter (ft)		15.13	22.27				
Hydraulic Radius (ft)		1.67	0.30				
$R^{2/3}$		1.41	0.45				
N	0.060	0.035	0.050				
ΔQ (ft^3/s)		79.14	4.71				83.85
Subsection Vel. (ft/s)		3.11	0.69				2.60

CHANNEL COMPUTATION SAMPLE

Figure 30-8A

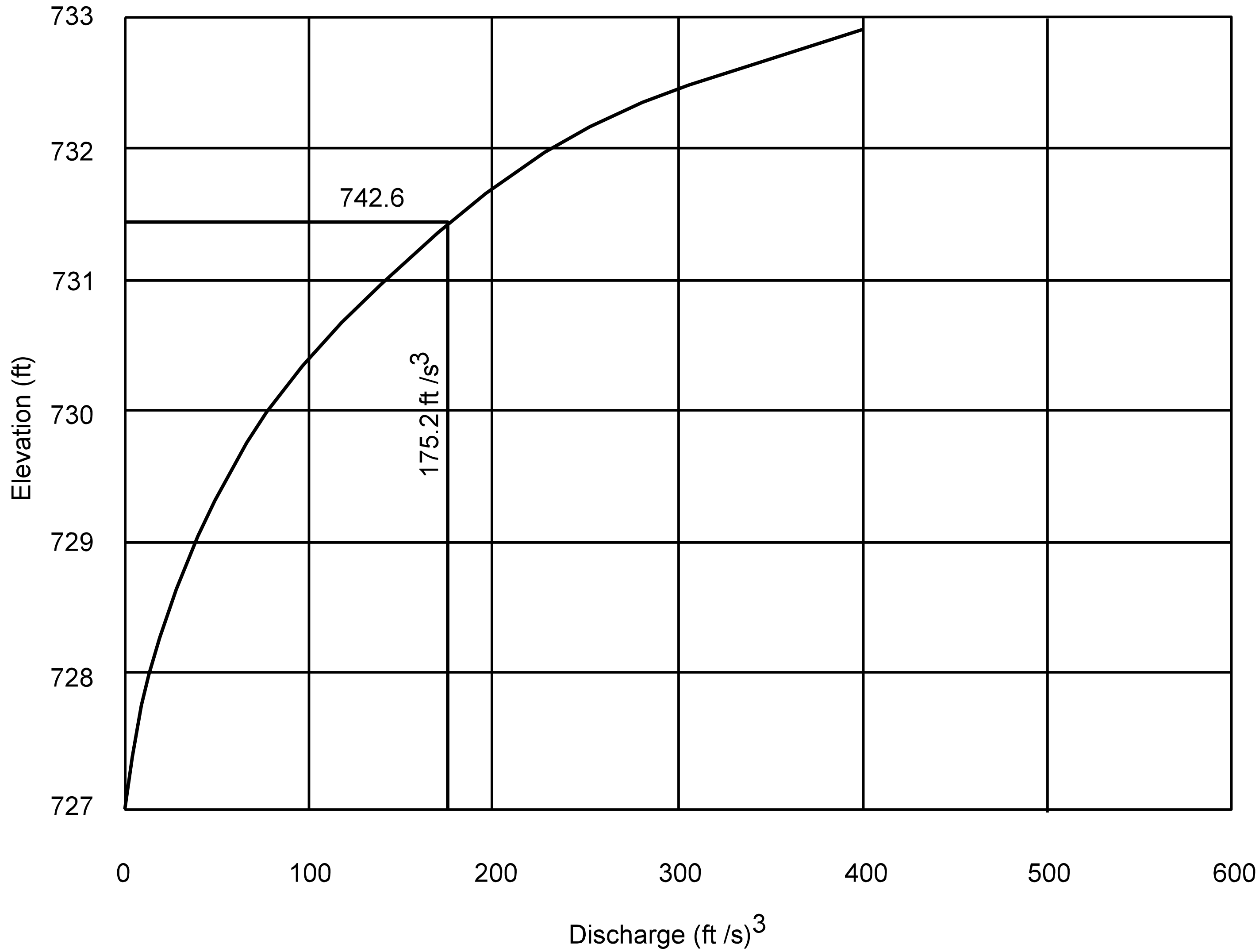
Elevation = 742.67, Slope = 0.0027							
Subsection ID	I	II	III	IV	V	VI	Totals/ Average
Area (ft^2)	1.67	38.33	46.44				86.44
Wetted Perimeter (ft)	16.27	16.27	50.90				
Hydraulic Radius (ft)	0.10	2.36	0.91				
$R^{2/3}$	0.22	1.77	0.94				
N	0.060	0.035	0.050				
ΔQ (ft^3/s)	0.47	149.7	67.4				217.57
Subsection Vel. (ft/s)	0.28	3.91	1.45				2.52

Elevation = 743.70							
Subsection ID	I	II	III	IV	V	VI	Totals/ Average
Area (ft^2)	43.22	50.56	98.89				192.7
Wetted Perimeter (ft)	35.37	16.27	52.93				
Hydraulic Radius (ft)	1.22	3.10	1.87				
$R^{2/3}$	1.14	2.13	1.52				
N	0.060	0.035	0.050				
ΔQ (ft^3/s)	63.41	237.6	232.1				553.1
Subsection Vel. (ft/s)	1.47	4.70	2.35				2.77

Elevation = _______, Slope = _______							
Subsection ID	I	II	III	IV	V	VI	Totals/ Average
Area (ft^2)							
Wetted Perimeter (ft)							
Hydraulic Radius (ft)							
$R^{2/3}$							
N							
ΔQ (ft^3/s)							
Subsection Vel. (ft/s)							

CHANNEL COMPUTATION SAMPLE

Figure 30-8A (Contd.)



EXAMPLE STAGE-DISCHARGE CURVE

Figure 30-8B

```
*F
T1 SOME CREEK WATER SURFACE PROFILE
T2 (WSPRO USER'S MANUAL, J.O. SHEARMAN
T3 SUBCRITICAL FLOW
SI 1
*
Q     283.13      283.13      283.13
WS    182.27      182.58      182.88
*
XS SEC-A 30.48
GR   32.6 , 189.82   40.2 , 186.25   51.8 , 183.42   57.9 , 181.04
GR   75.6 , 179.98   82.6 , 179.98   89.3 , 179.85   94.5 , 179.98
GR   96.9 , 180.77  103.0 , 181.62  106.7 , 183.02  112.2 , 184.03
GR  115.8 , 184.40  121.9 , 184.46
N    .065     .027   .065
SA   51.8    112.2
*
XS SEC-B 64.0
GR   43.9 , 189.61   55.5 , 184.55   62.5 , 181.50   66.7 , 180.92
GR   67.7 , 180.95   83.8 , 179.79   90.5 , 179.76   96.3 , 179.82
GR  105.2 , 180.01  108.8 , 180.95  117.0 , 184.12  117.3 , 184.46
GR  121.9 , 184.58
SA   55.5   117.0
*
XS SEC-C 114.3
GR   60.0 , 188.78   68.3 , 185.43   75.3 , 181.65   83.5 , 180.28
GR   84.1 , 179.67   89.6 , 179.61   93.0 , 179.37   96.6 , 179.24
GR   99.7 , 179.37  107.6 , 179.73  108.2 , 180.68  114.0 , 181.71
GR  119.8 , 183.91  121.0 , 184.27  121.9 , 184.30
SA   68.3   121.0
*
XS SEC-D 152.4
GR   67.1 , 188.69   73.1 , 184.00   77.1 , 180.89   83.2 , 180.80
GR   84.1 , 180.07   89.9 , 180.01   95.1 , 180.01   96.0 , 179.52
GR  100.0 , 179.55  103.6 , 179.58  103.9 , 180.07  113.4 , 181.29
GR  118.6 , 183.91  121.9 , 184.00
SA   73.1   118.6
*
XS SEC-E 195.1
GR   62.2 , 188.66   73.1 , 180.77   78.6 , 180.80   84.1 , 180.52
GR   89.3 , 180.46   94.8 , 180.46  100.3 , 180.37  105.8 , 180.74
GR  111.2 , 180.68  117.0 , 183.18  121.9 , 183.48  122.2 , 185.31
SA   73.1   111.2
*
EX
ER
END
```

WSPRO INPUT DATA FILE

Figure 30-8C

```
*************************  W S P R O ******************************
    Federal Highway Administration  -  U. S. Geological Survey
           Model for Water-Surface Profile Computations
        Run Date & Time: 10/18/95  9:04 am   Version V092695
          Input File:  ex82.wsp    Output File:  ex82.LST
*-----------------------------------------------------------------*

*F
***                    Input Data In Free Format                  ***

T1  SOME CREEK WATER SURFACE PROFILE
T2  (WSPRO USER'S MANUAL, J.O. SHEARMAN
T3  SUBCRITICAL FLOW
SI  1


                        Metric (SI) Units Used in WSPRO

         Quantity            SI Units                 Precision
         ----------          -------------------      ---------
         Length              meters                   0.001
         Depth               meters                   0.001
         Elevation           meters                   0.001
         Widths              meters                   0.001

         Velocity            meters/second            0.001
         Discharge           cubic meters/second      0.001
         Slope               meter/meter              0.001

         Angles              degrees                  0.01
         ----------          -------------------      ---------

Q   283.13   283.13   283.13

 ***   Processing Flow Data; Placing Information into Sequence 1  ***

WS  182.27   182.58   182.88
```

```
          *************************** W S P R O ************************
             Federal Highway Administration  -  U. S. Geological Survey
                    Model for Water-Surface Profile Computations
                  Input Units:  Metric   /  Output Units:  Metric
          *-------------------------------------------------------------*
                           SOME CREEK WATER SURFACE PROFILE
                         (WSPRO USER'S MANUAL, J.O. SHEARMAN
                                  SUBCRITICAL FLOW
                     *-----------------------------------------*
                     * Starting To Process Header Record SEC-A *
                     *-----------------------------------------*

XS SEC-A 30.48
GR   32.6 , 189.82   40.2 , 186.25   51.8 , 183.42   57.9 , 181.04
GR   75.6 , 179.98   82.6 , 179.98   89.3 , 179.85   94.5 , 179.98
GR   96.9 , 180.77  103.0 , 181.62  106.7 , 183.02  112.2 , 184.03
GR  115.8 , 184.40  121.9 , 184.46
N    0.065    0.027   0.065
SA   51.8    112.2

 *** Completed Reading Data Associated With Header Record SEC-A ***
 *** Storing X-Section Data In Temporary File As Record Number 1***

 ***          Data Summary For Header Record SEC-A              ***
SRD Location:      30.  Cross-Section Skew:   .0   Error Code   0
Valley Slope:  .00000   Averaging Conveyance By Geometric Mean.
Energy Loss Coefficients ->  Expansion:  .50   Contraction:   .00

                         X,Y-coordinates (14 pairs)
     X         Y          X           Y          X          Y
 --------  ---------  ----------  ----------  ---------  ---------
  32.600    189.820      40.200     186.250      51.800    183.420
  57.900    181.040      75.600     179.980      82.600    179.980
  89.300    179.850      94.500     179.980      96.900    180.770
 103.000    181.620     106.700     183.020     112.200    184.030
 115.800    184.400     121.900     184.460
 --------  ---------  ----------  ----------  ---------  ---------

                      Minimum and Maximum X,Y-coordinates
Minimum X-Station:    32.600  (associated Y-Elevation:  189.820)
Maximum X-Station:   121.900  (associated Y-Elevation:  184.460)
Minimum Y-Elevation: 179.850  (associated X-Station:     89.300)
Maximum Y-Elevation: 189.820  (associated X-Station:     32.600)

                         Roughness Data (3 Subareas)
                  Roughness   Horizontal
         SubArea  Coefficient Breakpoint
         -------  ----------- ----------
            1        0.065        ---
                     ---        51.800
            2        0.027        ---
                     ---       112.200
            3        0.065        ---
         -------  ----------- ----------

                     *-----------------------------------------*
                     * Finishing Processing Header Record SEC-A *
                     *-----------------------------------------*
```

```
          *************************** W S P R O ************************
             Federal Highway Administration  -  U. S. Geological Survey
                    Model for Water-Surface Profile Computations
                   Input Units:  Metric   /  Output Units:  Metric
          *-------------------------------------------------------------*
                           SOME CREEK WATER SURFACE PROFILE
                         (WSPRO USER'S MANUAL, J.O. SHEARMAN
                                  SUBCRITICAL FLOW
                    *-----------------------------------------*
                    * Starting To Process Header Record SEC-B *
                    *-----------------------------------------*

   XS SEC-B 64.0
   GR   43.9 , 189.61   55.5 , 184.55   62.5 , 181.50   66.7 , 180.92
   GR   67.7 , 180.95   83.8 , 179.79   90.5 , 179.76   96.3 , 179.82
   GR  105.2 , 180.01  108.8 , 180.95  117.0 , 184.12  117.3 , 184.46
   GR  121.9 , 184.58
   SA   55.5   117.0

    *** Completed Reading Data Associated With Header Record SEC-B ***
    *** Storing X-Section Data In Temporary File As Record Number 2***

    ***            Data Summary For Header Record SEC-B            ***
   SRD Location:      64.  Cross-Section Skew:   .0   Error Code   0
   Valley Slope:  .00000   Averaging Conveyance By Geometric Mean.
   Energy Loss Coefficients ->  Expansion:  .50   Contraction:   .00

                            X,Y-coordinates (13 pairs)
         X         Y          X          Y          X          Y
    --------  ---------   ---------  ---------  ---------  ---------
     43.900    189.610      55.500    184.550     62.500    181.500
     66.700    180.920      67.700    180.950     83.800    179.790
     90.500    179.760      96.300    179.820    105.200    180.010
    108.800    180.950     117.000    184.120    117.300    184.460
    121.900    184.580
    --------  ---------   ---------  ---------  ----------  ---------

                        Minimum and Maximum X,Y-coordinates
   Minimum X-Station:    43.900  (associated Y-Elevation:  189.610)
   Maximum X-Station:   121.900  (associated Y-Elevation:  184.580)
   Minimum Y-Elevation: 179.760  (associated X-Station:     90.500)
   Maximum Y-Elevation: 189.610  (associated X-Station:     43.900)

                            Roughness Data (3 Subareas)
                     Roughness   Horizontal
            SubArea  Coefficient Breakpoint
            -------  ----------- ----------
               1        0.065       ---
                        ---        55.500
               2        0.027       ---
                        ---       117.000
               3        0.065       ---
            -------  ----------- ----------

                    *------------------------------------------*
                    * Finishing Processing Header Record SEC-B *
                    *------------------------------------------*
```

```
*************************** W S P R O ************************
   Federal Highway Administration  -  U. S. Geological Survey
          Model for Water-Surface Profile Computations
         Input Units:  Metric   /  Output Units:  Metric
*---------------------------------------------------------------*
                  SOME CREEK WATER SURFACE PROFILE
                 (WSPRO USER'S MANUAL, J.O. SHEARMAN
                          SUBCRITICAL FLOW
          *-----------------------------------------*
          * Starting To Process Header Record SEC-C *
          *-----------------------------------------*

XS SEC-C 114.3
GR   60.0 , 188.78   68.3 , 185.43   75.3 , 181.65   83.5 , 180.28
GR   84.1 , 179.67   89.6 , 179.61   93.0 , 179.37   96.6 , 179.24
GR   99.7 , 179.37  107.6 , 179.73  108.2 , 180.68  114.0 , 181.71
GR  119.8 , 183.91  121.0 , 184.27  121.9 , 184.30
SA   68.3   121.0

 *** Completed Reading Data Associated With Header Record SEC-C ***
 *** Storing X-Section Data In Temporary File As Record Number 3***

 ***       Data Summary For Header Record SEC-C                ***
SRD Location:     114.  Cross-Section Skew:   .0   Error Code   0
Valley Slope:  .00000   Averaging Conveyance By Geometric Mean.
Energy Loss Coefficients ->  Expansion:  .50   Contraction:   .00

                       X,Y-coordinates (15 pairs)
    X          Y           X           Y           X          Y
 --------  ---------  ----------  ---------  ---------  ---------
  60.000    188.780     68.300     185.430     75.300    181.650
  83.500    180.280     84.100     179.670     89.600    179.610
  93.000    179.370     96.600     179.240     99.700    179.370
 107.600    179.730    108.200     180.680    114.000    181.710
 119.800    183.910    121.000     184.270    121.900    184.300
 --------  ---------  ----------  ---------  ---------  ---------

                    Minimum and Maximum X,Y-coordinates
Minimum X-Station:    60.000  (associated Y-Elevation:  188.780)
Maximum X-Station:   121.900  (associated Y-Elevation:  184.300)
Minimum Y-Elevation: 179.240  (associated X-Station:     96.600)
Maximum Y-Elevation: 188.780  (associated X-Station:     60.000)

                       Roughness Data (3 Subareas)
                  Roughness   Horizontal
         SubArea  Coefficient Breakpoint
         -------  ----------- ----------
            1        0.065        ---
                     ---        68.300
            2        0.027        ---
                     ---       121.000
            3        0.065        ---
         -------  ----------- ----------

          *------------------------------------------*
          * Finishing Processing Header Record SEC-C *
          *------------------------------------------*
```

```
                   *************************** W S P R O ************************
                      Federal Highway Administration  -  U. S. Geological Survey
                             Model for Water-Surface Profile Computations
                            Input Units:  Metric   /  Output Units:  Metric
                   *---------------------------------------------------------------*
                                   SOME CREEK WATER SURFACE PROFILE
                                  (WSPRO USER'S MANUAL, J.O. SHEARMAN
                                           SUBCRITICAL FLOW
                             *-----------------------------------------*
                             * Starting To Process Header Record SEC-D *
                             *-----------------------------------------*

            XS SEC-D 152.4
            GR   67.1 , 188.69   73.1 , 184.00   77.1 , 180.89   83.2 , 180.80
            GR   84.1 , 180.07   89.9 , 180.01   95.1 , 180.01   96.0 , 179.52
            GR  100.0 , 179.55  103.6 , 179.58  103.9 , 180.07  113.4 , 181.29
            GR  118.6 , 183.91  121.9 , 184.00
            SA   73.1   118.6

             *** Completed Reading Data Associated With Header Record SEC-D ***
             *** Storing X-Section Data In Temporary File As Record Number 4***

             ***          Data Summary For Header Record SEC-D              ***
            SRD Location:     152.  Cross-Section Skew:   .0   Error Code   0
            Valley Slope:  .00000   Averaging Conveyance By Geometric Mean.
            Energy Loss Coefficients ->  Expansion:  .50   Contraction:   .00

                                      X,Y-coordinates (14 pairs)
                X          Y            X            Y           X          Y
             --------  ---------  ----------   ---------   ---------  ---------
              67.100    188.690      73.100     184.000      77.100    180.890
              83.200    180.800      84.100     180.070      89.900    180.010
              95.100    180.010      96.000     179.520     100.000    179.550
             103.600    179.580     103.900     180.070     113.400    181.290
             118.600    183.910     121.900     184.000
             --------  ---------  ----------   ---------   ---------  ---------

                                  Minimum and Maximum X,Y-coordinates
            Minimum X-Station:    67.100  (associated Y-Elevation:  188.690)
            Maximum X-Station:   121.900  (associated Y-Elevation:  184.000)
            Minimum Y-Elevation: 179.520  (associated X-Station:     96.000)
            Maximum Y-Elevation: 188.690  (associated X-Station:     67.100)

                                      Roughness Data (3 Subareas)
                                  Roughness   Horizontal
                         SubArea  Coefficient Breakpoint
                         -------  ----------- ----------
                            1        0.065        ---
                                     ---       73.100
                            2        0.027        ---
                                     ---      118.600
                            3        0.065        ---
                         -------  ----------- ----------

                             *------------------------------------------*
                             * Finishing Processing Header Record SEC-D *
                             *------------------------------------------*
```

```
      *************************** W S P R O ***********************
        Federal Highway Administration  -  U. S. Geological Survey
               Model for Water-Surface Profile Computations
              Input Units:  Metric   /  Output Units:  Metric
     *---------------------------------------------------------------*
                      SOME CREEK WATER SURFACE PROFILE
                     (WSPRO USER'S MANUAL, J.O. SHEARMAN
                              SUBCRITICAL FLOW
                *-----------------------------------------*
                * Starting To Process Header Record SEC-E *
                *-----------------------------------------*

XS SEC-E 195.1
GR   62.2 , 188.66   73.1 , 180.77   78.6 , 180.80   84.1 , 180.52
GR   89.3 , 180.46   94.8 , 180.46  100.3 , 180.37  105.8 , 180.74
GR  111.2 , 180.68  117.0 , 183.18  121.9 , 183.48  122.2 , 185.31
SA   73.1   111.2

 *** Completed Reading Data Associated With Header Record SEC-E ***
 *** Storing X-Section Data In Temporary File As Record Number 5***

 ***          Data Summary For Header Record SEC-E              ***
SRD Location:     195.  Cross-Section Skew:   .0   Error Code   0
Valley Slope:  .00000   Averaging Conveyance By Geometric Mean.
Energy Loss Coefficients ->  Expansion:  .50   Contraction:   .00

                         X,Y-coordinates (12 pairs)
     X         Y           X           Y           X          Y
 --------  ---------  ----------  ---------  ---------  ---------
  62.200    188.660      73.100    180.770      78.600    180.800
  84.100    180.520      89.300    180.460      94.800    180.460
 100.300    180.370     105.800    180.740     111.200    180.680
 117.000    183.180     121.900    183.480     122.200    185.310
 --------  ---------  ----------  ---------  ---------  ---------

                     Minimum and Maximum X,Y-coordinates
Minimum X-Station:    62.200  (associated Y-Elevation:  188.660)
Maximum X-Station:   122.200  (associated Y-Elevation:  185.310)
Minimum Y-Elevation: 180.370  (associated X-Station:    100.300)
Maximum Y-Elevation: 188.660  (associated X-Station:     62.200)

                         Roughness Data (3 Subareas)
                  Roughness   Horizontal
         SubArea  Coefficient Breakpoint
         -------  ----------- ----------
            1        0.065        ---
                     ---        73.100
            2        0.027        ---
                     ---       111.200
            3        0.065        ---
         -------  ----------- ----------

                *-------------------------------------------*
                * Finishing Processing Header Record SEC-E *
                *-------------------------------------------*
```

```
*************************** W S P R O *******************************
      Federal Highway Administration  -  U. S. Geological Survey
             Model for Water-Surface Profile Computations
           Input Units:  Metric   /  Output Units:  Metric
*-------------------------------------------------------------------*

                    SOME CREEK WATER SURFACE PROFILE
                  (WSPRO USER'S MANUAL, J.O. SHEARMAN
                           SUBCRITICAL FLOW
EX

          *=================================================*
          *    Summary of Boundary Condition Information    *
          *=================================================*

         Reach          Water Surface      Friction
 #      Discharge        Elevation          Slope      Flow Regime
---     ---------      -------------      ---------    -----------
 1        283.12          182.279          ******      Sub-Critical
 2        283.12          182.589          ******      Sub-Critical
 3        283.12          182.889          ******      Sub-Critical
---     ---------      -------------      ---------    -----------

             *===========================================*
             *    Beginning 3 Profile Calculation(s)    *
             *===========================================*
```

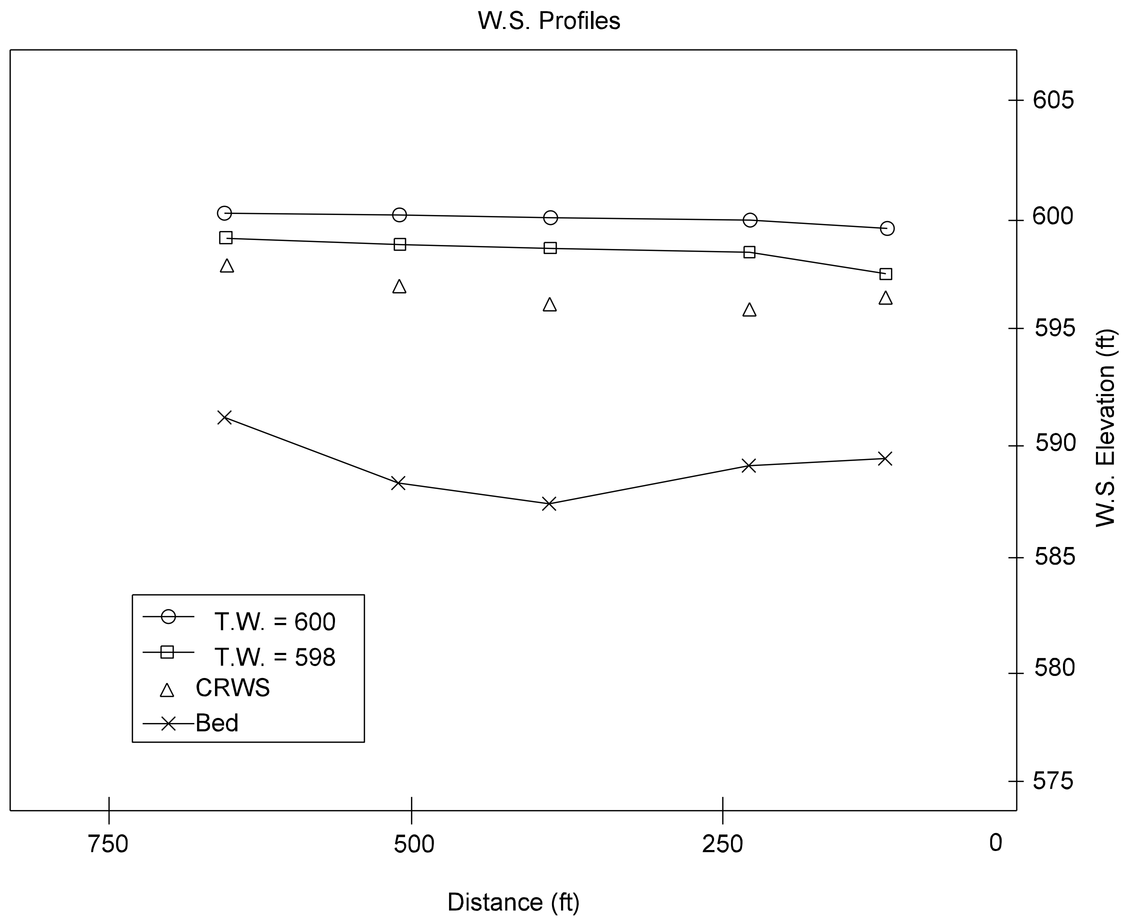
```
*************************** W S P R O *******************************
      Federal Highway Administration  -  U. S. Geological Survey
             Model for Water-Surface Profile Computations
            Input Units:  Metric   /  Output Units:  Metric
*--------------------------------------------------------------------*

                     SOME CREEK WATER SURFACE PROFILE
                   (WSPRO USER'S MANUAL, J.O. SHEARMAN
                            SUBCRITICAL FLOW

                    WSEL      VHD        Q        AREA       SRDL      LEW
                    EGEL       HF        V          K        FLEN      REW
                    CRWS       HO     FR #         SF       ALPHA      ERR
                  ---------  -----  -------    -------  ---------  -------
Section: SEC-A      182.278  .518   283.124     88.821  *********   54.749
Header Type: XS     182.797  *****    3.187    4791.65  *********  104.723
SRD:     30.481     181.963  *****     .763      *****      1.000   ******

===135 CONVEYANCE RATIO OUTSIDE OF RECOMMENDED LIMITS.
                  "SEC-B"   KRATIO = 1.42

Section: SEC-B      182.554   .324  283.124    112.218     33.521   60.102
Header Type: XS     182.879   .082    2.522    6807.56     33.521  112.933
SRD:     64.003     181.799   .000     .553      .0025      1.000     .000

Section: SEC-C      182.603   .415  283.124     99.251     50.302   73.554
Header Type: XS     183.018   .093    2.852    6305.92     50.302  116.337
SRD:    114.305     181.855   .045     .598      .0019      1.000     .000

Section: SEC-D      182.652    .49  283.124     91.159     38.101   74.848
Header Type: XS     183.144   .086    3.105    5581.40     38.101  116.091
SRD:    152.407     182.094   .038     .667      .0023      1.000     .002

Section: SEC-E      182.729   .579  283.124     88.929     42.702   70.408
Header Type: XS     183.308   .119    3.183    5113.09     42.702  115.939
SRD:    195.109     182.357   .043     .770      .0028      1.120     .003
```

```
*************************** W S P R O ********************************
      Federal Highway Administration  -  U. S. Geological Survey
             Model for Water-Surface Profile Computations
           Input Units:  Metric   /  Output Units:  Metric
*----------------------------------------------------------------------*

                     SOME CREEK WATER SURFACE PROFILE
                    (WSPRO USER'S MANUAL, J.O. SHEARMAN
                             SUBCRITICAL FLOW

                        WSEL        VHD        Q         AREA        SRDL      LEW
                        EGEL         HF        V           K         FLEN      REW
                        CRWS         HO       FR #        SF        ALPHA      ERR
                     ---------    -----    -------     -------  ---------  -------
Section: SEC-A         182.588     .373    283.124     104.563  *********   53.954
Header Type: XS        182.962    *****      2.707     6149.63  *********  105.542
SRD:      30.481       181.963    *****       .607       *****      1.000   ******

Section: SEC-B         182.746     .272    283.124     122.420     33.521   59.662
Header Type: XS        183.018     .056      2.312     7772.47     33.521  113.428
SRD:      64.003       181.799     .000       .489       .0017      1.000    -.001

Section: SEC-C         182.776     .358    283.124     106.739     50.302   73.232
Header Type: XS        183.135     .073      2.652     7028.30     50.302  116.795
SRD:     114.305       181.855     .043       .541      .0015       1.000     .000

Section: SEC-D         182.812     .427    283.124      97.804     38.101   74.642
Header Type: XS        183.239     .069      2.894     6216.38     38.101  116.409
SRD:     152.407       182.094     .034       .604      .0018       1.000     .000

Section: SEC-E         182.868     .508    283.124      95.291     42.702   70.216
Header Type: XS        183.376     .096      2.970     5687.75     42.702  116.262
SRD:     195.109       182.357     .040       .701      .0023       1.129    -.001
```

```
*************************** W S P R O *******************************
       Federal Highway Administration  -  U. S. Geological Survey
              Model for Water-Surface Profile Computations
             Input Units:  Metric   /  Output Units:  Metric
*--------------------------------------------------------------------*

                      SOME CREEK WATER SURFACE PROFILE
                    (WSPRO USER'S MANUAL, J.O. SHEARMAN
                             SUBCRITICAL FLOW

                    WSEL        VHD       Q        AREA       SRDL      LEW
                    EGEL         HF       V          K        FLEN      REW
                    CRWS         HO    FR #         SF       ALPHA      ERR
                 ---------    -----  -------    -------  ---------  -------
Section: SEC-A    182.888      .282  283.124    120.274  *********   53.185
Header Type: XS   183.171     *****    2.353    7604.10  *********  106.335
SRD:     30.481   181.963     *****     .500     *****       1.000   ******

Section: SEC-B    182.987      .222  283.124    135.562     33.521   59.107
Header Type: XS   183.210      .038    2.088    9071.46     33.521  114.054
SRD:     64.003   181.799      .000     .425      .0012      1.000     .000

Section: SEC-C    183.004      .299  283.124    116.770     50.302   72.811
Header Type: XS   183.304      .055    2.424    8030.60     50.302  117.395
SRD:    114.305   181.855      .038     .478      .0011      1.000     .000

Section: SEC-D    183.028      .357  283.124    106.930     38.101   74.363
Header Type: XS   183.386      .053    2.647    7122.06     38.101  116.839
SRD:    152.407   182.094      .028     .533      .0014      1.000     .000

Section: SEC-E    183.067      .427  283.124    104.524     42.702   69.941
Header Type: XS   183.494      .073    2.708    6554.45     42.702  116.723
SRD:    195.109   182.357      .034     .618      .0017      1.141    -.001

ER

 ********************  Normal end of WSPRO execution.  ********************
  *****************  Elapsed Time:  0 Minutes  8 Seconds  *****************
```

WSPRO RESULTS

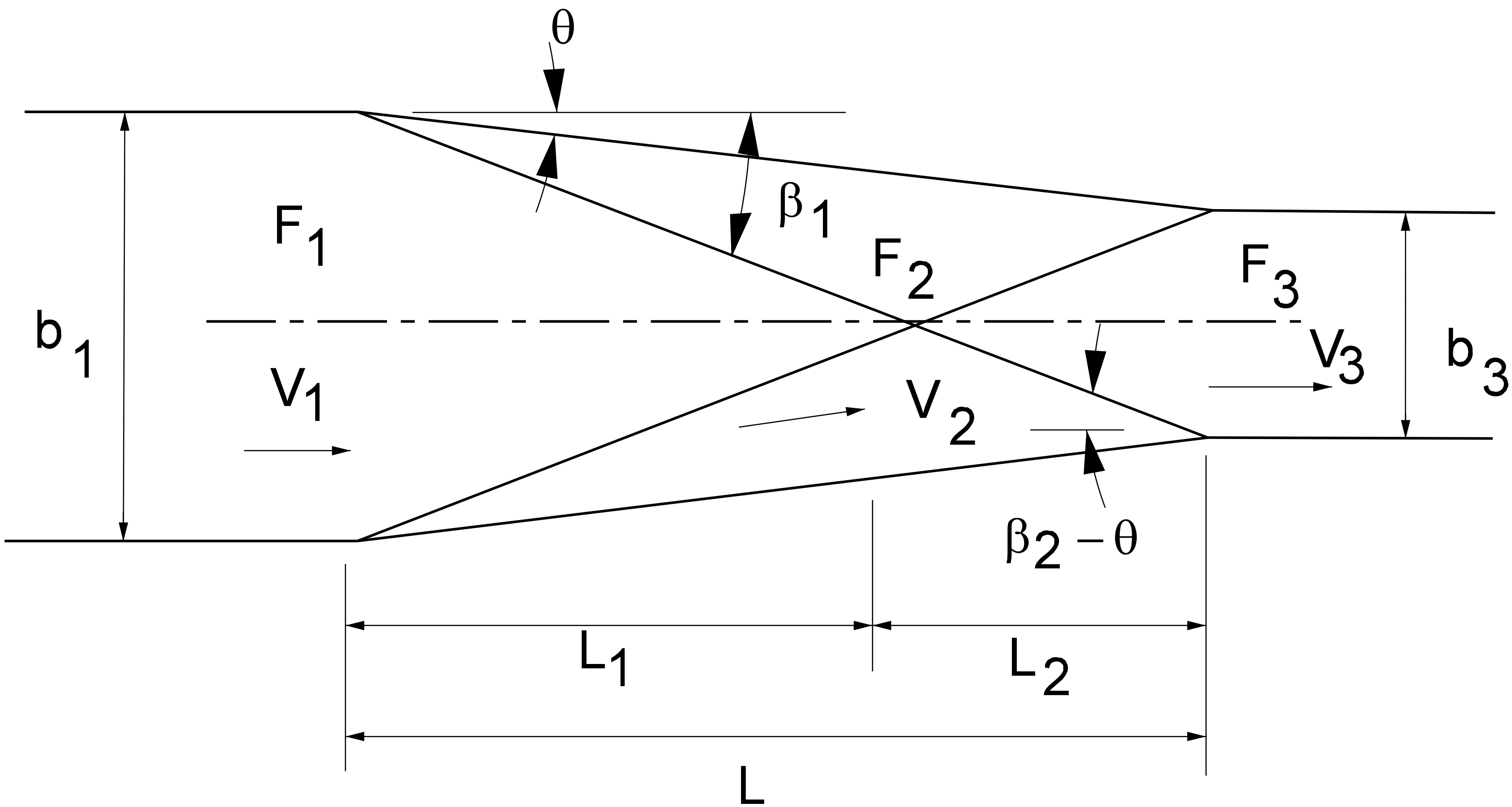
Figure 30-8D



Note: CRWS = Critical Water Surface Profiles for Q = 1000 ft^3/s

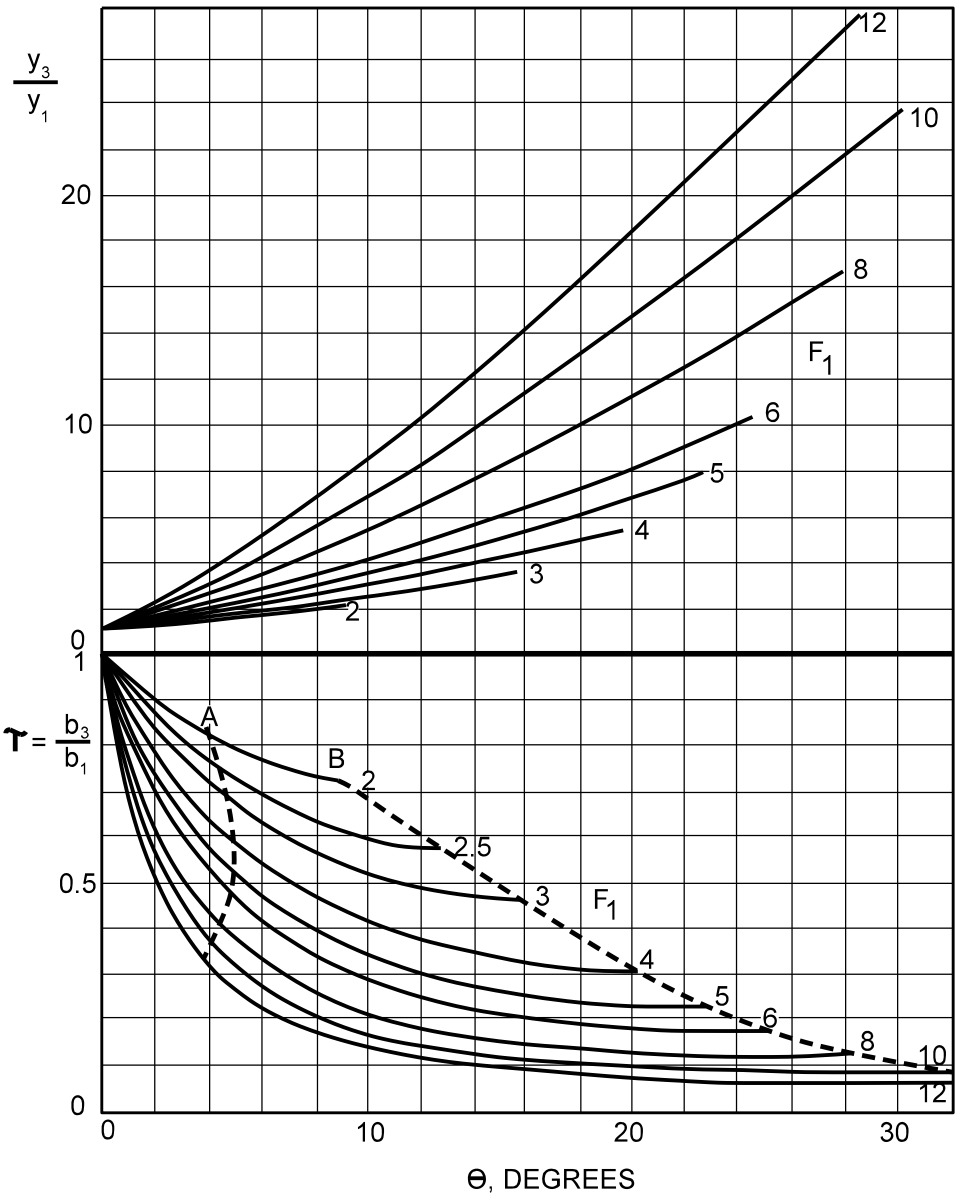
WATER SURFACE PROFILES FOR Q = 1000 ft^3/s AND VARIABLE TAILWATER (TW) ELEVATIONS

Figure 30-8E



DESIGN OF STRAIGHT-WALL CONTRACTIONS IN SUPERCRITICAL FLOW
Example 30-8.6

Figure 30-8F



CONTRACTION RATIO τ AND DEPTH RATIO y_3 / y_1 FOR SUPERCRITICAL FLOW IN CONTRACTION OF ANGLE Θ
EXAMPLE 30-8.6

Figure 30-8G

TABLE OF CONTENTS

LIST OF FIGURES

CHAPTER THIRTY-ONE

CULVERTS

31-1.0 INTRODUCTION

31-1.01 Definition of a Culvert

A culvert is defined as the following:

1. a structure (pipe, cast-in-place reinforced concrete, precast reinforced concrete, structural plate arch, etc.) which is designed hydraulically to take advantage of submergence to increase hydraulic capacity;

2. a structure used to convey surface runoff through an embankment;

3. a structure, as distinguished from a bridge, which is covered with embankment and is composed of structural material around the entire perimeter, although it can be supported on spread footings with the streambed serving as the bottom of the culvert; or

4. a structure with less than a 20 ft span length along the centerline of roadway between extreme ends of openings for multiple barrels. Figure 31-1A, Maximum Span Length for Culvert, provides schematics which define a culvert based on span length for various structural configurations. However, a structure designed hydraulically as a culvert is treated as discussed in this Chapter, regardless of span length.

In addition, the following apply to defining a culvert.

1. Mainline Culvert. A structure under a mainline roadway.

2. Public-Road-Approach Culvert. A structure under a public-road approach.

3. Drive Culvert. A structure under a drive or field entrance.

4. Concrete-Culvert Extension. New construction that extends an existing reinforced concrete slab-top, box-, or arch-culvert structure. Acceptable methods for constructing the extension include cast-in-place reinforced concrete or installation of precast reinforced-concrete box sections.

31-1.02 Purpose

This Chapter provides design procedures for the hydraulic design of a highway culvert which are based on FHWA Hydraulic Design Series Number 5 (HDS #5) *Hydraulic Design of Highway Culverts*. This Chapter also provides the following:

1. the results of culvert analysis using microcomputers which emphasizes the use of the HYDRAIN system and the HY8 culvert analysis software; and

2. a summary of the design philosophy included in the AASHTO *Highway Drainage Guidelines*, Chapter IV.

31-1.03 Definitions

The following are definitions of concepts to be considered in culvert design.

1. Backwater. The increase in water-surface elevation caused by the introduction of a culvert into an open channel or other open drainage system.

2. Critical Depth, d_C. 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.

3. Crown. The inside top of a culvert.

4. Flow Type. The USGS has established culvert-flow types which assist in determining the flow conditions at a particular culvert site. Diagrams of these flow types are provided in Section 31-5.0.

5. Free Outlet. A free outlet has a tailwater equal to or lower than critical depth. For a culvert having a free outlet, lowering of the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.

6. Improved End Treatment (Improved Inlet). An improved end treatment has an entrance geometry which decreases the flow contraction at the end treatment and thus increases the capacity of a culvert. The end treatment is referred to as side- or slope-tapered (walls or bottom tapered).

7. Invert. The flowline of the culvert (inside bottom).

8. Normal Flow. Normal flow occurs in a channel reach if the discharge, velocity, and depth of flow do not change throughout the reach. The water surface and channel bottom will be parallel.

9. Slope. The following applies.

   a. A steep slope occurs where critical depth is greater than normal depth.
   b. A mild slope occurs where critical depth is less than normal depth.

10. Submergence. The following applies.

    a. A submerged outlet occurs where the tailwater elevation is higher than the crown of the culvert.

    b. A submerged inlet occurs where the headwater is greater than 1.2*D*, where *D* is the culvert diameter or barrel height.

31-1.04 Symbols

To provide consistency within this Chapter and throughout this *Manual*, the symbols in Figure 31-1B, Culvert Symbols, will be used. These symbols have wide use in culvert publications.

31-2.0 POLICY

31-2.01 Definition

Policy is a set of goals that establish a definite course of action or method of action and that are selected to guide and determine present and future decisions. Policy is implemented through design criteria for making decisions (see Section 31-3.0).

31-2.02 Culvert Policy

The following policies are specific to a culvert.

1. Each culvert should be hydraulically designed. However, the minimum pipe size specified in Section 31-3.05(03) will sometimes control.

2. The design storm frequency/frequencies selected should be consistent with the criteria described in Section 31-3.03.

3. Survey information should include topographic features, channel characteristics, high-water information, existing-structure data, and other related site-specific information.

4. Culvert location in both plan and profile should approximate the alignment of the natural channel to avoid sediment build-up in the barrel.

5. A culvert should be designed to accommodate debris, or proper provisions should be made for debris maintenance.

6. A culvert should be located and designed to present a minimum hazard to traffic and pedestrians.

7. The detail of documentation for each culvert site should be commensurate with the risk and importance of the structure. Design data and calculations should be assembled in an orderly fashion and retained for future reference as provided for in Chapter Twenty-eight.

8. Where necessary as directed by INDOT, some means should be provided for personnel and equipment access to facilitate maintenance.

31-3.0 DESIGN CRITERIA

31-3.01 Definition

Design criteria are the standards by which a policy is implemented. They form the basis for the selection of the final design configuration. Listed below by categories are the design criteria which should be considered.

31-3.02 Site Criteria

The following apply.

1. Structure-Type Selection. A culvert is used at the locations as follows:

   a. where a bridge is not hydraulically required;
   b. where debris and ice are tolerable; and
   c. where its use will be more economical than a bridge.

   A bridge is used as follows:

a. where more economical than a culvert;
b. to avoid floodway encroachment;
c. to accommodate ice or large debris; and
d. where a culvert will generate excessive velocity, backwater, or other hydraulic deficiency.

2. Length and Slope. The culvert length and slope should be chosen to approximate existing topography and, as practical, the culvert invert should be aligned with the channel bottom and the skew angle of the stream. The roadway clear-zone requirements and the embankment geometry may dictate the culvert length. See Chapter Forty-nine.

3. Location in Plan. A severe or abrupt change in channel alignment upstream or downstream is not recommended. The following applies.

   a. A small culvert with no defined channel is placed normal to the roadway centerline.

   b. A large culvert perpetuating drainage in a defined channel should be skewed as necessary to minimize channel relocation and erosion.

   c. All utilities should be located before determining the final location of a culvert to minimize conflicts.

4. Location in Profile. The culvert profile will likely approximate the natural stream profile. Exceptions which must be approved by the Hydraulics Team can be considered as follows:

   a. arrest stream degradation by utilizing a drop-end treatment or broken-back culvert;
   b. improve hydraulic performance by utilizing a slope-tapered end treatment; or
   c. avoid conflicts with other utilities that are difficult to relocate such as sanitary sewers.

5. Debris Control. Debris control should be designed using Hydraulic Engineering Circular No. 9 *Debris-Control Structures*, and may be considered as follows:

   a. where experience or physical evidence indicates the watercourse will transport a heavy volume of controllable debris;

   b. for a culvert under a high fill; or

c. where clean-out access is limited. However, access must be available to clean out the debris-control device.

31-3.03 Design-Storm Frequency

See Figure 31-3A, Design Storm Frequency (Culvert).

31-3.04 Hydraulic Design Criteria

31-3.04(01) Allowable Headwater (*AHW*)

Allowable headwater is the depth of water that can be ponded at the upstream end of a culvert during the design flood. *AHW* will be limited by one or more of the following.

1. New Alignment. The maximum backwater (increase in headwater elevation over the sum of *TW* depth plus inlet invert elevation) should not exceed 1.5 in. The 1.5-in. maximum may be modified as follows:

    a. the backwater dissipates to 1.5 in. or less at the right-of-way-line; or

    b. the channel is sufficiently deep to contain the increased elevation without overtopping the banks.

    The Hydraulics Engineer must approve exceptions to the 1.5-in. backwater allowance.

2. Existing Conditions. The IDNR limits surcharge to 1 ft, in an urban or rural location. Existing conditions are defined as the water surface profile that results from only those encroachments that were constructed prior to December 31, 1973. Although IDNR policy will allow for a slight increase over existing conditions, INDOT will not. INDOT policy for a culvert replacement or rehabilitation is that the surcharge created by a proposed structure must be equal to or less than the existing surcharge, unless the existing surcharge is less than 1 ft. This will allow future widening of the structure. If the surcharge created by an existing structure is greater than 1 ft, the proposed surcharge for the culvert replacement or extension must not be greater than 1 ft above the natural-channel flood profile.

3. Right of Way. The ponding limit from the *AHW* cannot exceed the right-of-way limit for a structure on a new alignment.

4. Upstream Channel. The ponding limit from the *AHW* cannot exceed the banks of an upstream channel for a structure on a new alignment.

5. Other. Other constraints on *AHW* include the following:

   a. grades of adjacent drives;
   b. finished floor elevation of adjacent buildings or other improvements; and
   c. elevation of existing cropland or other property.

31-3.04(02) Roadway Serviceability

For the appropriate design storm, headwater caused by the proposed structure cannot exceed the following for the roadway.

1. If Q_{100} is the appropriate design storm, the resulting headwater elevation must be at least 2 ft below the edge-of-pavement elevation.

2. If the appropriate design storm frequency is less than Q_{100}, the resulting headwater elevation must not exceed the edge-of-pavement elevation.

31-3.04(03) Maximum Velocity

Each culvert requires outlet protection to prevent erosion. The protection used must be in accordance with the following.

1. Revetment riprap is required for a structure with an outlet velocity, V_o, of 6.5 ft/s or lower.

2. Class 1 riprap is required for a structure with 6.5 ft/s $\leq V_o <$ 10 ft/s.

3. Class 2 riprap is required for a structure with 10 ft/s $\leq V_o <$ 13 ft/s.

4. An energy dissipator is required if $V_o \geq$ 13 ft/s. See Chapter Thirty-four for the design of an energy dissipator.

If clear zone or other issues prohibit the use of the required riprap gradation, the designer must contact the Hydraulics Team for additional instructions.

31-3.04(04) Minimum Velocity

The minimum velocity in the culvert barrel should result in a tractive force, $\tau = \gamma dS$, greater than critical τ of the transported streambed material at a low-flow rate. The designer should use 3 ft/s if the streambed material size is not known.

31-3.04(05) Tailwater Relationship

For a channel, the designer should consider the following.

1. Evaluate the hydraulic conditions of the downstream channel to determine a tailwater depth (see Chapter Thirty).
2. Calculate backwater curves for sensitive locations or use a single cross-section analysis.
3. Use the critical depth and equivalent hydraulic grade line if the culvert outlet is operating with a free outfall.
4. Use the headwater elevation of a nearby downstream culvert if it is above the channel depth.

For a confluence or large body of water, the designer should use the high-water elevation that has the same frequency as the design flood if events are known to occur concurrently (statistically dependent) to determine the tailwater.

31-3.04(06) Storage (Temporary or Permanent)

Storage should not be considered in the hydraulic design of a culvert.

31-3.04(07) Culvert Sumping [Added Jan. 2011]

Sumping in a drainage structure consists of placing the structure invert elevation and scour protection at a specified depth below the waterway or stream flowline to satisfy the IDEM Water Quality Section 401 permit requirements. This sumping allows the natural movement of stream bed material through the structure. Sumping should be provided for each structure over one of the Waters of the United States.

1. Three-Sided Structure. The sump depth should be 18 in. for a stream bed of sand, 12 in. for a stream bed of other soil, or 3 in. for a stream bed of rock or till. The stream bed and

scour protection should be as shown on the INDOT *Standard Drawings*. A base slab should be used only if the geotechnical report identifies flowline-area soil that will not support riprap. No increase in structure size is required due to sumping. The sump area will not require backfill as part of the contract work, but will be allowed to fill in naturally over time.

2. Pipe or Box Structure. Such a structure should be sumped as shown on the INDOT *Standard Drawings* and Figure 31-3B(1).

   If the required sump exceeds 3 in., the structure diameter or rise may need to be increased by the sump value. The structure's design capacity should be checked to determine if such increase is required. If a pipe end section or riprap is required, these should be sumped to the same depth as the structure. The sump area of the structure and end section or riprap will not require backfill as part of the contract work, but will be allowed to fill in naturally over time.

Scour-protection limits should be shown on the plans. Quantities for geotextile and riprap, or a base slab, that are intended for scour protection, should be determined and identified as such in the Structure Data table for each applicable structure. Appropriate columns have been incorporated into the Structure Data table. Such quantities are not pay quantities, and therefore should not be incorporated into other pay quantities of geotextile, riprap, or concrete.

31-3.05 Culvert-Sizing Process

31-3.05(01) Priority System

The culvert-sizing process is performed in accordance with a priority system. The system consists of six trials where specific installations are considered prior to evaluating other structure types. The design priority system is as follows.

1. Trial 1. Single Circular-Pipe Installation.
2. Trial 2. Single Deformed-Pipe Installation.
3. Trial 3. Single Specialty-Structure Installation.
4. Trial 4. Multiple Circular-Pipes Installation.
5. Trial 5. Multiple Deformed-Pipes Installation.
6. Trial 6. Multiple Specialty-Structures Installation.

The principles of the priority system are summarized below.

1. A pipe structure is preferred to a specialty structure (e.g., precast reinforced-concrete box section, precast reinforced-concrete three-sided culvert, structural plate arch).

2. A circular pipe is preferred to a deformed pipe.

3. A single-cell installation is preferred to a multiple-cell installation.

Section 31-3.0 provides a decision flowchart for each of the six trials in the priority system, as the following figures.

31-3C	Culvert Design Process (Trial 1 - Single Circular Pipe)
31-3D	Culvert Design Process (Trial 2 - Single Deformed Pipe)
31-3E	Culvert Design Process (Trial 3 - Single Specialty Structure)
31-3F	Culvert Design Process (Trial 4 - Multiple Circular Pipes)
31-3G	Culvert Design Process (Trial 5 - Multiple Deformed Pipes)
31-3H	Culvert Design Process (Trial 6 - Multiple Specialty Structures)

31-3.05(02) Pipe-Culvert-Interior Designation

During the performance of Trials 1, 2, 4, or 5, specific pipe materials will not be considered. Instead, two generic designs are required. One design will size pipes with smooth interiors and the second will size pipes with corrugated interiors. The smooth-interior hydraulic design will be based on a Manning's *n* value of 0.012 and can use the nomographs or computer software used for sizing a reinforced concrete pipe. The corrugated hydraulic design is based on a Manning's *n* value of 0.024 and can utilize the nomographs or computer software used to size a corrugated-metal pipe. If the corrugated-pipe design indicates that structural-plate pipe is required, the Manning's *n* value must be in accordance with accepted engineering practice. See Figure 31-10A for typical values. The nomographs or computer software used to size a structural-plate pipe may be used to determine the required size for a larger structure.

The two hydraulic designs for an individual structure will be based on identical pipe lengths and invert elevations.

If separate hydraulic designs are performed for smooth and corrugated interior pipes, the following situations are possible.

1. Situation 1. The required smooth-interior and corrugated-interior pipe sizes are identical. The structure callout on the plans should include the required pipe size. No reference to an interior designation is made.

2. Situation 2. The required smooth-interior and corrugated-interior pipe sizes are different. The structure callout on the plans should indicate that the structure requires a smooth pipe of one size or a corrugated pipe of another.

3. Situation 3. An acceptable pipe size can be determined for one interior designation but not the other. If this occurs, the structure callout on the plans should indicate the required pipe size and interior designation.

4. Situation 4. No acceptable pipe size can be found for either interior designation. The designer must proceed to the next trial of the culvert-sizing process.

31-3.05(03) Minimum Culvert Size

If it is determined that a pipe is acceptable for a culvert structure, the proposed pipe size must be greater than or equal to that shown in Figure 31-3B, Minimum Pipe-Culvert Size.

31-3.05(04) Cover

In addition to the minimum-pipe-size requirement, cover is another factor that the designer must consider during the structure-sizing process. For a circular pipe structure, a minimum of 1 ft of cover (measured from the top of the pipe to the bottom of the asphalt or concrete pavement) must be provided. If the structure requires a deformed corrugated-interior pipe material, at least 1.5 ft of cover must be provided. The cover for a circular pipe structure should not exceed 100 ft. The cover for a deformed corrugated-interior pipe structure should not exceed 13 ft. If the pavement grade or structure invert elevations cannot be adjusted to satisfy the cover criteria discussed above, contact the Hydraulics Team for additional instructions.

31-3.05(05) Pipe-Extension Structure-Sizing Process

The sizing of a pipe-extension structure should be in accordance with the following.

1. Match Existing Pipe Size and Interior Designation. If practical, the pipe extension should be the same size and material as the existing pipe. However, at this stage, it is only necessary to identify the required interior designation for the extension.

2. Perform Appropriate Hydraulic Analysis. The appropriate hydraulic calculations must be performed to verify whether the extended structure satisfies the required design criteria. Because the structure's interior designation is known, it is only necessary to perform hydraulic calculations appropriate for that interior designation.

If the extended structure satisfies the required design criteria, the structure sizing process is complete. If the extended structure does not satisfy the required design criteria, the designer must reevaluate whether the existing structure can be replaced with a new structure. If it is not practical to replace the existing pipe because of construction method, traffic maintenance, or other constraints, contact the Hydraulics Team for further instructions.

31-3.05(06) Concrete-Culvert-Extension Sizing Process

If an existing cast-in-place reinforced-concrete slab-top culvert, box culvert, or arch culvert requires extension, the designer must decide whether the extension will be constructed using cast-in-place reinforced concrete or precast reinforced-concrete box sections. Once the extension method has been determined, the appropriate culvert-design criteria must be checked to verify that the extended structure satisfies the hydraulic requirements. If the analysis indicates that the extended structure does not satisfy the hydraulic requirements, the designer must reevaluate whether the existing structure can be removed and replaced with a new structure. If it is not possible to replace the existing culvert because of construction method, traffic maintenance, or other constraints, contact the Hydraulics Team for further instruction.

31-3.06 Other Culvert Features

31-3.06(01) Culvert Skew

The culvert skew should not exceed 45 deg as measured from a line perpendicular to the roadway centerline, without the approval of the Hydraulics Engineer.

31-3.06(02) Inlet or Outlet End Treatment

The culvert end-treatment type should be selected from the list shown below based on the given considerations and the end treatment coefficient, K_e. See Section 31-10.0 for the recommended value of K_e. Roadside safety should be considered in the selection and design. See Section 49-3.0 for a detailed discussion of INDOT practices for the safety treatment of a drainage structure.

The following discusses the types of culvert end treatments and their advantages and disadvantages.

1. Projecting Inlet or Outlet.
    a. Extends beyond the roadway embankment.
    b. Is susceptible to damage during roadway maintenance or an errant vehicle.
    c. Has a low construction cost.

d. Has poor hydraulic efficiency for thin material.

e. Should include anchoring the end treatment to strengthen the weak leading edge for a culvert of 42 in. diameter or larger. The anchorage should include a sufficient weight of concrete to resist buoyant forces (see the INDOT *Standard Drawings*).

f. May be strengthened by use of a concrete collar, if necessary.

Where a projecting inlet or outlet is within the roadside clear zone, the designer should consider the use of a grated box end section (GBES) or a safety metal end section (SMES). See Chapter Forty-nine for INDOT criteria on roadside clear zone. See the *INDOT Standard Drawings* for the GBES and SMES.

2. <u>Mitered End Treatment</u>.

   a. Is hydraulically more efficient than a thin edge projecting.

   b. Should be mitered to match the fill slope.

   c. Should include anchoring the end treatment to strengthen the weak leading edge for a culvert of 42 in. diameter or larger.

3. <u>Improved End Treatment</u>.

   a. Should be considered for a culvert which will operate in inlet control.

   b. Can increase the hydraulic performance of the culvert, but may also add to the total culvert cost. Therefore, it should only be used if economically justified.

4. <u>Pipe End Section</u>.

   a. Is available for either corrugated-metal or concrete pipe.

   b. Retards embankment erosion and incurs less damage from maintenance.

   c. May improve a projecting metal-pipe entrance by increasing hydraulic efficiency, reducing accident hazard, and improving the pipe entrance's appearance.

   d. Is hydraulically equivalent to a headwall, but can be equivalent to a beveled or side-tapered entrance if a flared, enclosed transition occurs before the barrel.

5. Wingwall.

   a. Is used to retain the roadway embankment to avoid a projecting culvert barrel.

   b. Is used where the side slopes of the channel are unstable.

   c. Is used where the culvert is skewed to the normal channel flow.

   d. Provides the best hydraulic efficiency if the flare angle is between 30°° and 60°.

   e. Should be provided for a precast-concrete drainage structure.

6. Apron.

   a. Is used to reduce scour from a high headwater depth or from approach velocity in the channel.

   b. Should extend at least one pipe diameter upstream.

   c. Should not protrude above the normal streambed elevation.

   d. May be constructed of riprap and an appropriate geotextile or concrete.

7. Cutoff Wall.

   a. Is used to prevent piping along the culvert barrel and undermining at the culvert end.

   b. Should be used for a culvert with headwalls.

   c. Should be of minimum 20 in. depth or as shown in the INDOT *Standard Drawings* or *Standard Specifications.*

8. Weep Hole. A weep hole should not be used.

31-3.06(03) Pipe-Length Determination

After the structure size and cover have been determined, the designer must determine the required length. The design length for a culvert structure should be rounded to the next higher 1.5 ft.

31-3.06(04) Buoyancy Protection

Pipe end sections or concrete anchors where a projecting end treatment or outlet is used, or other means of anchoring to provide buoyancy protection, should be considered for a flexible culvert. The seriousness of buoyancy depends on the steepness of the culvert slope, depth of the potential headwater (debris blockage may increase), flatness of the upstream fill slope, height of the fill, large culvert skew, or mitered ends. See the INDOT *Standard Drawings* and *Standard Specifications.*

31-3.06(05) Relief Opening

Where a culvert serving as a relief opening has its outlet set above the normal stream flow line, precautions should be made to prevent headcutting or erosion from undermining the culvert outlet.

31-3.06(06) Erosion and Sediment Control

Temporary measures should be shown on the construction plans. The measures may include the use of a silt box, brush silt barrier, filter cloth, temporary silt fence, or check dam. For more information, see Chapter Thirty-seven. The scour elevation to be shown on the Layout sheet is the scour elevation for Q_{500}.

31-4.0 DESIGN PHILOSOPHY

31-4.01 Overview

The design of a culvert system for a highway crossing a floodplain involves using information from other chapters in this Part (e.g., hydrology, channels). Each of these should be consulted as appropriate. The discussion below focuses on alternative analyses and design methods.

31-4.02 Alternative Analyses

A culvert alternative should be selected which satisfies the topography, design policies, and design criteria.

Alternatives should be analyzed for hydraulic equivalency, risk, and cost.

Select an alternative which best integrates engineering and economic and political considerations. The selected culvert should satisfy the applicable structural and hydraulic criteria, and should be based on the following:

1. construction and maintenance costs;
2. risk of failure or property damage;
3. roadside safety; and
4. land-use requirements.

31-4.03 Design Methods

The designer should choose either of the following:

1. to use a culvert or a storm drain; or

2. to use nomographs or computer software. The use of nomographs not based on HDS #5 is subject to the approval of the Hydraulics Engineer.

31-4.03(01) Structure Type

The following applies to the structure type.

1. Culvert. This is one of the following:

    a. a covered structure with both ends open;
    b. a type of structure designed using the procedures described in HDS #5; or
    c. a type of structure which may be circular, deformed, or specialty.

> ** *PRACTICE POINTER* **
>
> If twin box culverts are required, space should not be left between them.

2. Storm Drain. This is one of the following:

a. a covered structure with at least one end in a manhole inlet or catch basin and is usually a part of a system of pipes;

b. designed using HYDRA software included in HYDRAIN; or

c. designed using other computer models or by hand calculations.

See FHWA-SA-96-078 Urban Drainage Design Manual HEC #22, and Chapter Thirty-six and for more information.

3. Specialty Structure. A specialty structure can be used in either a culvert or storm-drain application. See Section 31-4.05 for more information on specialty structures.

31-4.03(02) Hydrology Methods

See Chapter Twenty-nine for detailed information on hydrology. A constant discharge is assumed for culvert design, is always the peak discharge, and will yield a conservatively-sized structure where temporary storage is available but not used.

31-4.03(03) Computational Methods

Nomographs require a trial-and-error solution which most often provides a reliable design, and require additional computations for tailwater, outlet velocity, hydrographs, routing, and roadway overtopping. They are available for many culvert sizes and shapes (see HDS #5).

31-4.03(04) Computer Software

HY-8 is the only computer program allowed for the hydraulic analysis of a culvert. The FHWA *Hydraulic Design of Highway Culverts* (HDS #5) is also acceptable. HDS #5 has also been updated and released as a CD ROM (the FHWA Hydraulics Library).

1. HYDRAIN Microcomputer System.

   a. Is recommended by AASHTO;
   b. includes HY8; and
   c. has a *User's Manual*.

2. HY8 (FHWA Culvert Analysis Software).

a. Is an interactive program written in Basic.

b. Uses the theoretical basis for the nomographs.

c. Can compute tailwater, improved end treatments, road overtopping, hydrographs, routing, and multiple independent barrels.

d. Develops and plots tailwater rating curves.

e. Develops and plots performance curves.

f. Is documented in the HYDRAIN *User's Manual* and HY8 *Applications Guide*.

31-4.04 Modifying or Replacing an Existing Culvert

If considering whether to modify or replace an existing culvert, the designer should first obtain a copy of the road log from the district office. The road log includes an inventory of the locations, sizes, and types of drainage structures located on each state highway. During the field data collection process, the size, location, and type of each culvert should be verified. Each structure should be inspected. See the FHWA *Culvert Inspection Manual* for information on structure inspection. See Figure 31-4A, Culvert Inspection Report Form. An editable version of this form may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/. If necessary, district operations or maintenance should be contacted to clean each plugged or partially-plugged structure so an adequate inspection can be performed. The district should be notified of changes that need to be made in the road log.

Once inspection of all culverts to be addressed has been completed, a list of those requiring modification or replacement should be provided to the district Office of Highway Management. The Office will slip-line or replace pipes of less than 36 in. diameter requiring excavation of less than 5 ft. See Section 31-4.04(02) Item 1 for an explanation of slip lining. This work should be done before the road work is contracted.

Rehabilitation or replacement should be considered as part of the project work for a culvert of 36 in. or greater diameter that has poor roadway geometry or that has a remaining life of less than the anticipated life of the proposed road work.

31-4.04(01) Determining Need for Culvert Modification or Replacement

Each culvert to be modified or replaced should be evaluated by an individual familiar with structure inspection.

1. General Considerations. The items to check include, but are not limited to, those as follows:

   a. structure alignment with the channel and the potential for erosion or scour;
   b. erosion of the approaches, particularly the areas behind the wingwalls;
   c. loss of fill material from beneath the roadway;
   d. local and contraction scour and general channel degradation;
   e. indications of foundation undermining and the potential for foundation undermining;
   f. structure settlement;
   g. timber decay;
   h. roadway geometry;
   i. hydraulic adequacy; and
   j. approach erosion.

2. Metal Pipe. Items to check when inspecting a metal pipe include, but are not limited to, those as follows:

   a. corrosion, including holes which could cause erosion of the surrounding backfill material; and
   b. excessive deformation.

   A metal pipe found to be in poor condition should be considered for slip lining or replacement.

3. Concrete Pipe. Items to check when inspecting a concrete pipe include, but are not limited to, those as follows:

   a. cracking, efflorescence, delaminating, or spalling of concrete;
   b. exposed or corroded concrete reinforcement;
   c. deterioration at widening joints;
   d. settlement or separation of joints allowing backfill material into the pipe; and
   e. deterioration of the structure.

   A concrete pipe found to be in poor condition should be considered for resetting, slip lining, or possibly replacement.

4. Jacking a Pipe. If a pipe is to be jacked under a road, space should be provided for a jacking pit. Temporary right of way will be required if there is not sufficient permanent right of way. The designer should discuss this issue at the preliminary field check.

31-4.04(02) Culvert Modification

A Structure Data Table should be included in the plans for drainage structures requiring modification. Detail sheets should be provided where required.

1. Slip Lining. Slip lining is a technique for rehabilitating a culvert of 18 in. diameter or larger. Slip lining is also suitable for use for a box- or arch-type culvert. It is completed by pushing or pulling sections of polyethylene pipe through the existing structure and filling the space between the polyethylene and existing structure with grout. The capacity of a structure can often be increased due to the low friction factor of the polyethylene liner. Factors to consider when deciding whether or not to slip line a structure are as follows:

    a. The structure barrel should be relatively straight and free from obstructions.
    b. The backfill around the structure should be free from large voids.
    c. There should be sufficient room to work from at least one end of the existing structure.
    d. The structure should be under at least 6.5 ft of fill or in a location where a road closure is undesirable or impossible.

2. Culvert Extension. A culvert that is structurally and hydraulically adequate, but of insufficient length, may be considered for an extension. Each culvert with a diameter of 50 in. or greater that will be extended 5 ft or more will require a geotechnical evaluation. See Section 31-3.05(05) for criteria regarding culvert extension.

3. Culvert End Treatment. See Section 31-3.06(02) for desirable design criteria regarding a culvert end treatment.

4. Headwalls and Anchors. Removal of headwalls or anchors damages the existing structure. As a minimum, 40 in. of new structure should be placed for each headwall removed.

    Each protruding headwall which is not in accordance with the obstruction-free-zone criteria should be considered for removal or modification. A headwall which is shielded from impact by guardrail should not be removed unless it is located within clearance range of the guardrail as shown in Figure 49-4A.

31-4.04(03) Culvert Replacement

Each culvert with a diameter of 50 in. or greater that is to be replaced will require a geotechnical report and a hydraulic analysis. If a legal drain is involved, the county surveyor should be contacted for the replacement-structure parameters. County soils survey and county drainage maps are available. The USGS 7.5-min series topography maps provide information regarding drainage patterns at a large scale. The topography maps show rivers, creeks, and streams which indicate the drainage pattern of the basin as a whole.

The designer should provide the flow-line elevation for the structure to be replaced. The established temporary benchmarks should be shown on the detail sheet. Cross sections should be provided where required for each culvert replacement or new installation.

If a culvert is already in the INSTIP program for replacement, the designer should attempt to incorporate the replacement into the project work.

31-4.04(04) Backfill Materials

See Section 17-2.09 for culvert-backfill requirements.

<u>31-4.05 Specialty Structures</u>

Specialty structures include those described below.

31-4.05(01) Precast-Concrete Box Culvert

A precast-concrete box culvert may be recommended by the Hydraulics Team. A box structure is considered oversize if its clear-span length is more than 12’-0”. The recommended layout method for a box culvert is to extend it to the point where the roadway sideslope intercepts the stream flowline. The sideslope at the end of a box culvert should be protected with guardrail or be located beyond the clear zone.

31-4.05(02) Precast-Concrete Oversize Box Structure

A precast-concrete oversize box structure may be recommended by the Hydraulics Team. A box structure is considered oversize if its clear-span length is more than 12’-0”, but not more than 20’-0”. Product information is available from local suppliers.

The hydraulic recommendations letter will indicate if a three-sided structure with a base slab is an acceptable alternate to an oversize box structure. The designer should consult with the Hydraulics Team for guidance as to whether the two structure types are interchangeable for the specific site. A cost comparison should be used in making the final structure selection.

If the distance between the top of the structure and the top of the pavement section, is less than 2 ft as measured at the edge of travel lane, all top slab reinforcement in a box structure, or all reinforcement in a three-sided structure, should be epoxy coated. A note should be placed in the Structure Data Table's comments column indicating this.

An oversize box culvert should be laid out so that the total structure length is a multiple of the box-segment length for the given box size. It is not necessary to add a tolerance for the joints between segments in determining the total structure length. The available segment masses (weights) and lengths are shown in Figure 31-4B.

31-4.05(03) Wingwalls and Headwalls for Precast-Concrete Structure

Wingwalls and headwalls for a precast-concrete structure will be required. Such wingwalls and headwalls may b precast or cast in place.

The information to be shown on the plans is as follows:

1. a plan view showing the total length of the structure, skew angle, distance from roadway centerline to each end of structure, and the flare angle of all wingwalls;

2. an elevation view of the end of the structure including wingwalls and headwall. The span and rise of the structure should be dimensioned. The heights of the headwalls should be shown;

3. wingwalls labeled A through D with a table showing all dimensions and elevations for each wingwall;

4. a table summarizing the wingwall areas required;

5. a conceptual drawing showing a typical section through each wingwall that shows the approximate footing configuration. Footing dimensions should not be shown. The contractor is responsible for the footing design; and

6. the allowable soil bearing pressure. A table should be included on the plans listing the soil parameters for wingwall design as follows:

a. angle of friction between wingwall footing and foundation soil, δ;
b. angle of internal friction of the foundation soil, φ;
c. ultimate cohesion of foundation soil, C; and
d. ultimate adhesion between foundation soil and concrete, C_A.

These soil parameters will be provided in the geotechnical report for the structure. If the geotechnical report is lacking this information, it should be requested from the Production Management Division's Office of Geotechnical Services.

The headwalls' quantities will be included in the structure quantities.

If a project has at least one precast-concrete box structure, and at least one precast-concrete three-sided drainage structure, each with wingwalls, the wingwalls' quantities for both types of structures may be combined.

31-4.05(04) Plans Details, and Design Computations and Shop Drawings

Only the conceptual layout for a precast-concrete 3-sided or 4-sided structure, or precast wingwalls and headwalls, should be shown on the plans. Once the work is under contract, the fabricator will design and detail the structure. For each 3-sided structure, or for a 4-sided structure of greater than 12'-0" span, the fabricator will provide design computations and shop drawings which are to be checked by, and are subject to the approval of, the designer.

The contractor may choose to substitute a three-sided structure as a cost-reduction incentive. Details for a hydraulically-equivalent three-sided structure should not be shown on the plans.

31-4.05(05) Structure-Size Increments

For structure box sections and structure-extension box sections, the size increments are as follows.

1. <u>Structure of 12'-0" Span or Less</u>. Span and rise range from a minimum of 3 ft through a maximum of 12 ft, in 1-ft increments. The rise must be less than or equal to the span.

2. <u>Oversize Structure</u>. Span is 14, 16, 18, or 20 ft. Rise is 4, 5, 6, 7, or 8 ft.

31-5.0 DESIGN EQUATIONS

31-5.01 General

An exact theoretical analysis of culvert flow is complex due the requirements as follows:

1. analyzing non-uniform flow with regions of both gradually varying and rapidly varying flow;

2. determining how the flow type changes as the flow rate and tailwater elevations change;

3. applying backwater and drawdown calculations, energy, and momentum balance;

4. applying the results of hydraulic model studies; and

5. determining if hydraulic jumps occur and if they are inside or downstream of the culvert barrel.

31-5.02 Approach

The procedure considers the following.

1. 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 end-treatment geometry, the barrel characteristics, and the tailwater.

2. 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.

3. Culvert Sizing. The culvert sizing process must satisfy the criteria as follows:

    a. allowable headwater elevation at Q_{100};

    b. roadway serviceability for storm of specific magnitude, depending on functional classification; and

    c. maximum pipe outlet velocity or energy dissipator design.

4. Computer Software. The HY8 hydraulics computer software and FHWA Hydraulics Library CD design methods are acceptable for structure sizing.

31-5.03 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.

31-5.03(01) Headwater Factors

These include the following.

1. Headwater depth is measured from the inlet invert of the inlet control section to the surface of the upstream pool.

2. Inlet area is the cross-sectional area of the face of the culvert. The inlet face area is the same as the barrel area.

3. Inlet-edge configuration describes the entrance type. Inlet-edge configurations include thin-edge projecting, mitered, square edge in a headwall, and beveled edge. See Section 31-10.0 for the edge configuration of an INDOT culvert inlet.

4. Inlet shape is the same as the shape of the culvert barrel. Shapes include rectangular, circular, elliptical, and arch. Check for an additional control section, if different than the barrel.

31-5.03(02) Hydraulics

Three regions of flow are shown in Figure 31-5A, Unsubmerged, Submerged, and Transition. These are described as follows.

1. 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 through model tests of the weir geometry or by measuring prototype discharges. These tests are then used to develop equations. HDS #5 Appendix A includes the equations which were developed from model test data. See Figure 31-5B, Flow Type I.

2. Submerged. For headwater 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. HDS #5 Appendix A includes flow equations which were developed from model test data. See Figure 31-5C, Flow Type V.

3. 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.

31-5.03(03) 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. In the inlet control nomographs, *HW* is measured to the total upstream energy grade line including the approach velocity head.

31-5.04 Outlet Control

Outlet control has depths and velocities which 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 the culvert in outlet control, as follows.

1. Interior Designation. The pipe-culvert interior designation (i.e., the barrel roughness) will be either smooth or corrugated. See Section 31-3.05(02) for INDOT practice. The roughness is represented by a hydraulic resistance coefficient such as the Manning's *n* value. Manning's *n* values are provided in Section 31-10.0.

2. Barrel Area. Barrel area is measured perpendicular to the flow.

3. Barrel Length. Barrel length is the total culvert length from the entrance invert to the exit invert. Because the design height of the barrel and the slope influence the actual length, an approximation of barrel length is necessary to begin the design process.

4. Barrel Slope. Barrel slope is the actual slope of the culvert barrel and is often the same as the natural stream slope. However, if the culvert inlet or outlet is raised or lowered, the barrel slope is different from the stream slope.

31-5.04(01) Tailwater Elevation

Tailwater is based on the downstream water-surface elevation. Backwater calculations from a downstream control, a normal depth approximation, flood insurance studies, or IDNR historic flood profiles are used to define the tailwater elevation (see Section 31-3.03).

31-5.04(02) 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. See Figure 31-5D, Flow Type IV.

The following equations apply.

1. Losses. $H_L = H_E + H_f + H_V + H_b + H_j + H_g$ (Equation 31-5.1)

   Where: H_L = total energy loss, ft
   H_E = entrance loss, ft
   H_f = friction losses, ft
   H_V = exit loss, or 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)

2. Velocity. $V = Q/A$ (Equation 31-5.2)

   Where: V = average barrel velocity, ft/s
   Q = flow rate, ft^3/s
   A = cross sectional area of flow with the barrel full, ft^2

3. Velocity head. $H_V = \frac{V^2}{2g}$ (Equation 31-5.3)
   Where: g = acceleration due to gravity, 32.2 ft/s^2

4. Entrance loss. $H_E = \frac{K_E V^2}{2g}$ (Equation 31-5.4)
   Where: K_E = entrance loss coefficient; see Section 31-10.0.

5. Friction loss. $H_f = \left(\frac{29Ln^2}{R^{1.33}}\right)\left(\frac{V^2}{2g}\right)$ (Equation 31-5.5)
   Where L = length of the culvert barrel, ft
   n = Manning's roughness coefficient (see Section 31-10.0)

R = hydraulic radius of the full culvert barrel = A/P, ft
P = wetted perimeter of the barrel, ft

6. Exit loss. $H_O = \frac{V^2 - V_d^2}{2g}$ (Equation 31-5.6)

Where: V_d = channel velocity downstream of the culvert, ft/s (usually neglected; see Equation 31-5.5). If neglected,

$$H_O = H_V = \frac{V^2}{2g}$$ (Equation 31-5.7)

7. Barrel losses. $H = H_E + H_O + H_f$ (Equation 31-5.8)

$$H = \left[1 + K_E + \left(\frac{19.63 L n^2}{R^{1.33}}\right)\right]\left(\frac{V^2}{2g}\right)$$

8. Energy Grade Line. The energy grade line represents the total energy at a point along the culvert barrel. Equating the total energy at Sections 1 and 2, upstream and downstream of the culvert barrel in Figure 31-5D, Flow Type IV, the resulting relationship is as follows:

$$HW_O + \frac{V_U^2}{2g} = TW + \frac{V_d^2}{2g} + H_L$$ (Equation 31-5.9)

Where: HW_O = headwater depth above the outlet invert, ft
V_U = approach velocity, ft/s
TW = tailwater depth above the outlet invert, ft
V_d = downstream velocity, ft/s
H_L = sum of all losses (Equation 31-5.1)

9. 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.

31-5.04(03) Nomographs

The following describes the assumptions for the culvert nomographs in the FHWA Hydraulics Library CD.

1. Full Flow. The nomographs were developed assuming that the culvert barrel is flowing full and that the following apply.

a. $TW \geq D$, Flow Type IV (see Figure 31-5D), or $d_C \geq D$, Flow Type VI (see Figure 31-5E).

b. 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.

c. V_d is small and its velocity head can be neglected.

d. Equation 31-5.9 becomes the following:

$$HW = TW + H - S_O L \qquad \text{(Equation 31-5.10)}$$

Where: HW = depth from the inlet invert to the energy grade line, ft
H = value read from the nomographs (Equation 31-5.8), ft
$S_O L$ = drop from inlet to outlet invert, ft

2. Partly-Full Flow. Equations 31-5.1 through 31-5.10 were developed for full-barrel flow. The equations also apply to a flow situation which is effectively a full-flow condition, if $TW < d_C$; see Figure 31-5F, Flow Type VII. Backwater calculations may be required beginning at the downstream water surface and proceeding upstream. If the depth intersects the top of the barrel, full flow extends from that point upstream to the culvert entrance.

3. Partly-Full Flow, Approximate Method. Based on backwater calculations performed by FHWA, it has been determined that the hydraulic grade line pierces the plane of the culvert outlet at a point halfway between critical depth and the top of the barrel, or $(d_C + D)/2$ above the outlet invert. TW should be used if it is higher than $(d_C + D)/2$. The following equation should be used.

$$HW = h_O + H - S_O L \qquad \text{(Equation 31-5.11)}$$

Where: h_O = the larger of TW or $(d_C + D)/2$, ft

Adequate results are obtainable down to $HW = 0.75D$. For a lower headwater, backwater calculations are required. See Figure 31-5G if $TW < d_C$, or Figure 31-5H if $TW \geq d_C$.

31-5.05 Outlet Velocity

Culvert outlet velocity should be calculated to determine the extent of erosion protection required at the culvert exit. A culvert affects an outlet velocity which is higher than the natural stream velocity. See Section 31-3.0 for the INDOT policy on outlet protection.

31-5.05(01) Inlet Control

The velocity is calculated from Equation 31-5.2 after determining the outlet depth. Either of the following methods may be used to determine the outlet depth.

1. Calculate the water surface profile through the culvert. Begin the computation at d_C at the entrance and proceed downstream to the exit. Determine the depth and flow area at the exit.

2. Assume normal depth and velocity. This approximation may be used because the water surface profile converges towards normal depth if the culvert is of adequate length. This outlet velocity may be slightly higher than the actual velocity at the outlet. Normal depth may be obtained from design aids described in publications such as HDS #3.

31-5.05(02) 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. The following applies.

1. Critical depth is used where the tailwater is less than critical depth.

2. Tailwater depth is used where tailwater is greater than critical depth but below the top of the barrel.

3. The total barrel area is used where the tailwater exceeds the top of the barrel.

31-5.06 Roadway Overtopping

A roadway is designed to avoid overtopping during the appropriate design storm given for the road-serviceability requirement. However, for a storm that exceeds the road-serviceability design storm, it is necessary to calculate *HW* elevations and velocities.

Roadway overtopping will begin once the headwater rises to the elevation of the roadway. The overtopping will 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 a roadway embankment are shown in HDS #1 *Hydraulics of Bridge Waterways* and in the documentation of HY-7, Bridge Waterways Analysis Model.

Equation 31-5.12 defines roadway overtopping as follows:

$$Q_r = C_d L HW_r^{1.5} \quad \text{(Equation 31-5.12)}$$

Where:
- Q_r = overtopping flow rate, ft^3/s
- 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

The length is difficult to determine where the crest is defined by a roadway sag vertical curve. Either of the following may be used.

1. Subdivide the length into a series of segments. The flow over each segment is calculated for a given headwater. The flows for each segment are then added together to determine the total flow.

2. 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 is calculated for a given upstream water-surface elevation using Equation 31-5.9. The following applies.

1. Roadway overflow plus culvert flow must equal total design flow.

2. A trial-and-error process is necessary to determine the flow passing through the culvert and the amount flowing across the roadway.

3. Performance curves for the culvert and the road overflow may be summed to yield an overall performance.

31-5.07 Performance Curves

Performance curves are plots of flow rate versus headwater depth or elevation, velocity, or outlet scour. The culvert performance curve is composed of the controlling portions of the individual performance curves for each of the following control sections (see Figure 31-5 I, Overall Performance Curve), as follows.

1. Inlet. The inlet performance curve is developed using the inlet control nomographs.

2. Outlet. The outlet performance curve is developed using Equations 31-5.1 through 31-5.10, the outlet control nomographs, or backwater calculations.

3. Roadway. A roadway performance curve is developed using Equation 31-5.12.

4. Overall. An 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.

   a. Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. These flow rates should occur above and below the design discharge and include the entire flow range of interest. Both inlet and outlet control headwaters should be calculated.

   b. Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.

   c. If the culvert headwater elevation exceeds 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 31-5.12 to calculate flow rates across the roadway.

   d. 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 31-5 I, Overall Performance Curve.

31-6.0 DESIGN PROCEDURES

The following design procedure provides a convenient and organized method for designing a culvert for a constant discharge, considering inlet and outlet control. The procedure does not address the affect of storage which is discussed in Chapter Thirty-five and Section 31-9.0. Storage will not be considered in the design of a structure which is not part of a detention facility. The following also applies.

1. The designer should be familiar with all equations in Section 31-5.0 before using these procedures.

2. Following the design method without an understanding of culvert hydraulics can result in an inadequate, unsafe, or costly structure.

3. The computation form has been provided in Section 31-10.0 to guide the user. It includes 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.

4. Step 1: Assemble Site Data And Project File.

   a. See Section 28-5.0. The minimum required data are as follows:

      (1) USGS, site and location maps;
      (2) embankment cross section;
      (3) roadway profile;
      (4) channel cross section at outlet of proposed structure;
      (5) photographs;
      (6) survey (sediment, debris); and
      (7) design data for nearby structures for which data is readily available.

   b. Studies and regulatory requirements by other agencies including the following:

      (1) small dam: NRCS, USACOE;
      (2) canal: NRCS, USACOE;
      (3) floodplain: NRCS, USACOE, FEMA, USGS, NOAA, IDNR; and
      (4) storm drain: local or private, including drains regulated by county drainage board.

   c. Design criteria.
      (1) review Section 31-3.0 for applicable criteria, and
      (2) prepare analysis.

5. Step 2: Determine Hydrology.

   a. See Chapter Twenty-nine.
   b. Minimum required data are drainage area map and a discharge.
   c. Determine magnitude of each required design storm.

6. Step 3: Analyze Downstream Channel.

   a. See Chapter Thirty.
   b. Minimum required data are cross section and slope of channel, and the rating curve for the channel.
   c. Perform analysis for each required design-storm magnitude.

7. Step 4: Summarize Data On Design Form.

   a. See Chart in Section 31-10.0.
   b. Data from Steps 1-3.
   c. Perform analysis for each required design-storm magnitude.

8. Step 5: Select Design Discharge, Q_d.

   a. See Section 31-3.0 and Chapter Twenty-nine.
   b. Determine flood frequency from criteria.
   c. Determine *Q*.

9. Step 6: Perform Structure-Sizing Process.

   a. See Section 31-3.0.
   b. Steps 7, 8 and 9 must be performed for each structure type analyzed in Trials 1 through 6 for the design-storm magnitudes appropriate for allowable headwater and roadway serviceability.
   c. Continue with trials until hydraulic design is complete.
   d. Select acceptable structure size, type, and pipe-interior designation, if applicable, and select the end treatment.

10. Step 7: Determine Inlet Control Headwater Depth, HW_i. Use the inlet-control nomograph (FHWA Hydraulics Library CD). A plastic sheet with a matte finish can be used to mark on so that the nomographs can be preserved.):

    a. Locate the size or height on the scale.

    b. Locate the discharge.

       (1) For a circular shape, use discharge.
       (2) For a box shape, use *Q* per foot of width.

    c. Locate *HW*/*D* ratio.

       (1) Use a straightedge.
       (2) Extend a straight line from the culvert size through the flow rate.
       (3) Mark the first *HW*/*D* scale. Extend a horizontal line to the desired scale, read *HW*/*D*, and identify on Chart in Section 31-10.0.

    d. Calculate headwater depth, *HWi*.

(1) Multiply HW/D by D to obtain HW to energy gradeline.
(2) Neglecting the approach velocity, $HWi = HW$.
(3) Including the approach velocity, $HWi = HW$ – (approach velocity head).

11. 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 the appropriate chart in HDS #5.

(1) Locate flow rate and read d_C.

(2) d_C cannot exceed D.

(3) If $d_C > 0.9D$, consult Handbook of Hydraulics (King and Brater) for a more accurate d_C, if needed, because curves are truncated where they converge.

c. Calculate $(d_C + D)/2$.

d. Determine h_O. It equals the larger of TW or $(d_C + D)/2$.

e. Determine entrance loss coefficient, K_E, from Section 31-10.0.

f. Determine losses through the culvert barrel, H.

(1) Use nomograph (FHWA Hydraulics Library CD) or Equation 31-5.8 or 31-5.9 if outside the range.

(2) Locate appropriate K_E scale.

(3) Locate culvert length, L or L_1. Use L if Manning's n matches that of the culvert. Use L_1 to adjust for a different culvert n value.

$$L_1 = \frac{2L_{n1}}{n} \qquad \text{(Equation 31-5.13)}$$

Where:

L_1	=	adjusted culvert length, ft
L	=	actual culvert length, ft
n_1	=	desired Manning's n value
n	=	Manning's n value from chart

(4) Mark point on turning line. Use a straightedge and connect the size with the length.

(5) Read H. Use a straightedge, connect Q and turning point, and read H on the Head Loss scale.

g. Calculate outlet control headwater, HW_{oi}:

(1) Use Equation 31-5.14, if V_U and V_d are neglected, as follows:

$$HW_{oi} = H + h_O - SO_L \qquad \text{(Equation 31-5.14)}$$

(2) Use Equations 31-5.1, 31-5.6, and 31-5.9 to include V_U and V_d.

(3) If HW_{oi} is less than 1.2D, and the outlet controls, the following apply.

(a) The barrel may flow partly full.

(b) The approximate method of using the greater of tailwater or $(d_C + D)/2$ may not be applicable.

(c) backwater calculations should be used to check the result.

(d) If the headwater depth falls below 0.75D, the approximate nomograph method should not be used.

12. Step 9: Determine Controlling Headwater, HW_C.

a. Compare HW_i and HW_{oi}, and use the higher value.

b. Compare HW_C with allowable HW and adjust culvert size if necessary.

13. Step 10: Compute Discharge Over The Roadway, Q_r.

a. Choose Q_t and thereby establish a TW.

b. Assume an upstream depth over the roadway, HW_r, calculate the length of roadway crest, L, and calculate the overtopping flow rate using Equation 31-5.15 as follows:

$$Q_r = C_d L (HW_r)^{1.5} \qquad \text{(Equation 31-5.15)}$$

See Section 31-5.06.

c. Calculate the flow in the culvert by using Equations 31-5.8 and 31-5.10 solving for V and then Q_d.

14. Step 11: Calculate Outlet Velocity, V_O, and Depth, d_n. If inlet is the controlling headwater using the design-storm magnitude appropriate for the velocity, calculate the following:

a. flow depth at culvert exit using normal depth or water-surface profile;

b. flow area, A; and

c. exit velocity, $V_O = Q/A$.

If outlet is the controlling headwater using the design-storm magnitude appropriate for the velocity, calculate the following:

a. flow depth at culvert exit;

(1) use d_C if it is greater than TW,
(2) use TW if $d_C < TW < D$, *or*
(3) use D if it is less than TW.

b. flow area, A; and

c. exit velocity, $V_O = Q/A$.

15. Step 12: Review Results. Compare alternative design with constraints and assumptions. If one or more of the following is exceeded, repeat Steps 5 through 12.

a. The barrel must have adequate cover; see Section 31-3.05(04).
b. The length should be close to the approximate length.
c. The allowable backwater should not be exceeded.
d. The roadway serviceability must be satisfied.
e. The outlet velocity should not be excessive; see Section 31-3.0.

16. Step 13: Related Designs. Consider the following options (see Sections 31-5.04 and 31-5.05).

a. Improved end treatments if culvert is in inlet control and has limited available headwater.

b. Energy dissipator if $V_O \geq 13$ ft/s (see Section 31-3.0 and Chapter Thirty-four).

c. Sediment-control storage for site with sediment concerns such as alluvial fans (see Chapter Thirty-seven).

17. Step 14: Documentation.

a. See Chapter Twenty-eight.
b. Prepare report and file with background information.

31-7.0 NOMOGRAPH DESIGN

The following example problem follows the Design Procedure Steps described in Section 31-6.0.

1. Step 1: Assemble Site Data And Project File.

a. Site survey project file should include the following:

(1) USGS site and location maps;
(2) roadway profile;
(3) embankment cross section;
(4) if two-lane roadway, ADT > 3000; and
(5) new alignment.

See Figure 31-7.1 for site data.

b. Survey notes should indicate no sediment or debris problems and no nearby structures.

c. Studies by other agencies: none.

d. Environmental risk assessment shows the following:

(1) no buildings near floodplain;
(2) no sensitive floodplain values;
(3) no FEMA involvement; and
(4) convenient detours exist.

e. Design criteria.

(1) *HW*: Q_{100}. Maximum backwater is 1.5 in.

(2) *RS*: Q_{100}. Headwater elevation must be 2 ft below the edge-of-pavement elevation.

(3) *VEL*: Q_{50}. Maximum velocity is 13 ft/s without energy dissipator. See Section 31-3.0 for INDOT criteria on outlet protection.

2. Step 2: Determine Hydrology. USGS regression equations yield the following:

Q_{50} = 134 ft^3/s, and Q_{100} = 148 ft^3/s

3. Step 3: Analyze Downstream Channel. See Figure 31-7.2 for cross section of channel (slope = 0.001).

Point	Station (ft)	Elevation (ft)
1	12.3	194.0
2	22.3	193.0
3	28.3	192.8
4	31.0	190.4
5	43.0	190.4
6	45.7	192.8
7	62.0	197.6

The rating curve for the channel calculated with normal depth yields the following:

Q (ft^3/s)	*TW* (ft)	*V* (ft/s)
44.5	1.73	1.97
89.0	3.13	2.90
133.5	2.57	2.47
148.3	3.30	3.00
178.0	3.60	3.20

4. Step 4: Summarize Data On Design Form. See Figure 31-7A, Chart 17 and Performance Curve, for design example.

5. Step 5: Select Design Discharge.

Q_d = Q_{100} = 148 ft^3/s for *HW* and *RS*
Q_{50} = 134 ft^3/s for outlet velocity

6. Step 6: Perform Structure-Sizing Process.

a. Start with Trial 1, Singular Circular Pipe, and proceed through the trials until the hydraulic design is complete.

b. Review site conditions to determine what limitations may be applicable.

Assume shoulder PI at approximate elevation of 198.3 ft.
Assume pavement thickness of -0.83 ft.
Assume minimum cover of -1.00 ft.
Pipe thickness is -0.60 ft.
Inlet elevation is -190.7 ft.
Maximum allowable rise is 5.17 ft.

c. Try a circular pipe with diameter = 60 in.

d. Assume an end section which satisfies the fill-slope requirement for both RCP and CMP. K_E = 0.5 from Figure 31-10B, Entrance Loss Coefficients (Outlet Control, Full or Partly Full).

e. TW = 3.3 ft. $AHW = El._i + TW + 0.13 = 190.7 + 3.3 + 0.13 = 194.1$ ft

f. With a very flat channel slope of 0.001, the culvert will likely operate under outlet control. However, for ease of calculations, Step 7 will determine inlet control for the largest permissible pipe and, if a trial size is too small, Step 7 will indicate continuing on to the next trial.

7. <u>Step 7: Determine Inlet Control Headwater Depth, HW_i</u>. Use inlet-control nomograph.

D = 5 ft, K_E = 0.5
Q = 148 ft^3/s
From nomograph, HW/D = 1.16
HW_i = 1.16 (5) = 5.8 ft
$El._{h1} = HW_i + El._i = 5.8 + 190.7 = 196.5$ ft > 194.1 ft, therefore No Good.
Go to Trial 2, Single Deformed Pipe.

a. Maximum rise = 60 in.
Maximum allowable corrugated-metal pipe-arch size = 72 in. x 48 in.

b. From nomograph, HW/D = 1.55

c. HW_i = 1.55 (4) = 6.2 ft

d. $El._{h1} = 6.2 + 190.7 = 196.9$ ft > 194.1 ft, therefore No Good.

e. Try reinforced-concrete deformed pipe, 100 in. x 63 in.

f. From nomograph, $HW/D = 0.75$. $HW_i = 0.75\ (5.27) = 3.95$

g El.$_{h2}$ = 3.95 + 190.7 = 194.7 ft > 194.1 ft, therefore No Good.

Go to Trial 3, Single Specialty Structure.

With a maximum of a 1.5-in. increase in backwater allowed and a flat slope of 0.001, a wide shallow box will be necessary. Therefore, try a reinforced-concrete box, 132 in. x 48 in., with a wingwall 30 deg to 75 deg to barrel. $K_E = 0.5$ from Figure 31-10B.

Use inlet-control nomograph.

$D = 4$ ft
$Q/B = 148.3/11 = 13.48$ ft^3/s per foot
$HW/D = 0.71$
$HW_i = 0.71\ (4) = 2.84$ ft
El$_i$ = 190.7 + 2.84 = 193.54 ft < 194.1 ft, therefore OK

Go to outlet control.

8. Step 8: Determine Outlet-Control Headwater Depth at Inlet, HW_{oi}.

$TW = 3.3$ ft for $Q = 148.3$ ft^3/s
$d_C = 1.73$ ft from Critical Depth, Rectangular Section chart
$(d_C + D)/2 = (0.52 + 1.2)/2 = 0.86$
h_O = the larger of TW or $(d_C + D)/2 = 3.3$ ft
$K_E = 0.5$
Determine H from Chart 15, Outlet-Control Nomograph.

a. K_E, scale = 0.5
b. Culvert length, $L = 300$ ft
c. $n = 0.012$
d. Area = 11 x 4 = 44 ft^2
e. $H = 0.4$ ft
f. El.$_{hO}$ = El$_O$ + H + h_O = 190.3 + 0.4 + 3.3 = 194 ft < 194.1 ft

9. Step 9: Determine Controlling Headwater Elevation.

a. El.$_{hO}$ = 194.1 ft > El.$_{hC}$ = 193.5 ft

b. The culvert is in outlet control.

10. Step 10: Calculate Outlet Velocity, V_O.

a. Calculate flow depth at culvert exit.
Use TW if $d_C < TW < D$ 1.73 < 3.13 < 4 TW = 3.13 ft

b. Calculate flow area: (11) (3.13) = 34.43 ft^2

c. Calculate exit velocity: $V_O = Q_{50}/A$ = (134/34.43) = 3.9 ft/s

11. Step 11: Review Results. Compare alternative design with constraints and assumptions.

a. Check cover: 199 – (190.7 + 4 + 1) = 3.3 ft, therefore OK.

b. L = 300 ft

c. El.$_{hO}$ = 194.0 ft < AHW of 194.1 ft, therefore OK.

d. Overtopping flood > Q_{100}, and $RS = Q_{100}$
Pavement-edge elevation of 199 ft – El.$_{hO}$ of 194.0 ft = 5 ft > 2 ft, OK.

e. Outlet velocity of 3.9 ft/s < 13.3 ft/s, therefore OK.
Use revetment riprap; see Section 31-3.04(03).

The outlet-control nomographs are designed for full flow. In this example, full flow does not exist because the TW depth of 3.3 ft is less than the rise of 4 ft and the culvert is on a flat slope. The above answer should be considered approximate. A more-accurate solution is provided in Section 31-8.0 with the microcomputer analysis.

12. Step 12: Documentation. Prepare a Report which includes the Culvert Design Form shown on Figure 31-7A, Chart 17 and Performance Curve for Design Example.

31-8.0 MICROCOMPUTER SOLUTION

31-8.01 Overview

Culvert hydraulic analysis can also be accomplished with the aid of the HYDRAIN software. The following example has been produced using the HY8 Culvert Analysis Microcomputer Program. It is the computer solution of the data provided in Section 31-7.0. Although Trials 1 and 2 of the culvert-design process can be worked on HY8, they are not shown in this example. Trial 3, the reinforced concrete box culvert, is shown.

The screens shown in the figures may not exactly match the version of HY8 available to the user because some editorial changes have been made to fit the screens in this text for presentation.

31-8.02 Data Input

After creating a file, the user will be prompted for the discharge range, site data, and culvert shape, size, material, and end-treatment type. The discharge range for this example will be from 0 to 222 ft^3/s. The site data are entered by providing culvert invert data. The data input prompt screen is shown as Figure 31-8A, HY8 Data Input Prompt Screen. If embankment data points are input, the program will fit the culvert in the fill and subtract the appropriate length.

31-8.02(01) Culvert Data

As an initial size estimate, try a concrete box culvert, 132-in x 48-in. For the culvert, assume that a conventional end treatment with 1:1 bevels and 45-deg wingwalls will be used. As each group of data is entered, the user is allowed to edit incorrect entries. Figure 31-8A shows the screen that summarizes the culvert information.

31-8.02(02) 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 eight coordinates listed. The channel data prompt screen is shown as Figure 31-8B, HY8 Channel Data Prompt Screen. After opening the irregular-channel file, the user will be prompted for channel slope of 0.001, number of cross-section coordinates, 8, and subchannel option. The subchannel option will be option 2, left and right overbanks with n = 0.08, and main channel with n = 0.03.

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, press P and the computer will display the channel cross-section shown in Figure 31-8C, Channel Cross Section. The user can identify input errors by looking at the plot. To return to the data input screens, press any key. If data are correct, press Enter. The user can then enter the roughness data for the main channel and overbanks.

31-8.03 Rating Curve

The program now has sufficient information to develop a uniform flow rating curve for the channel and provide the user with a list of options. See Figure 31-8D, HY8 Rating Curve

Prompt Screen. Selecting option T on the Irregular Channel Data Menu will command the program to compute the rating-curve data and display Figure 31-8D. Selecting option I will permit the user to interpolate data between calculated points.

The Tailwater Rating Curve Table consists of tailwater elevation, *TWE*, at normal depth, natural channel velocity, *Vel.*, and the shear stress at the bottom of the channel for various flow rates. At the design flow rate of 148.3 ft^3/s, the tailwater elevation will be 193.7 ft. The channel velocity will be 3 ft/s, and the shear will be 0.162 lb/ft^2. This information will be useful in the design of a channel lining if needed. Entering P will command the computer to display the rating curve for the channel. This curve, shown in Figure 31-8E, Tailwater vs. Flow Rate, is a plot of tailwater elevation vs. flow rate at the exit of the culvert.

<u>31-8.04 Roadway Data</u>

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 once P is entered, as illustrated in Figure 31-8F, Roadway Profile. 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 50.8 ft.

<u>31-8.05 Data Summary</u>

All of the data has now been entered and the summary table is displayed as shown in Figure 31-8G, HY8 Data Summary Prompt Screen. At this point, the data can be changed or the user can save the data and continue by pressing Enter, which will bring up the Culvert Program Options Menu.

<u>31-8.06 Performance Curve for Culvert Size of 132 in. x 48 in.</u>

From the Culvert Program Options Menu, the culvert performance curve table can be obtained by selecting option S. If option S is selected, the program will compute the performance-curve table without considering overtopping in the analysis. Because this 132-in. x 48-in. culvert is a preliminary estimate, the performance without considering overtopping is calculated and is shown as Figure 31-8H, HY8 Performance Curve Prompt Screen, 132-in. Span by 48-in. Rise.

The table indicates the controlling headwater elevation, *HW*, the tailwater elevation, and the headwater elevations associated with all possible control sections of the culvert. It is apparent from the table that at 148.3 ft^3/s, HW = 194.2 ft, which exceeds the design headwater of 194.1 ft. Consequently, the 132-in. x 48-in. box culvert is inadequate for the site conditions. Figure 31-8 I, Inlet/Outlet Control Headwaters plot, can be obtained by entering P. In this example, the culvert is operating in outlet control (the upper curve) throughout the discharge range. Backwater should be calculated as follows:

BW = Headwater El. - (Inlet El. + Tailwater Depth)

For this example, BW at Q = 148.3 ft^3/s is 194.2 – (190.7 + 3.3) = 0.2 ft

The backwater for the 132-in. x 48-in. culvert is greater than allowable for 1.5 in. The next larger size of 144 in. x 48 in. will be analyzed.

31-8.07 Performance Curve for Culvert Size of 144 in. x 48 in.

Because the design headwater criterion was not satisfied by a size of 132 in. x 48 in., try a size of 144 in. x 48 in. and modify the file accordingly. The resulting performance table shown as Figure 31-8J indicates that the design headwater will not be exceeded at 148.3 ft^3/s. The designer must check the roadway serviceability at Q_{100} and the maximum velocity at Q_{50}.

This structure is sized to satisfy the INDOT criteria on new alignment of an increase of 1.5 in. maximum backwater if compared to existing conditions. If the culvert had been a replacement structure, the INDOT criteria is a maximum total backwater of 12 in. The box-culvert size can be modified to satisfy this criterion.

31-8.08 Performance Curve for Culvert Size of 84 in. x 48 in.

The user can modify the existing program file to analyze a smaller barrel. Try a size of 84 in. x 48 in. From the Culvert Program Options Menu, press E to edit the file, and then E again to edit the culvert size. The prompts will be the same as they were for the 144 in. x 48 in. culvert. The user will return to the Culvert Data Summary Table directly without seeing the tailwater and overtopping menus again. Press F to rename the data file, or press Enter to save the changes into the current file and return to the Culvert Program Options Menu. The performance of this culvert can be checked by selecting option S for no overtopping. The performance curve table shown as Figure 31-8K, HY8 Performance Curve Prompt Screen, 84-in. Span by 48-in. Rise, appears.

Check the backwater for Q_{100} of 148.3 ft^3/s as follows:

BW = Headwater Elevation – (Inlet Elevation + TW depth)

BW = 194.9 – (190.7 + 3.3) = 0.9 ft < 1 ft, therefore OK.

Therefore, a reinforced-concrete box culvert of 84-in. x 48-in. RCB will be adequate if the Allowable Backwater, *ABW*, is increased to 12 in.

31-8.09 Minimize Culvert Span

Rather than using a series of trials to increase or reduce the culvert headwater to an acceptable level, as in the preceding examples, the Minimize Culvert Span feature of HY8 can be used. This feature is intended to allow the designer to use HY8 as a tool to perform culvert design for a circular, box, elliptical, or arch-shaped culvert based on a user's defined allowable headwater elevation, assuming no overtopping. This feature can be activated by pressing M. Once this option 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 in it in 6-in. increments. 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. Other 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 in Figure 31-8L, HY8 Minimization Routine Table Prompt Screen, must be printed from this screen because they are not printed with the output-listing routine. The *AHW* of 195 ft based on the inlet elevation of 190.7 ft, *TW* of 3.3 ft, and allowable backwater of 1 ft yields the results shown in Figure 31-8L.

This feature is a timesaver, as it avoids the need for repetitively editing a culvert size to obtain a controlling headwater elevation.

31-8.10 Overtopping Performance Curve, Culver Size of 144. in x 48 in.

Return to the 144-in. x 48-in. culvert to determine the amount of overtopping and the actual headwater from the Culvert Program Options Menu, and select O for overtopping. Figure 31-8M, Summary of Culvert Flows Prompt Screen, will appear.

This computation table is used if overtopping is 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. The overtopping discharge of 201 ft^3/s occurs at the roadway sag-point elevation of 194.6 ft. For the maximum discharge of 222 ft^3/s, 215 ft^3/s will flow through the culvert and 7.42 ft^3/s will flow over the road. From this information, a total (culvert and overtopping) performance curve, shown in Figure 31-8N, Total Performance Curve, can be obtained by selecting option P. 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. It is useful for comparing the effects of various combinations of culverts. See Figure 31-8 O, HY8 Overtopping Performance Curve Prompt Screen, 144-in. Span by 48-in. Rise.

31-8.11 Review

For the design criteria set forth for the example in Section 31-7.0, the design of a reinforced-concrete box culvert of 144 in. x 48 in. is selected as follows:

Q_{100} = 148.3 ft^3/s, HW = 194.1 ft, AHW = 194.1 ft, V_O = 3.93 ft/s, V_C = 3 ft/s.
Q_{50} = 134 ft^3/s, V_O = 3.73 ft/s. No energy dissipation necessary.

Road sag point elevation = 194.6 ft, Overtopping Q = 201 ft^3/s.
Therefore, Roadway Serviceability, $RS > Q_{100}$.

An energy dissipater is warranted at a velocity higher than 13.3 ft/s. See Section 31-3.0.

31-9.0 IMPROVED END TREATMENTS

31-9.01 General

An improved end treatment is a flared culvert inlet with an enlarged face section and a hydraulically-efficient throat section. An improved end treatment may have a depression, or fall, incorporated into the end-treatment structure or located upstream of the end treatment. The depression is used to exert more head on the throat section for a given headwater elevation. Therefore, an improved end treatment improves culvert performance by providing a more-efficient control section (the throat). An improved end treatment with a fall also improves performance by increasing the head on the throat. The following also applies.

1. An improved end treatment is not recommended for use with a culvert flowing in outlet control because the simple beveled edge is of equal benefit.

2. Design criteria and methods have been developed for two basic end treatment designs -- the side-tapered end treatment and the slope-tapered end treatment.

3. Improved-end-treatment design charts are available for a rectangular box culvert or a circular pipe culvert.

4. The use of an improved end treatment must be accompanied by an economic justification for its use, subject to the approval of the Hydraulics Engineer.

31-9.02 Side-Tapered

The side-tapered end treatment has an enlarged face section with the transition to the culvert barrel accomplished by tapering the side walls (Figure 31-9A, Side-Tapered End Treatment). The face section is approximately the same height as the barrel height and the inlet floor is an extension of the barrel floor. The end treatment roof may slope upward slightly, provided that the face height does not exceed the barrel height by more than 10 percent (1.1*D*). The intersection of the tapered sidewalls and the barrel is defined as the throat section.

There are two possible control sections: the face and the throat. HW_f, shown in Figure 31-9A, is the headwater depth measured from the face section invert. HW_t is the headwater depth measured from the throat section invert. The throat of a side-tapered end treatment is a very efficient control section. The flow contraction is nearly eliminated at the throat. 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 end treatment. See Figure 31-9B, Side-Tapered End Treatment (Upstream 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 ensure 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.

31-9.03 Slope-Tapered

The slope-tapered end treatment also has an enlarged face section with tapered sidewalls meeting the culvert-barrel walls at the throat section (Figure 31-9C, Slope-Tapered End Treatment with Vertical Face). A vertical fall is incorporated into the end treatment between the face and throat sections. The fall concentrates more head on the throat section. At the location where the steeper slope of the end treatment intersects the flatter slope of the barrel, a third section, designated the bend section, is formed.

A slope-tapered end treatment 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 included herein. The size of the bend section is established by locating it a minimal distance upstream from the throat so that it will not control the flow.

The slope-tapered end treatment 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 the end treatment are larger than the face sections of an equivalent depressed side-tapered end treatment. The required face size can be reduced by the use of bevels or other favorable edge configurations. See Figure 31-9C, Slope-Tapered End Treatment with Vertical Face.

The slope-tapered end treatment is the most complex inlet improvement recommended herein. Construction difficulties are inherent, but the benefits in increased performance can be significant. With proper design, a slope-tapered end treatment passes more flow at a given headwater elevation than another configuration. A slope-tapered end treatment can be applied to either a box culvert or a circular-pipe culvert. For the latter application, a square to round transition is used to connect the rectangular slope-tapered end treatment to the circular end-treatment pipe.

31-9.04 Hydraulic Design

31-9.04(01) Inlet Control

A tapered end treatment's control sections include the face, the bend (for a slope-tapered end treatment), and the throat. A depressed side-tapered end treatment 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. See Figure 31-9D, Inlet-Control Performance Curves (Schematic).

1. Side-Tapered End Treatment. The throat should be designed to be the primary control section for the design range of flows and headwaters. Because 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 a lower flow rate and headwater, the face will usually control the flow.

2. Slope-Tapered End Treatment. The 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. The

bend section will not act as the control section if the dimensional criteria described herein are followed. However, the bend will contribute to the inlet losses which are included in the inlet loss coefficient, K_E.

31-9.04(02) Outlet Control

If a culvert with a tapered end treatment performs in outlet control, the hydraulics are the same as described in Section 31-5.0 for all culverts. The tapered-end-treatment entrance loss coefficient, K_E, is 0.2 for a side-tapered or a slope-tapered end treatment. 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.

31-9.05 Design Methods

Tapered end treatment design begins with the selection of the culvert-barrel size, shape, and material. The calculations are performed using the Culvert Design Form provided in Section 31-10.0. The design nomographs included in HDS #5 are used to design the tapered end treatment. 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 or without tapered end treatments, all of which satisfy the site design criteria. The designer must select the best design for the site under consideration.

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 end-treatment 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 end treatment is minor compared with the cost of the barrel.

Performance curves should be considered in understanding the operation of a culvert with a tapered end treatment. Each potential control section (face, throat, or outlet) has a performance curve, based on the assumption that a specific section controls the flow. Calculating and plotting the performance curves results in a graph similar to that shown in Figure 31-9E, Culvert Performance Curve (Schematic), including 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. In the higher discharge range, outlet control governs. The crest and bend performance curves are not calculated because they do not govern in the design range.

31-10.0 TABLES AND FORMS

Section 31-10.0 includes the figures as follows.

31-10A Recommended Manning's *n* Value. These are the recommended Manning's *n* values used in the hydraulic design of a culvert.

31-10B Entrance Loss Coefficient (Outlet Control, Full or Partly Full). This coefficient, K_E, is for a culvert based on the type of entrance.

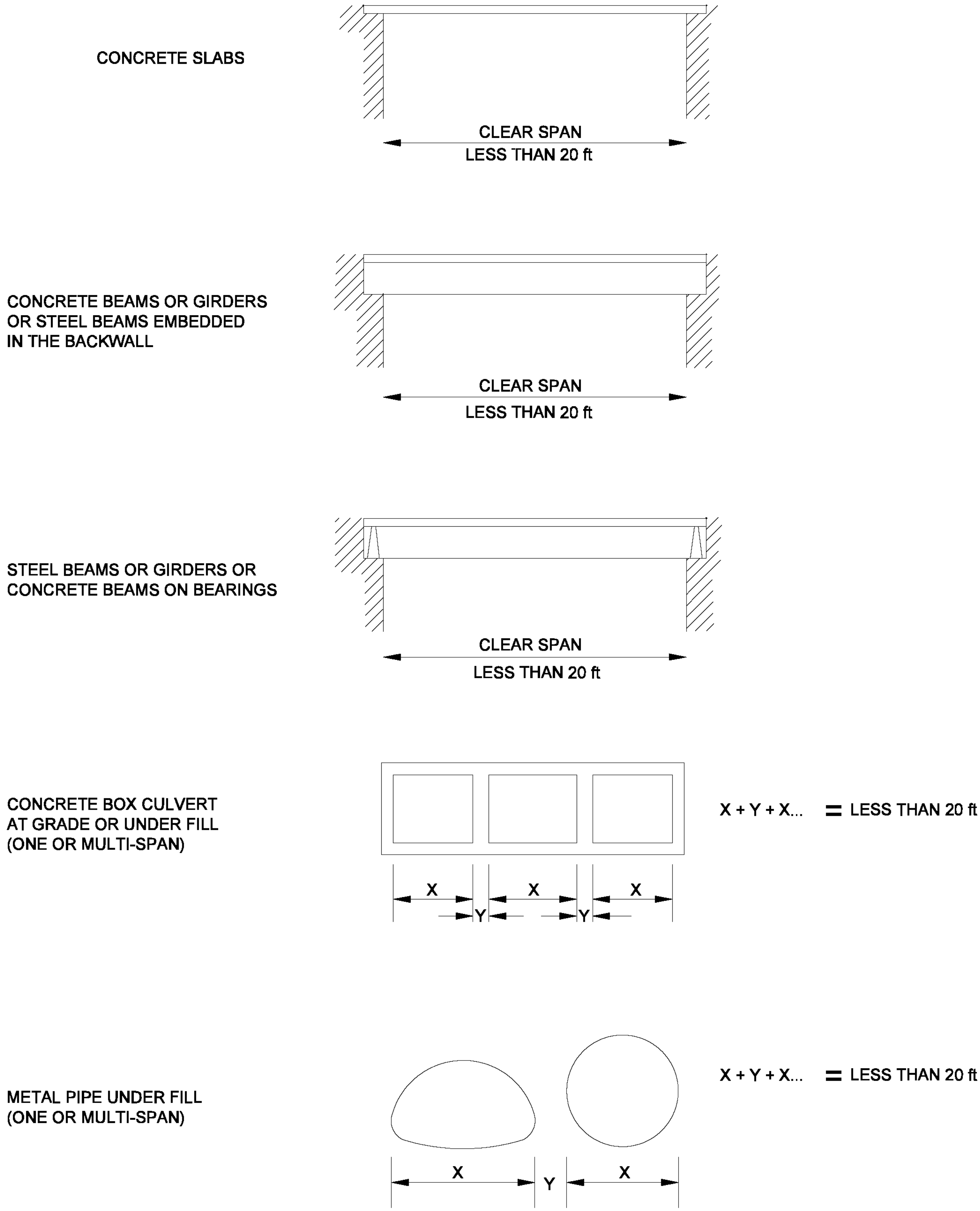
31-10C Entrance Loss Coefficient (Standard INDOT Culvert). This coefficient, K_E, is for a specific culvert as shown on the INDOT *Standard Drawings*.

Editable versions of the following forms may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/.

31-10D Culvert Design Form, Conventional End Treatment

31-10E Culvert Design Form, Tapered End Treatment

31-10F Culvert Design Form, Mitered End Treatment



MAXIMUM SPAN LENGTHS FOR CULVERTS

Figure 31-1A

Symbol	Definition	Unit
A	Area of cross section of flow	ft^2
AHW	Allowable headwater	ft
B	Barrel width	in. or ft
BW	Backwater	ft
D	Culvert diameter or barrel height	in. or ft
d	Depth of flow	ft
d_C	Critical depth of flow	ft
g	Acceleration due to gravity	ft/s^2
H	Sum of $H_E + H_f + H_O$	ft
H_b	Bend head loss	ft
H_E	Entrance head loss	ft
H_f	Friction head loss	ft
H_L	Total energy losses	ft
H_O	Outlet or exit head loss	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/s
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/s
V_d	Mean velocity in downstream channel	ft/s
V_O	Mean velocity of flow at culvert outlet	ft/s
V_U	Mean velocity in upstream channel	ft/s
γ	Unit weight of water	lb/ft^3
τ	Tractive force	lb/ft^2

CULVERT EQUATION SYMBOLS

Figure 31-1B

Functional Classification	Allowable Backwater	Roadway Serviceability	Allowable Velocity
Freeway	Q_{100}	Q_{100}	Q_{50}
Non-Freeway ≥ 4 Lanes	Q_{100}	Q_{100}	Q_{50}
Two-Lane Facility			
AADT ≥ 3000	Q_{100}	Q_{100}	Q_{50}
3000 > AADT ≥ 1000	Q_{100}	Q_{25}	Q_{25}
AADT < 1000	Q_{100}	Q_{10}	Q_{10}
Drive	Q_{100}	Q_{10}	Q_{10}

Note: The design-storm frequency for a culvert-extension structure is identical to that for a new culvert structure. Traffic volume is for a 20-year projection.

DESIGN-STORM FREQUENCY, CULVERT

Figure 31-3A

Structure Application	Minimum Circular-Pipe Size	Minimum Deformed-Pipe Area
Drive	15 in.	1.1 ft^2
Mainline or Public-Road Approach (2 lanes)	15 in.	1.1 ft^2
Mainline or Public-Road Approach (≥ 3 Lanes)	36 in.	6.7 ft^2

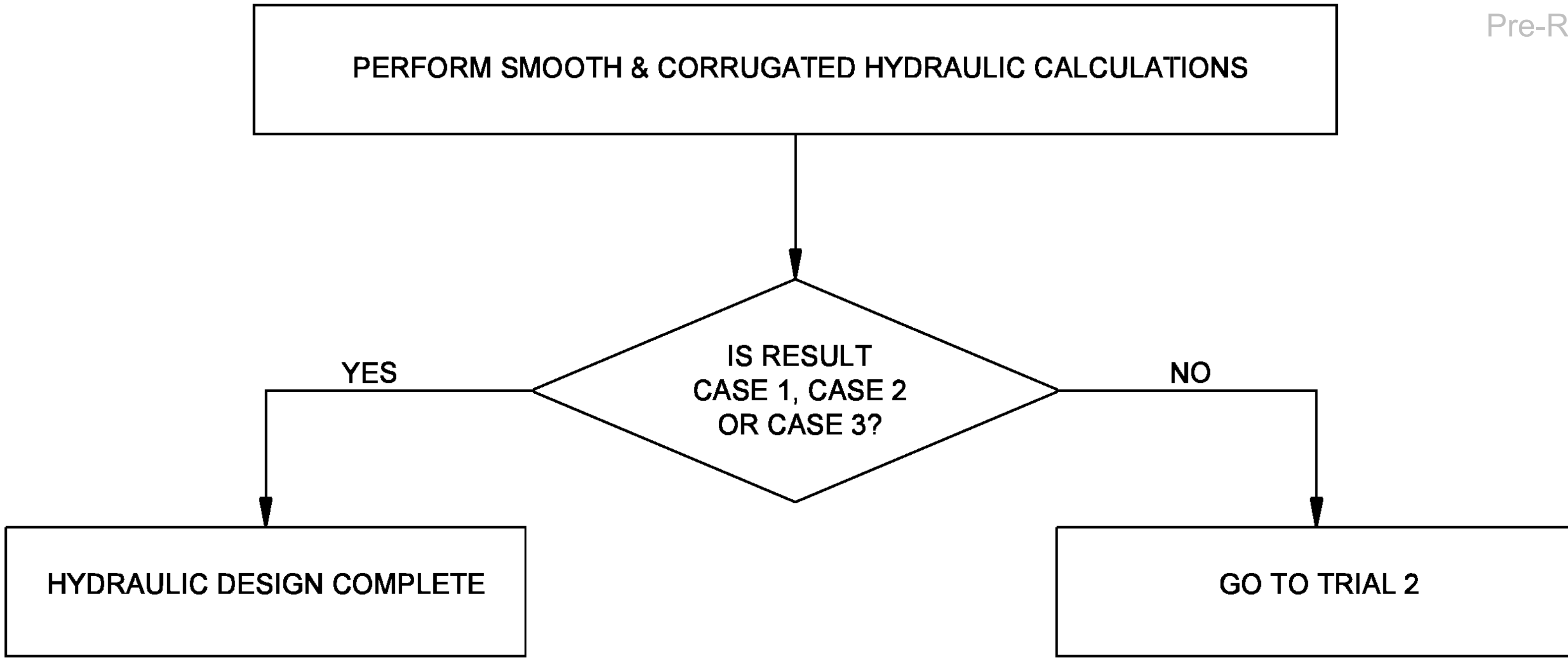
MINIMUM PIPE-CULVERT SIZE

Figure 31-3B

Structure Diameter or Span, *S* (ft)	Sump Required for Stream Bed of Sand (in.)	Sump Required for Stream Bed of Other Soil (in.)	Sump Required for Stream Bed of Rock or Till (in.)
< 4	6	3	3
$4 \le S < 12$	12	6	3
$12 \le S < 20$	18	12	3

Pipe or Box Structure Sump Requirement

31-3B(1)

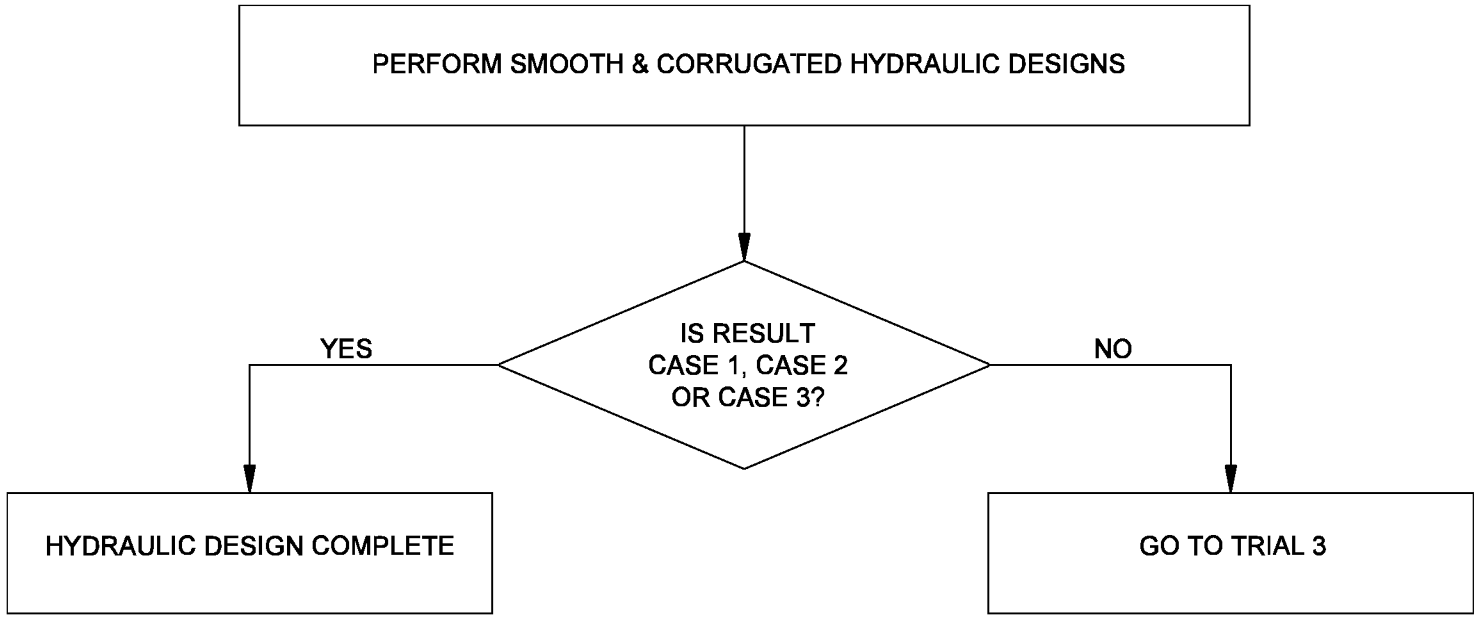


LEGEND

CASE 1 - REQUIRED SMOOTH AND CORRUGATED PIPE SIZES ARE IDENTICAL
CASE 2 - REQUIRED SMOOTH AND CORRUGATED PIPE SIZES ARE DIFFERENT
CASE 3 - THERE IS AN ACCEPTABLE PIPE SIZE FOR ONE INTERIOR TYPE, BUT NO ACCEPTABLE PIPE SIZE CAN BE FOUND FOR THE REMAINING INTERIOR TYPE

CULVERTS DESIGN PROCESS
(Trial 1 - Single Circular Pipe)

Figure 31-3C

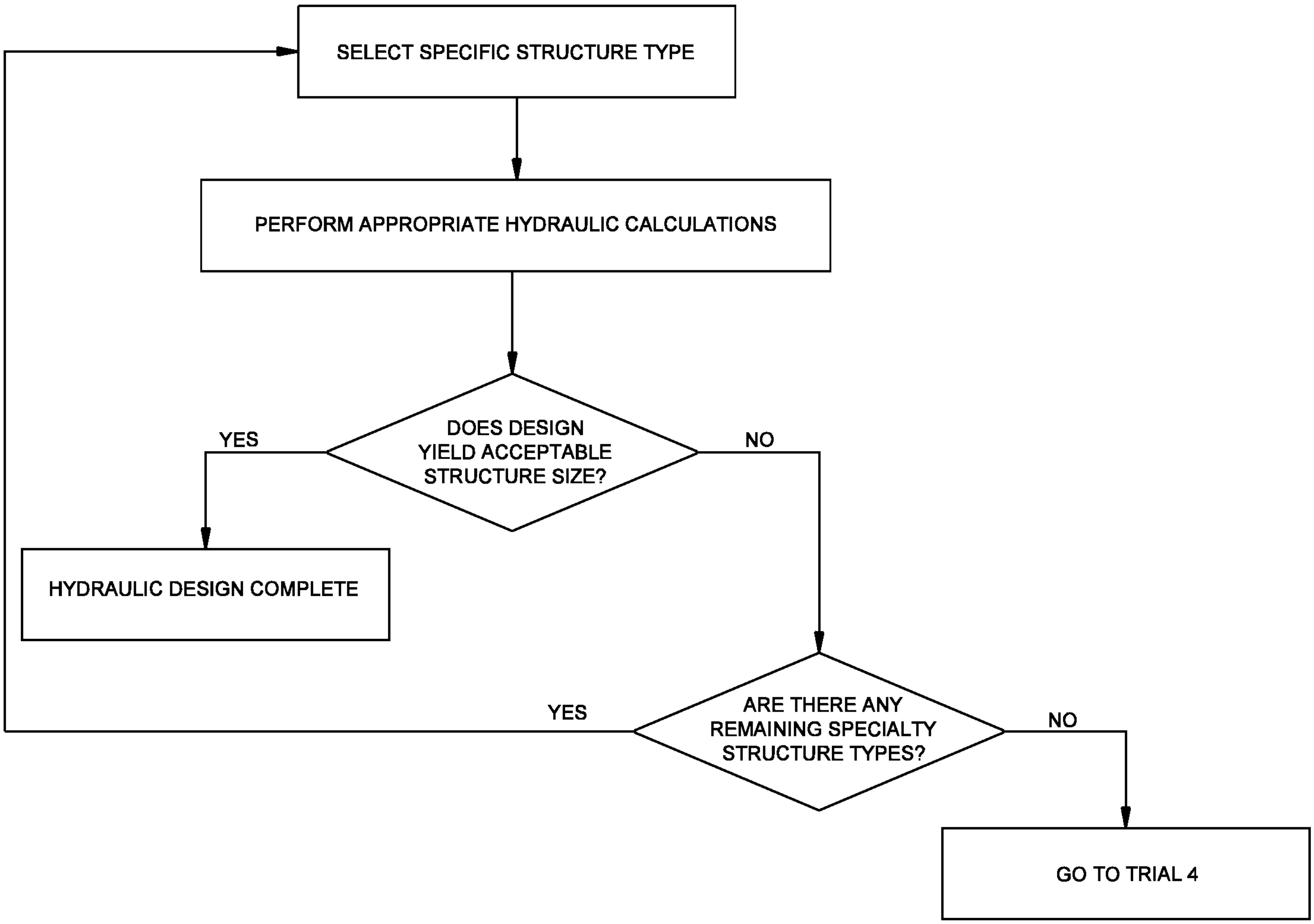


LEGEND

CASE 1 - REQUIRED SMOOTH AND CORRUGATED PIPE SIZES ARE IDENTICAL
CASE 2 - REQUIRED SMOOTH AND CORRUGATED PIPE SIZES ARE DIFFERENT
CASE 3 - THERE IS AN ACCEPTABLE PIPE SIZE FOR ONE INTERIOR TYPE, BUT NO ACCEPTABLE PIPE SIZE CAN BE FOUND FOR THE REMAINING INTERIOR TYPE

CULVERT DESIGN PROCESS
(Trial 2 - Single Deformed Pipe)

Figure 31-3D

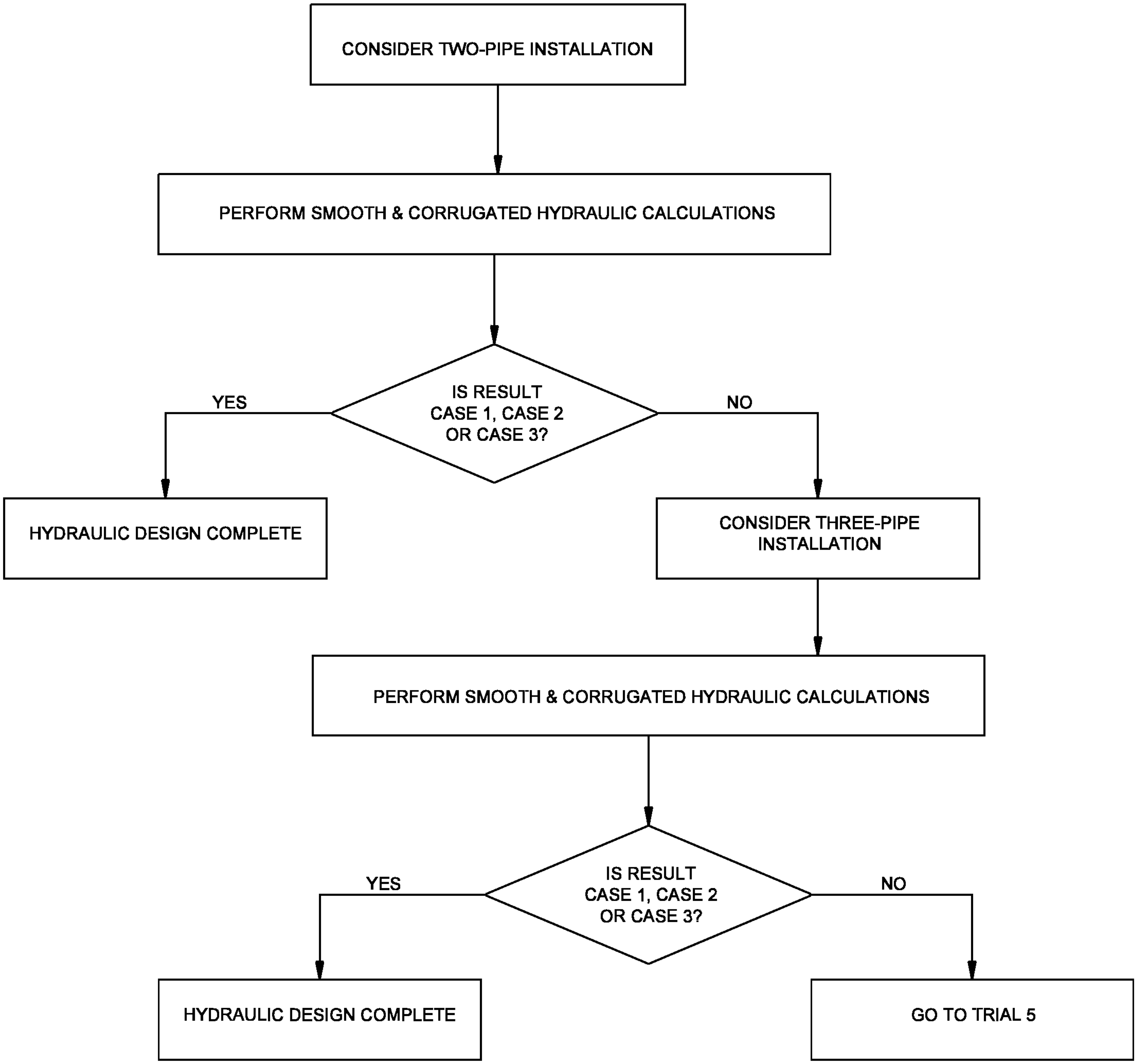


SPECIALTY STRUCTURE TYPES ARE TO BE CONSIDERED IN FOLLOWING ORDER:

1. PRECAST REINFORCED CONCRETE BOX SECTIONS
2. PRECAST REINFORCED CONCRETE THREE-SIDED CULVERT
3. STRUCTURAL PLATE ARCH
4. OTHER STRUCTURE TYPE APPROVED BY HYDRAULICS UNIT

CULVERT DESIGN PROCESS
(Trial 3 - Single Specialty Structure)

Figure 31-3E

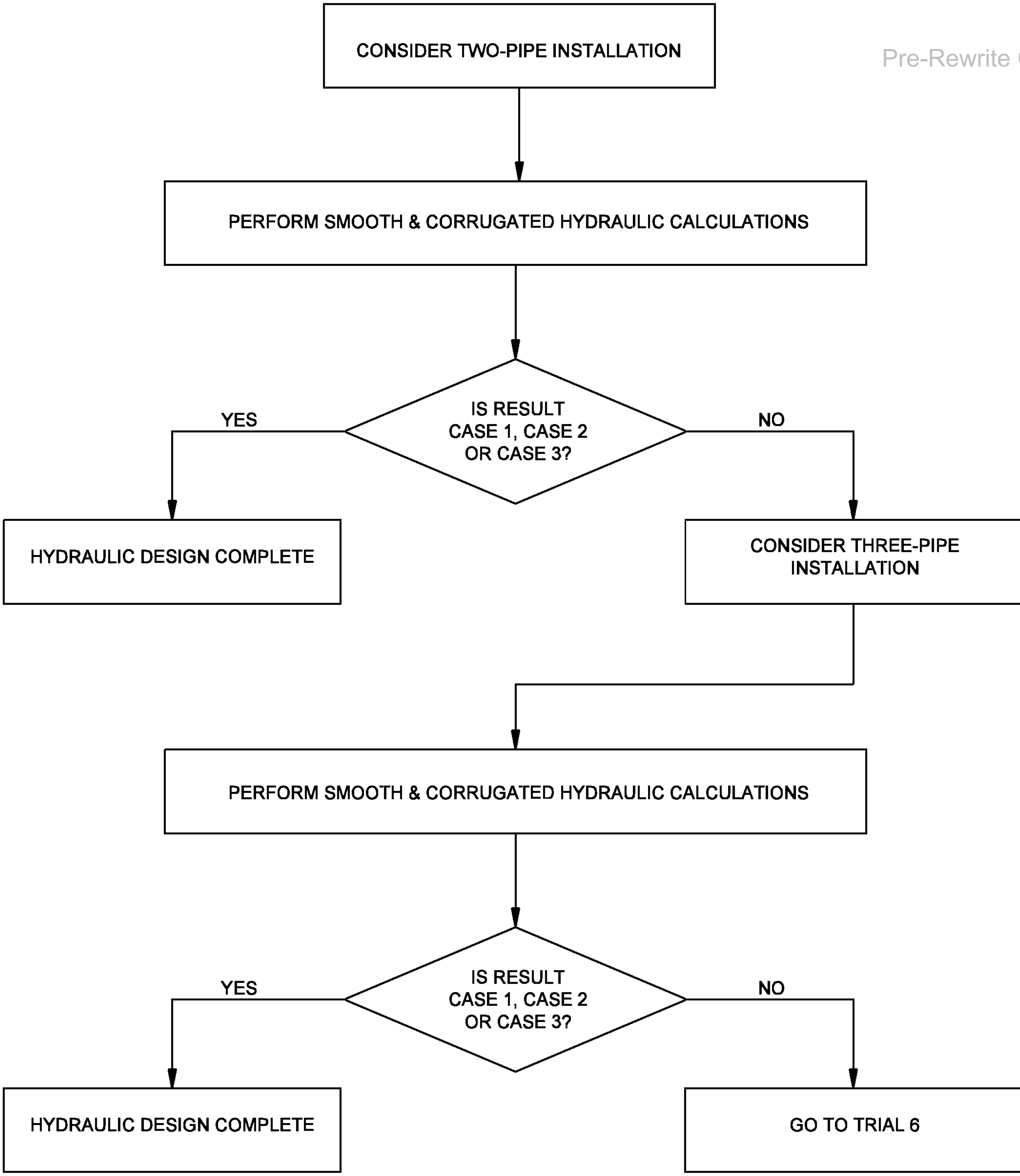


LEGEND

CASE 1 - REQUIRED SMOOTH AND CORRUGATED PIPE SIZES ARE IDENTICAL
CASE 2 - REQUIRED SMOOTH AND CORRUGATED PIPE SIZES ARE DIFFERENT
CASE 3 - THERE IS AN ACCEPTABLE PIPE SIZE FOR ONE INTERIOR TYPE, BUT NO ACCEPTABLE PIPE SIZE CAN BE FOUND FOR THE REMAINING INTERIOR TYPE

CULVERT DESIGN PROCESS
(Trial 4 - Multiple Circular Pipes)

Figure 31-3F

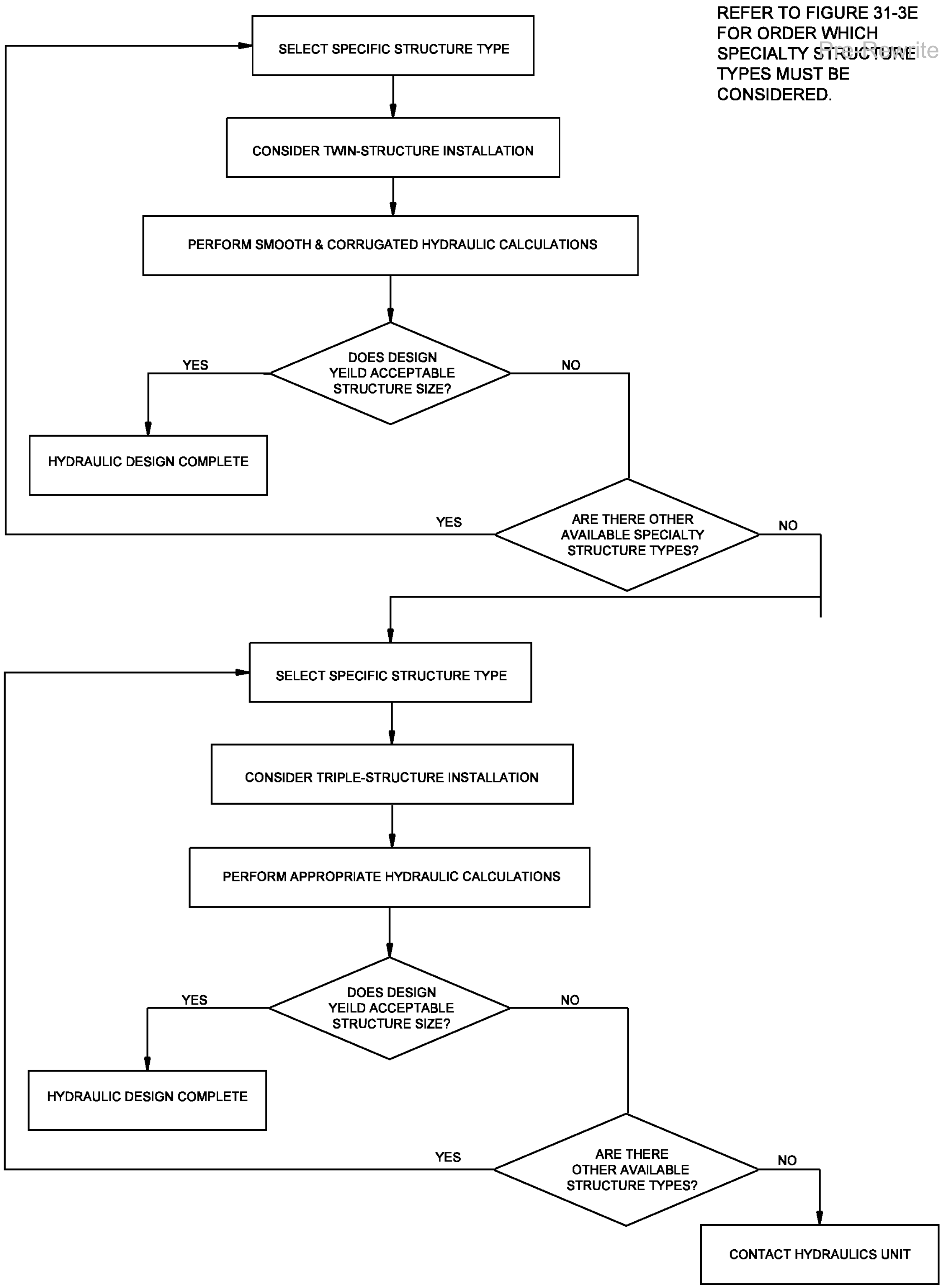


LEGEND

CASE 1 - REQUIRED SMOOTH AND CORRUGATED PIPE SIZES ARE IDENTICAL
CASE 2 - REQUIRED SMOOTH AND CORRUGATED PIPE SIZES ARE DIFFERENT
CASE 3 - THERE IS AN ACCEPTABLE PIPE SIZE FOR ONE INTERIOR TYPE, BUT NO ACCEPTABLE PIPE SIZE CAN BE FOUND FOR THE REMAINING INTERIOR TYPE

CULVERT DESIGN PROCESS
(Trial 5 - Multiple Deformed Pipes)

Figure 31-3G



CULVERT DESIGN PROCESS
(Trial 6 - Multiple Specialty Pipes)

Figure 31-3H

CULVERT INSPECTION REPORT

ROUTE R.P. DISTRICT:

INVENTORY DATA

NO. LANES: CLEARANCE: FILL:
DESCRIPTION:
CULVERT NO. - -
(route)-(co. no.)-(r.p.)

ROADWAY ITEMS – RATING	COMMENTS
ALIGNMENT	
PAVEMENT	
SHOULDERS	
EMBANKMENT	
OVERALL	

CULVERT ITEMS – RATING	COMMENTS
HEADWALLS	
WINGWALLS	
BARREL / BOX	
SETTLEMENT	
OVERALL	

CHANNEL ITEMS – RATING	COMMENTS
ALIGNMENT	
EROSION	
SCOUR	
DRIFT / SEDMT.	
ADEQUACY	
OVERALL	

OVERALL RATING:
MX. NEEDED
INSTIP CODE

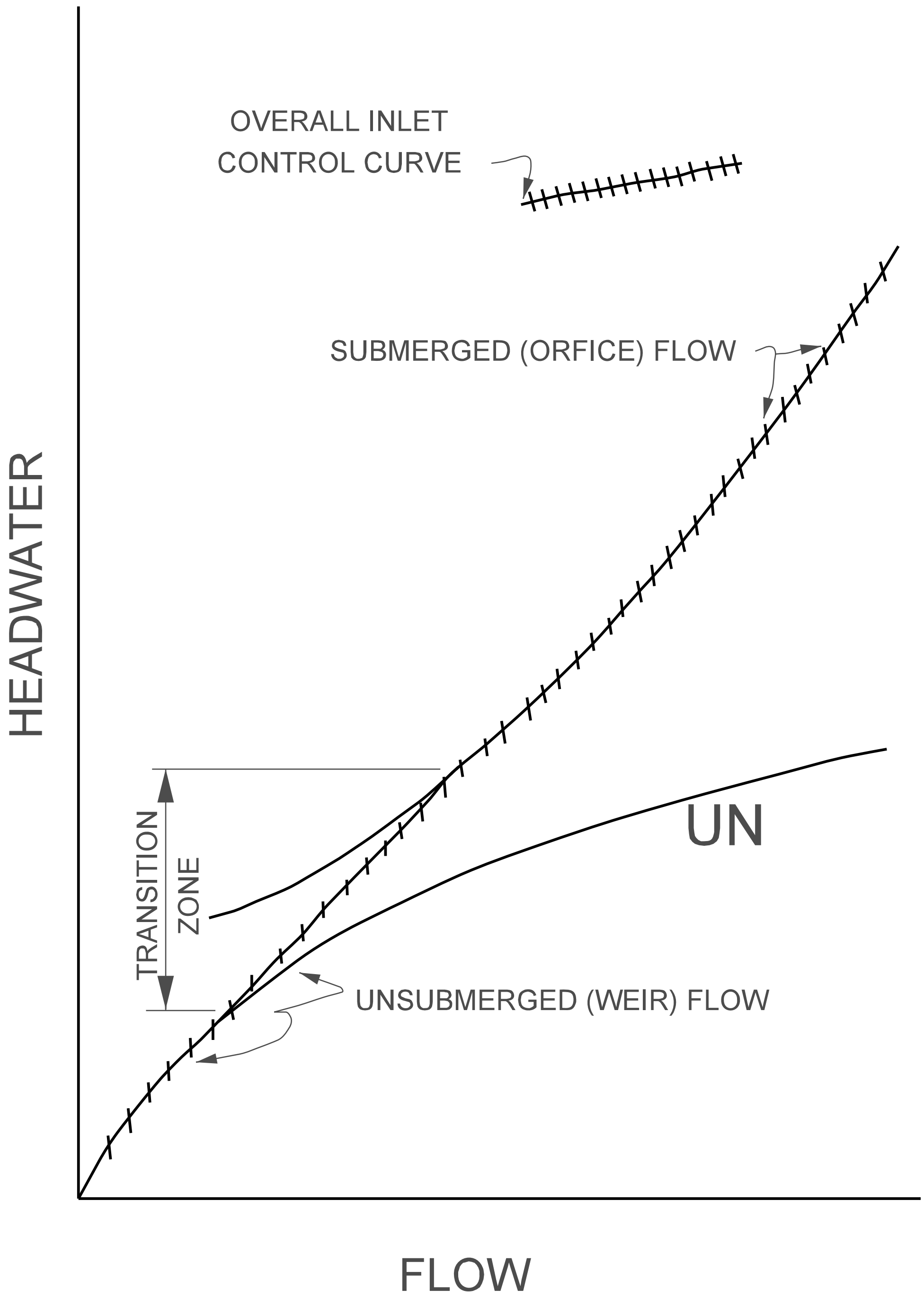
INSTIP Codes: 0 = No Work Needed. 1 = Replace Structure by Contract. 2 = Repair Structure by Contract.

INSPECTOR: DATE:

RISE	4 ft		5 ft		6 ft		7 ft		8 ft	
SPAN	Wt., T / ft	Lgth., ft	Wt., T / ft	Lgth., ft	Wt., T / ft	Lgth., ft	Wt., T / ft	Lgth., ft	Wt, T / ft	Lgth., ft
14 ft	3.15	6	3.30	6	3.45	6	3.60	6	3.75	5
16 ft	3.45	6	3.60	6	3.75	5	3.90	5	4.05	5
18 ft	3.75	5	3.90	5	4.05	5	4.20	5	4.35	5
20 ft	4.05	5	4.20	5	4.35	5	4.50	4	4.65	4

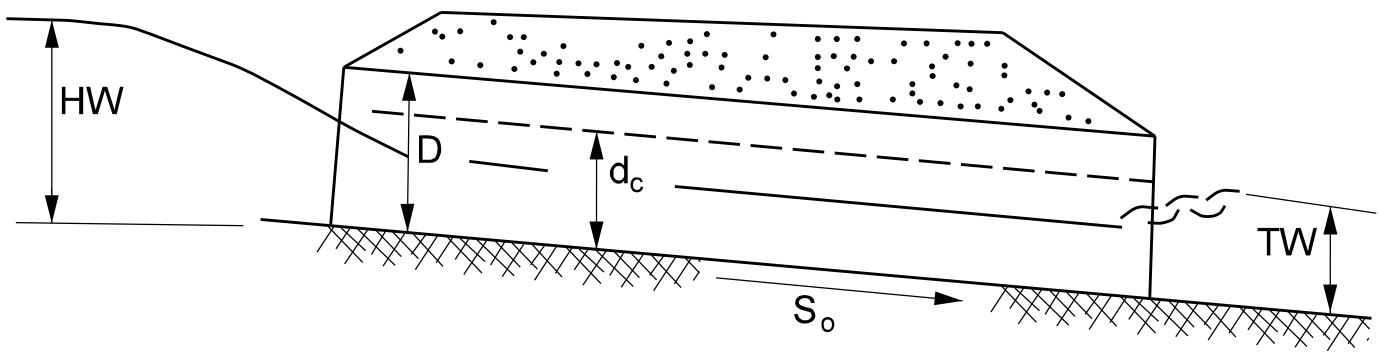
OVERSIZE-BOX-CULVERT SEGMENTS WEIGHT AND LENGTH

Figure 31-4B



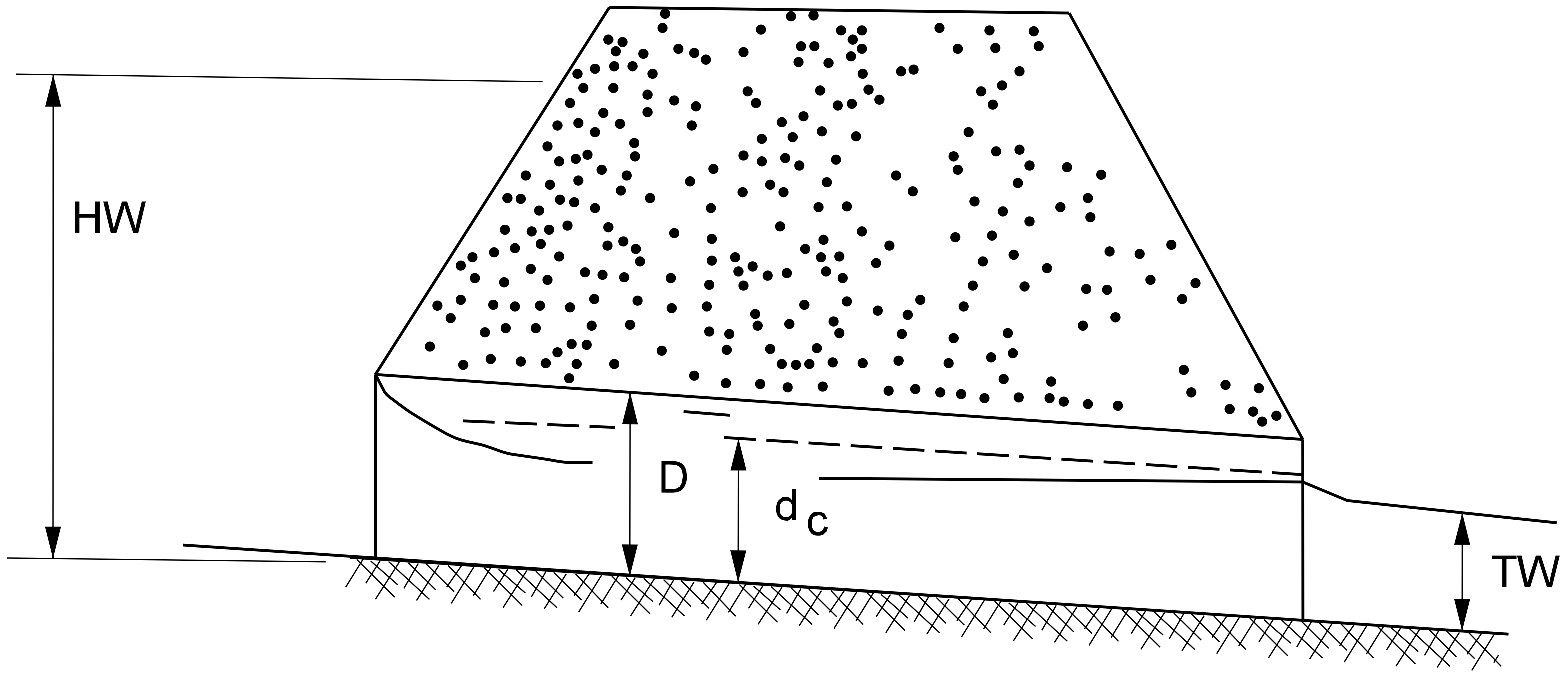
UNSUBMERGED, SUBMERGED AND TRANSITION

Figure 31-5A



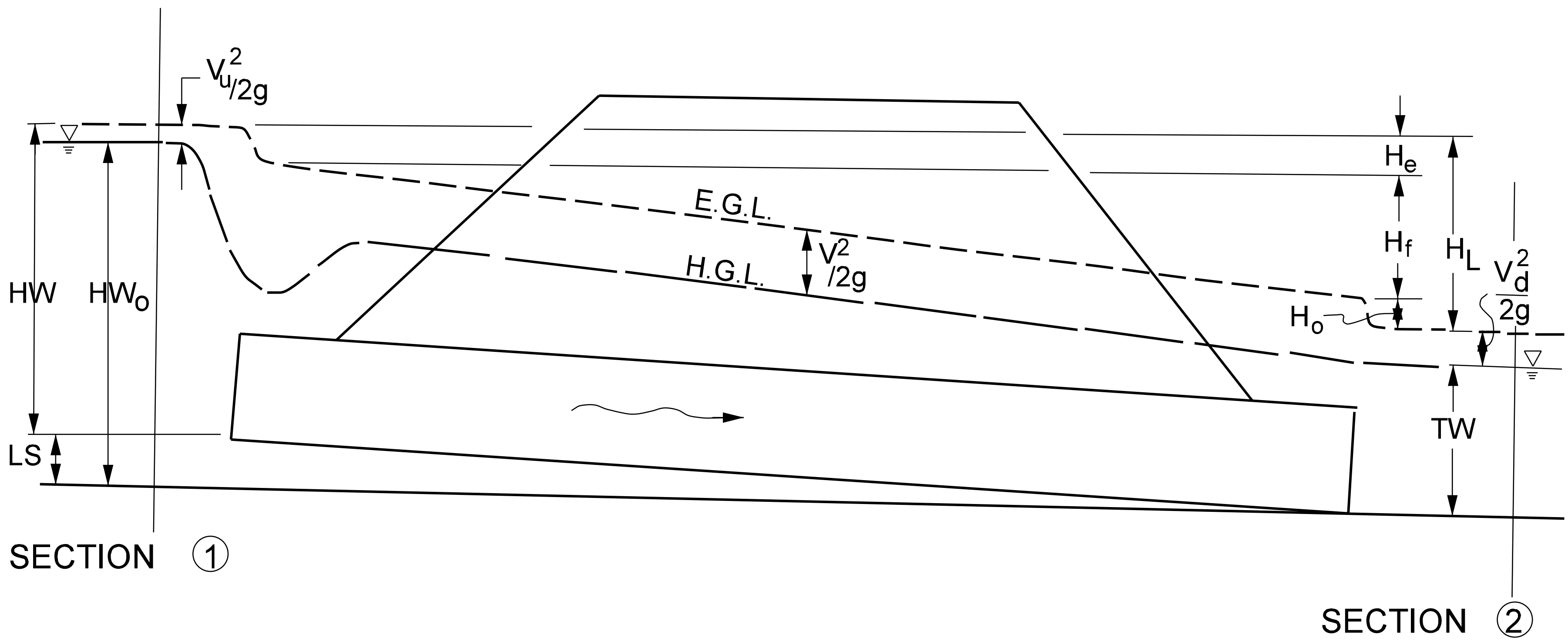
FLOW TYPE I

Figure 31-5B



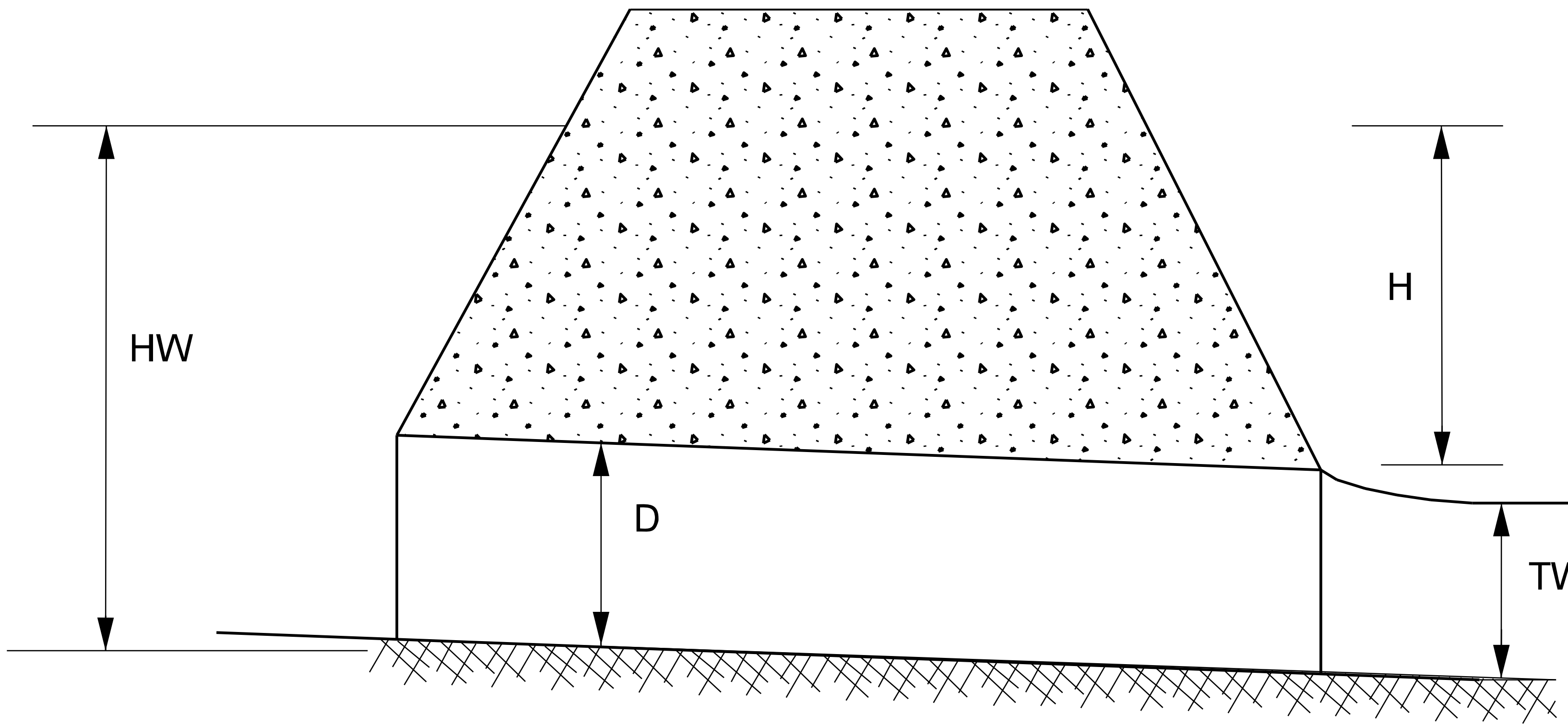
FLOW TYPE V

Figure 31-5C



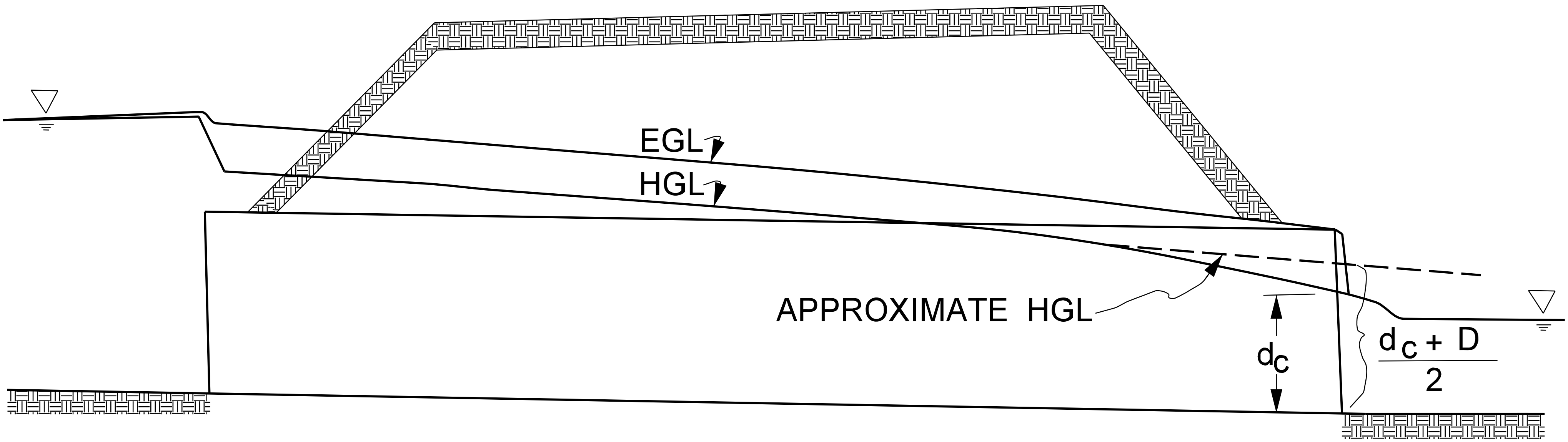
FLOW TYPE IV

Figure 31-5D



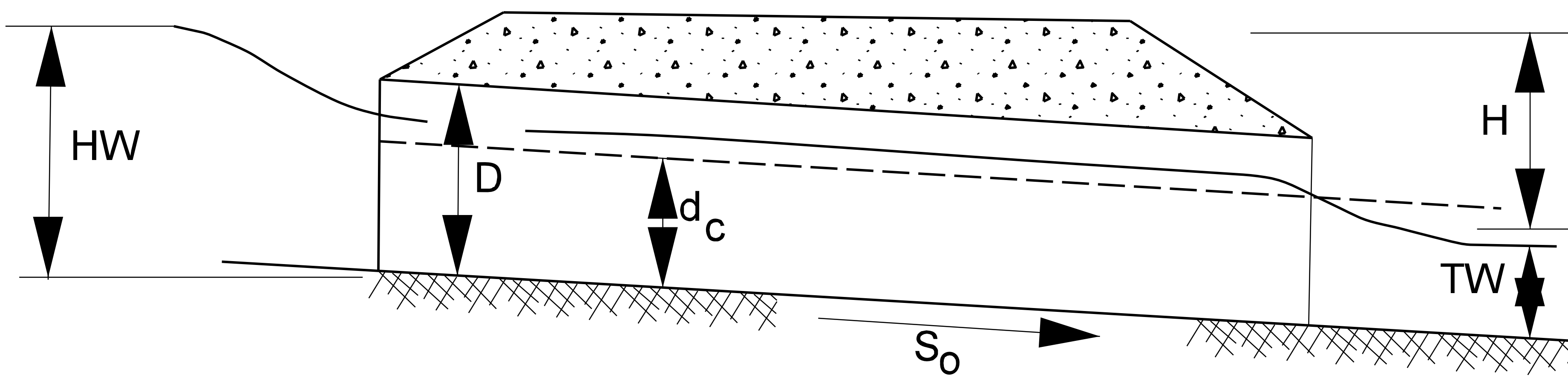
FLOW TYPE VI

Figure 31-5E



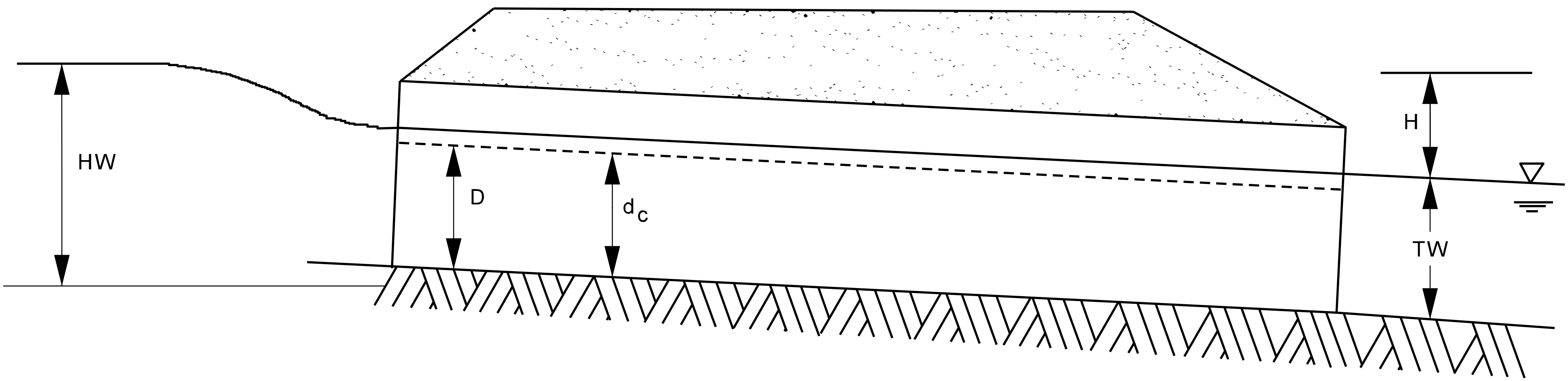
FLOW TYPE VII

Figure 31-5F



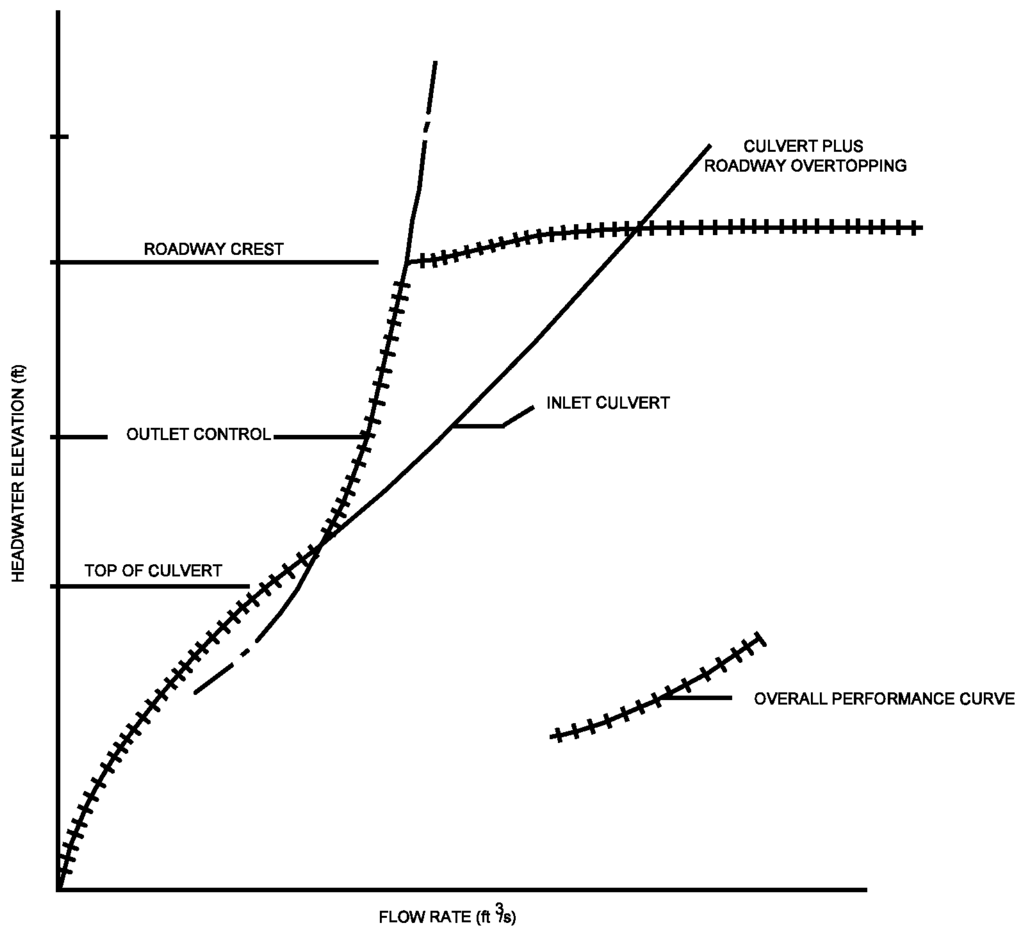
FLOW TYPE II, TW < d_C

Figure 31-5G



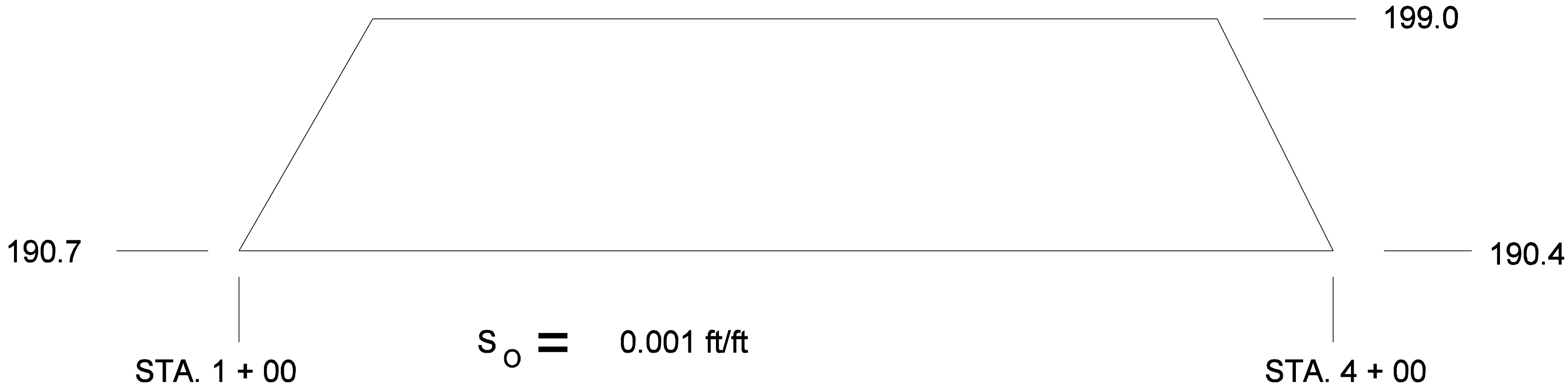
FLOW TYPE III, TW > d_c

Figure 31-5H



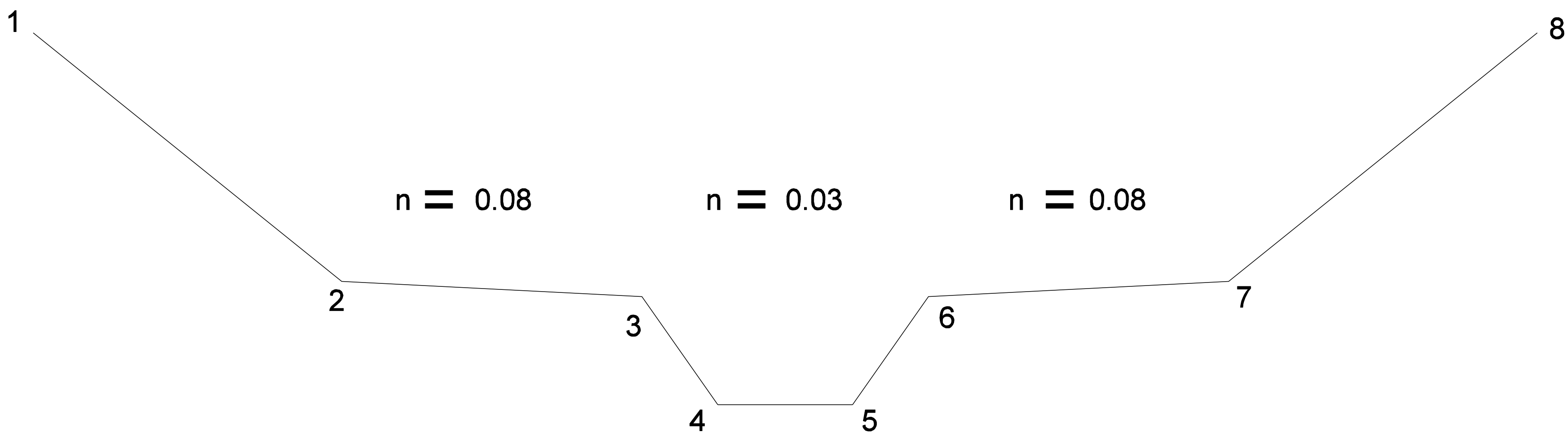
OVERALL PERFORMANCE CURVE

Figure 31-5l



NOMOGRAPH DESIGN EXAMPLE - SITE DATA

Figure 31-7.1



NOMOGRAPH DESIGN EXAMPLE DOWNSTREAM CHANNEL SECTION

Figure 31-7.2

PROJECT: Example Problem Nomograph

STATION: Test

SHEET 1 OF 1

CULVERT DESIGN FORM

DESIGNER/DATE: ____________ OF _____

REVIEWER/DATE: ____________ OF _____

HYDROLOGICAL DATA

SEE ADD=L. SHEETS

☒ METHOD: USGS
☐ DRAINAGE AREA: ________ ☐ STREAM SLOPE: ________
☐ CHANNEL SHAPE: ________
☐ ROUTING: ________ ☐ OTHER: ________

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (ft^3/s)	TW (ft)
50	133	3.13
100	148.3	3.30

See Figure 31-7A(1) for Details

CULVERT DESCRIPTION: MATERIAL-SHAPE-SIZE-ENTRANCE	TOTAL FLOW Q (ft^3/s)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i /D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c + D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
Trial 1: Single RCP or CMP 60 inch	148.3	148.3	1.16	5.80	-	196.5		No	Good							Case 4
Trial 2: Single Deformed Conc. 100” x 63”	148.3	148.3	0.75	3.97	-	194.7		No	Good							Case 4
Trial 2: Single CSP-A 72” x 48”	148.3	148.3	1.55	6.20	-	196.9		No	Good							Case 4
Trial 3: RCB 132” x 48”	148.3	148.3	0.71	2.83	-	193.5	3.3	1.73	2.87	3.3	0.5	0.40	194.0	194.0	4.3	Looks OK

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) HW_i /D = HW/D OR HW_t/D FROM DESIGN CHARTS

FALL = HW_i - (EL_{hd} - EL_{sf}); FALL IS ZERO FOR CULVERTS ON GRADE

(3) EL_{hi} = HW_i + EL_i (INVERT OF INLET CONTROL SECTION)

(4) TW BASED ON DOWNSTREAM CONTROL OR FLOW DEPTH IN CHANNEL

(6) h_o = TW or (d_c + D)/2 (WHICHEVER IS GREATER)

(7) $H = (1+k_e+(19.63\ n^2L)/R^{1.33})\ V^2/2g$

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

SUSCRIPT DEFINTIIONS:

- a Approximate
- f Culvert Face
- hd Design Headwater
- hi Headwater in Inlet Control
- ho Headwater in Outlet Control
- i Inlet Control Section
- o Outlet
- sf Streambed at Culvert Face
- tw Tailwater

COMMENTS/DISCUSSION:

This is an approximate solution because box not flowing full. Check with microcomputer HY8 for exact solution.

CULVERT BARREL SELECTED:

SIZE: 132 x 48

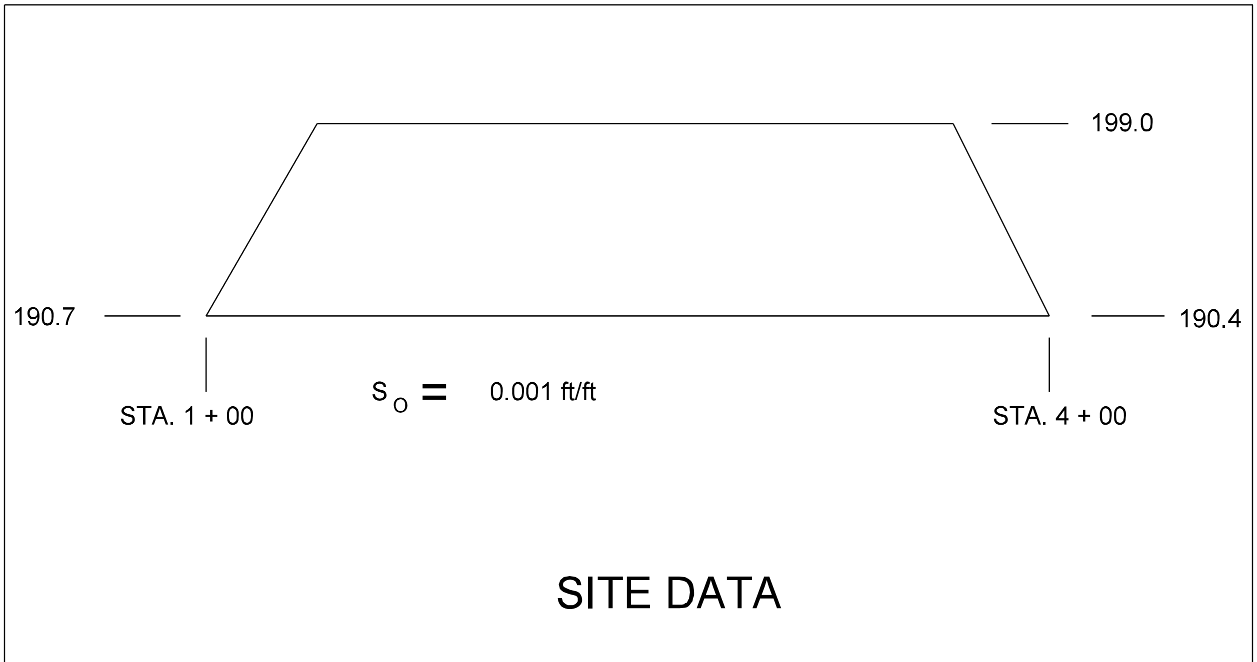
SHAPE: RCB

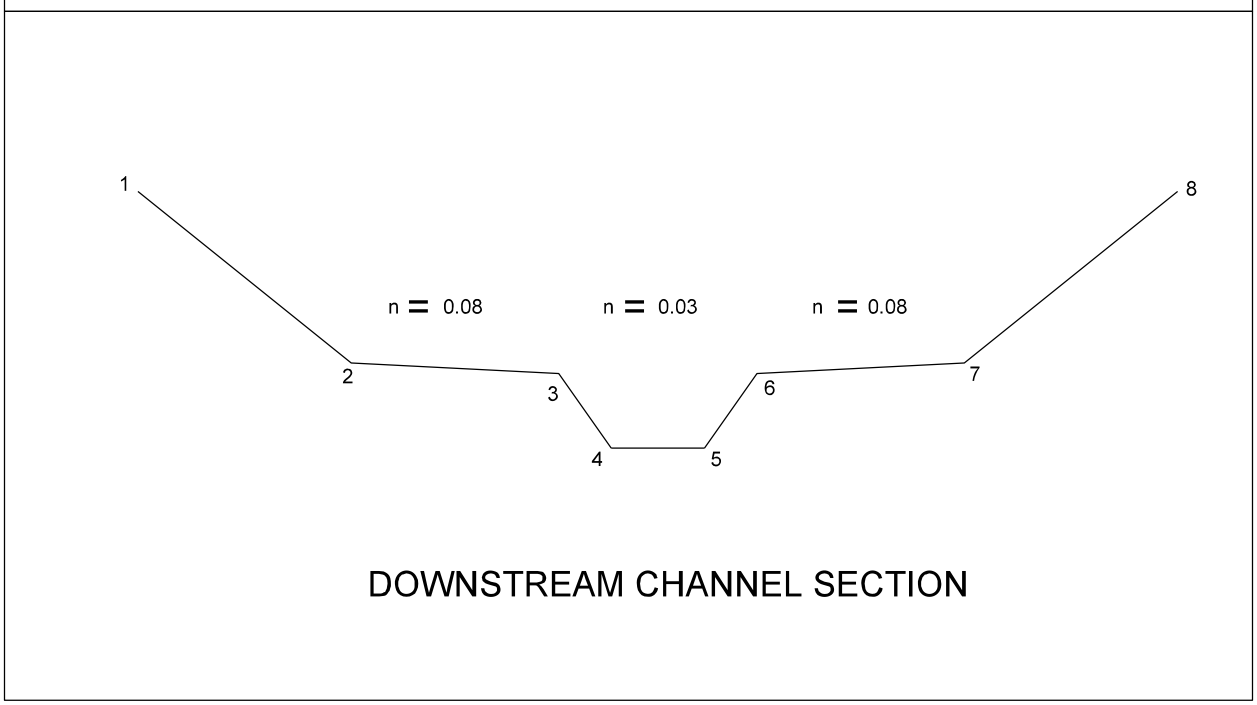
MATERIAL: Concrete n: 0.012

ENTRANCE: Conventional Bevel

CHART 17 AND PERFORMANCE CURVE FOR DESIGN EXAMPLE

Figure 31-7A





NOMOGRAPH DESIGN EXAMPLE

Figure 31-7A(1)

```
FILE NAME: 31833       FHWA CULVERT ANALYSIS        DATE: 10-12-1998
TAILWATER FILE: 3180   HY8, VERSION 6.0             CULVERT NO. 1 OF 1

******* CULVERT DATA SUMMARY ****************************************************

  BARREL SHAPE:            BOX
  BARREL SPAN:             132 in.
  BARREL RISE:             48  in.
  BARREL MATERIAL:         CONCRETE
  BARREL MANNING'S n:      0.012
  INLET TYPE:              CONVENTIONAL
  INLET EDGE AND WALL:     1:1 BEVEL (45 DEG. FLARE)
  INLET DEPRESSION:        NONE

       <ENTER> TO CONTINUE       <NUMBER> TO EDIT ITEM

1-Help   2-Prog   3       4       5-End   6       7-Edit  8       9-Dos  10
```

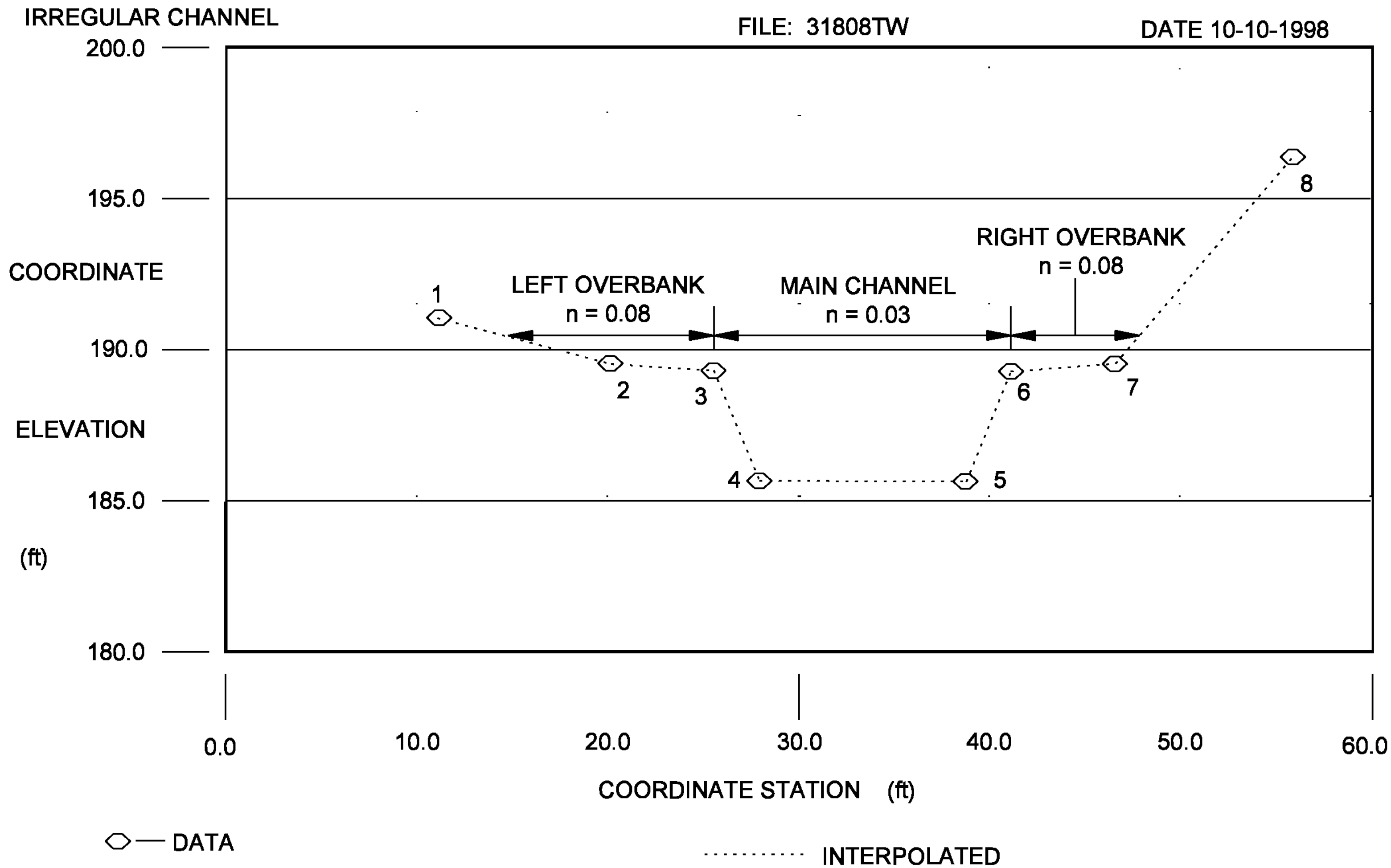
HY8 DATA INPUT PROMPT SCREEN

Figure 31-8A

```
IRREGULAR CHANNEL CROSS-SECTION

CROSS-SECTION    X        Y
COORD. NO.      (ft)     (ft)
       1         12.3    194.0
       2         22.3    193.0
       3         28.3    192.8
       4         31.0    190.4
       5         43.0    190.4
       6         45.7    192.8
       7         51.7    193.0
       8         62.0    197.6

<NUMBER>  TO EDIT COORDINATES
<I>  <D>  TO INSERT OR DELETE
<ENTER>   TO CONTINUE
<P>       TO PLOT CROSS-SECTION

1-Help  2-Prog  3       4       5-End   6       7       8       9-DOS  10
```

HY8 CHANNEL DATA PROMPT SCREEN

Figure 31-8B



CHANNEL CROSS SECTION

Figure 31-8C

CULVERT FILE: EX31803 FHWA CULVERT ANALYSIS DATE: 10-06-98
TAILWATER FILE: 3180 HY8, VERSION 6.0 CULVERT NO. 1 OF 1

******* UNIFORM FLOW RATING CURVE FOR DOWNSTREAM CHANNEL

FLOW (ft^3/s)	W.S.E (ft)	FROUDE NUMBER	DEPTH (ft)	VEL. (ft/s)	SHEAR (lb/ft^2)
0.00	190.4	0.000	0.00	0.00	0.00
22.2	191.5	0.267	1.17	1.57	0.06
44.5	192.1	0.280	1.73	1.97	0.09
66.7	192.6	0.287	2.20	2.27	0.11
89.0	192.9	0.293	2.57	2.47	0.12
111.2	193.2	0.299	2.86	2.70	0.14
133.5	193.5	0.304	3.13	2.90	0.15
148.3	193.7	0.307	3.30	3.00	0.16
178.0	194.0	0.312	3.60	3.20	0.18
200.2	194.0	0.312	3.63	3.23	0.18
222.5	194.0	0.312	3.63	3.23	0.18

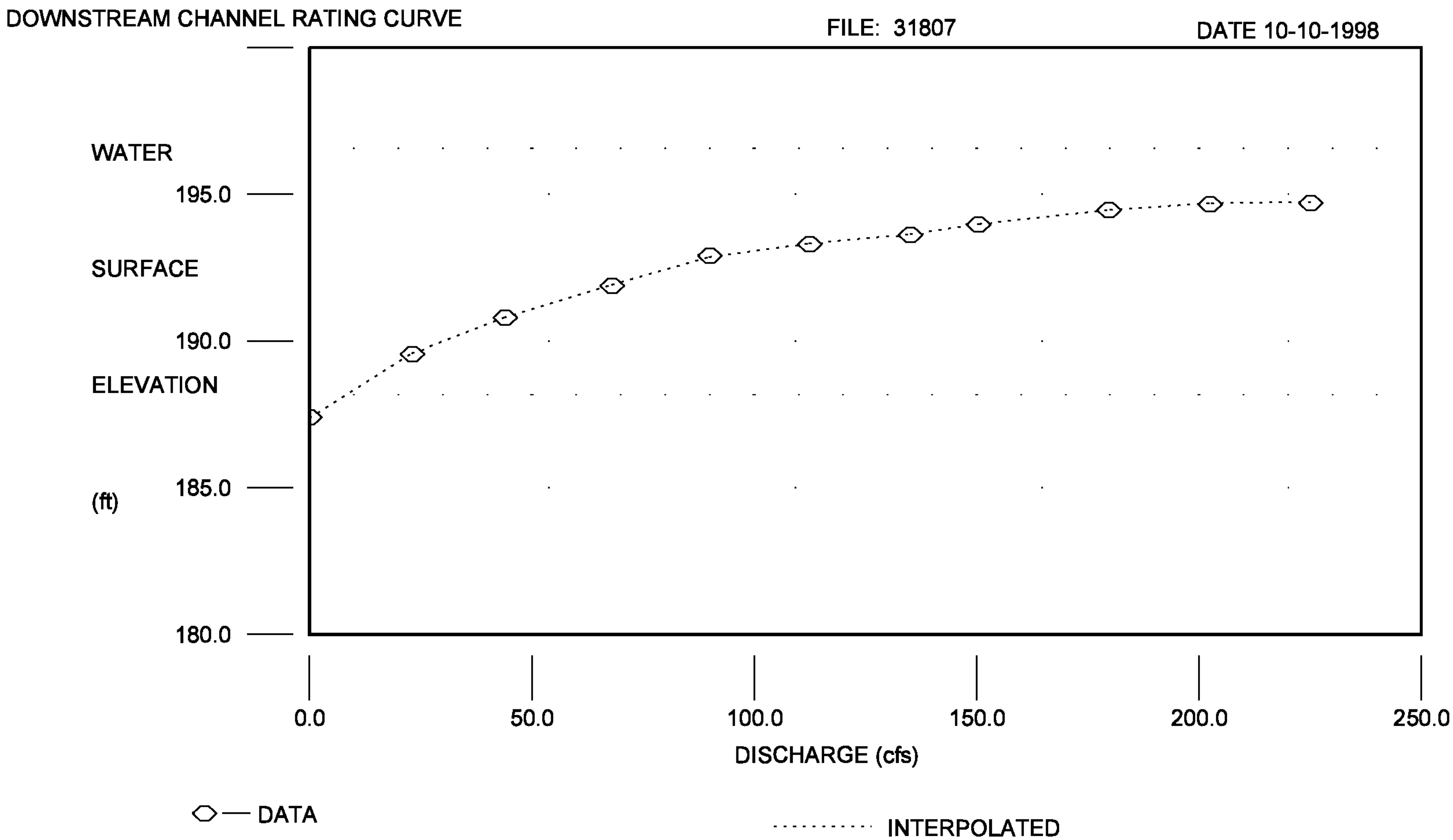
Note: Shear stress was calculated using R.

PRESS <D> FOR DATA
<P> TO PLOT RATING CURVE
<ESC> FOR CHANNEL SHAPE MENU
<ENTER>TO CONTINUE

1-Help 2-Progr 3 4 5-End 6 7 8 9-DOS 10

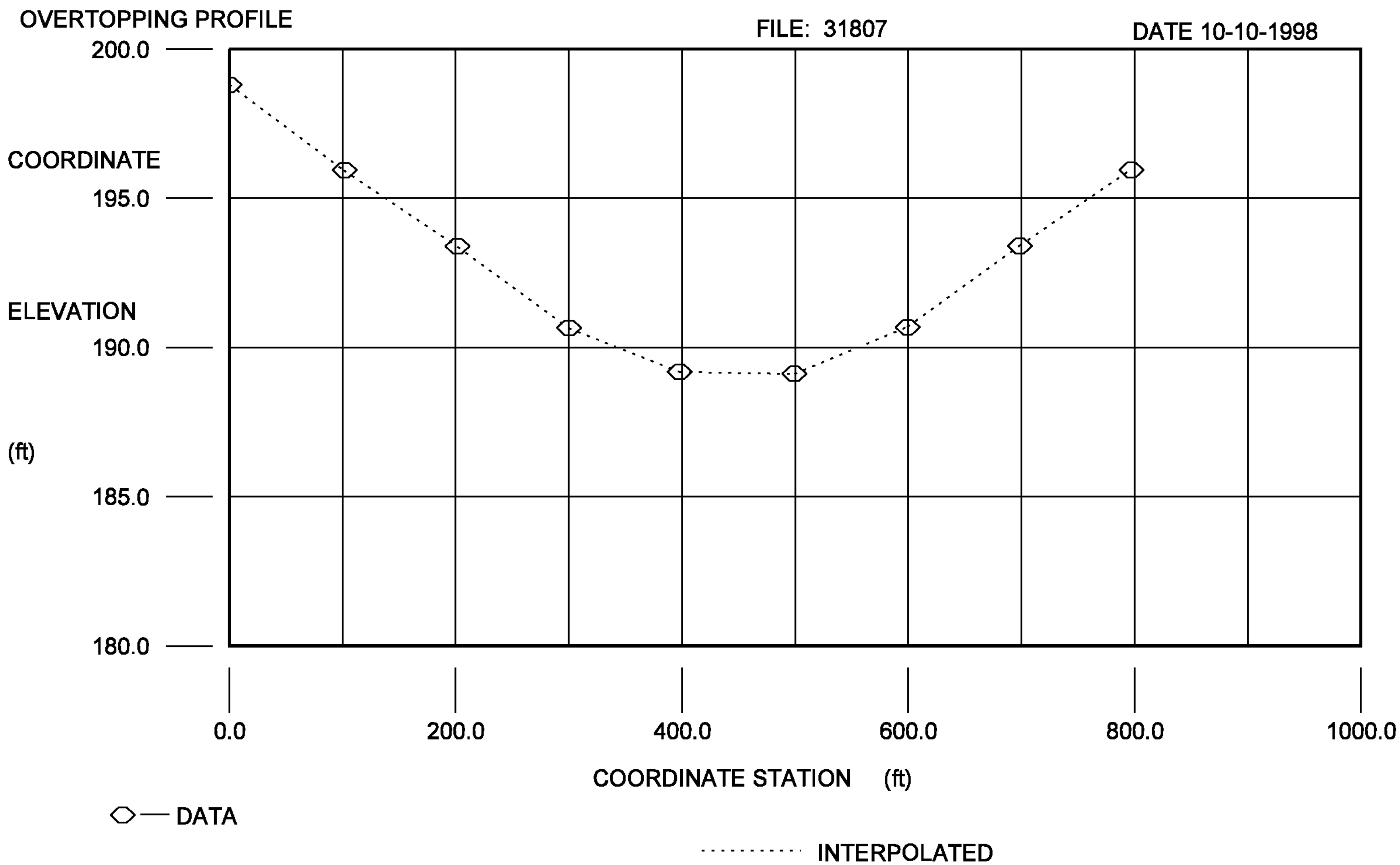
HY8 RATING CURVE PROMPT SCREEN

Figure 31-8D



TAILWATER VS. FLOWRATE

Figure 31-8E



ROADWAY PROFILE

Figure 31-8F

```
CURRENT DATE: 10-12-1998                        FILE DATE: 10-12-1998
CURRENT TIME: 11:34:41                          FILE NAME: 31833
                          FHWA CULVERT ANALYSIS
                           HY-8, VERSION 6.0
```

	SITE DATA			CULVERT SHAPE, MATERIAL, INLET				
CULV. NO.	INLET ELEV. (ft)	OUTLET ELEV. (ft)	CULVERT LENGTH (ft)	BARRELS SHAPE MATERIAL	SPAN (in.)	RISE (in.)	MANNING n	INLET TYPE
1	190.7	190.4	300	1 RCB	132	48	0.012	CONVENTIONAL
2								
3								
4								
5								
6								

```
                 OPTION MENU:      PRESS &lt;LETTER&gt;

      CULVERT FILE                 SINGLE CULVERT (NO OVERTOPPING)
      &lt;C&gt; Create                   &lt;S&gt; Calculate
      &lt;E&gt; Edit                     &lt;M&gt; Minimize Width
      &lt;N&gt; Name                     &lt;R&gt; Report - Display or Print
      &lt;D&gt; Directory                &lt;F&gt; File - Save PC and LST files

      AVAILABLE FILES              MULTIPLE CULVERTS & OVERTOPPING
      Culvert:   EX31803.INP       &lt;O&gt; Overtopping
       Output:   EX31803.PC        &lt;R&gt; Report - Display or Print
       Report:   None              &lt;L&gt; List - Save PC and LST files

      DESIGN OPTIONS               DEFAULT OPTIONS
      &lt;H&gt; Hydrograph               &lt;U&gt; Units Used - English
      &lt;I&gt; Routing                  &lt;W&gt; Outlet Control - Profiles
      &lt;J&gt; Dissipator               &lt;P&gt; Paths for files & Defaults

                   &lt;Enter&gt; for Documentation Menu
                       &lt;Q&gt; Quit

 1-Help   2-Progr 3      4      5-End   6      7      8      9-DOS   10
```

HY8 DATA SUMMARY PROMPT SCREEN

Figure 31-8G

PERFORMANCE CURVE FOR CULVERT 1 – One (132 in. by 48 in.) RCB

DIS-CHARGE FLOW (ft^3/s)	HEAD-WATER ELEV. (ft)	INLET CONTROL DEPTH (ft)	OUTLET CONTROL DEPTH (ft)	FLOW TYPE <F4>	NORMAL DEPTH (ft)	CRIT. DEPTH (ft)	OUTLET DEPTH (ft)	TW DEPTH (ft)	OUTLET VEL. (ft/s)	TW VEL. (ft/s)
0.00	190.7	0.00	-0.30	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
22.2	191.7	0.77	1.03	3-M1t	0.70	0.53	1.17	1.17	1.83	1.57
44.5	192.3	1.23	1.67	3-Mlt	1.13	0.83	1.73	1.73	2.43	1.97
66.7	192.8	1.60	2.17	3-Mlt	1.47	1.07	2.20	2.20	2.90	2.27
89.0	193.3	1.93	2.60	3-Mlt	1.77	1.30	2.57	2.57	3.27	2.47
111.2	193.7	2.27	3.03	3-Mlt	2.07	1.50	2.87	2.87	3.70	2.70
133.5	194.0	2.57	3.37	3-Mlt	2.33	1.70	3.13	3.13	4.07	2.90
148.3	194.2	2.80	3.50	3-Mlt	2.50	1.83	3.30	3.30	4.27	3.00
178.0	194.6	3.17	3.87	3-Mlt	2.83	2.07	3.60	3.60	4.70	3.20
200.2	194.7	3.47	4.07	3-Mlt	3.07	2.23	3.63	3.63	5.27	3.23
222.5	194.9	3.73	4.20	3-Mlt	3.30	2.40	3.63	3.63	5.83	3.23

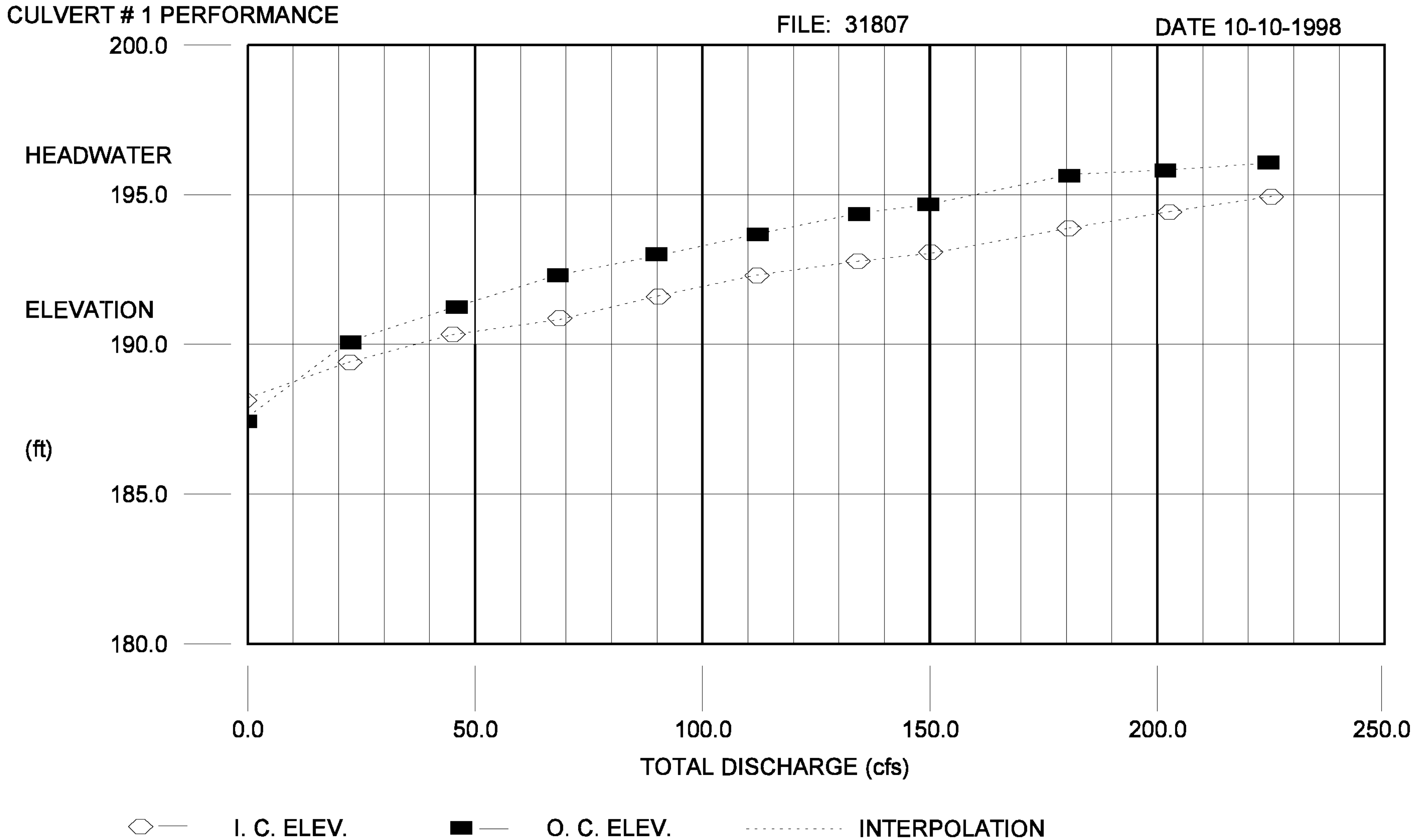
El. inlet face invert 190.7 ft El. outlet invert 190.4 ft
El. inlet throat invert 0.00 ft El. inlet crest 0.00 ft

PRESS: <KEY> TO CONTINUE <W> FOR PROFILE TABLE
<P> TO PLOT <I> FOR IMPROVED INLET TABLE

1- Help 2 3 4-Type 5-End 6 7 8 9-DOS 10

HY8 PERFORMANCE CURVE PROMPT SCREEN
132 in SPAN BY 48 in RISE CULVERT

Figure 31-8H



INLET/OUTLET CONTROL HEADWATERS

Figure 31-8 I

PERFORMANCE CURVE FOR CULVERT 1 – One (144 in. by 48 in.) RCB

DIS-CHARGE FLOW (ft3/s)	HEAD-WATER ELEV. (ft)	INLET CONTROL DEPTH (ft)	OUTLET CONTROL DEPTH (ft)	FLOW TYPE <F4>	NORMAL DEPTH (ft)	CRIT. DEPTH (ft)	OUTLET DEPTH (ft)	TW DEPTH (ft)	OUTLET VEL. (ft/s)	TW VEL. (ft/s)
0.00	190.7	0.00	-0.30	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
22.2	191.7	0.73	1.03	3-M1t	0.67	0.50	1.17	1.17	1.67	1.57
44.5	192.3	1.17	1.63	3-Mlt	1.03	0.77	1.73	1.73	2.23	1.97
66.7	192.8	1.50	2.13	3-Mlt	1.37	1.00	2.20	2.20	2.67	2.27
89.0	193.2	1.83	2.57	3-Mlt	1.67	1.23	2.57	2.57	3.00	2.47
111.2	193.6	2.13	2.90	3-Mlt	1.93	1.43	2.87	2.87	3.34	2.70
133.5	193.9	2.43	3.23	3-Mlt	2.17	1.60	3.13	3.13	3.73	2.90
148.3	194.1	2.63	3.42	3-Mlt	2.33	1.73	3.30	3.30	3.93	3.00
178.0	194.5	2.97	3.87	3-Mlt	2.63	1.97	3.60	3.60	4.30	3.20
200.2	194.6	3.23	3.93	3-Mlt	2.87	2.10	3.63	3.63	4.83	3.23
222.5	194.7	3.50	4.07	3-Mlt	3.07	2.27	3.63	3.63	5.37	3.23

El. inlet face invert 190.7 ft El. outlet invert 190.4 ft
El. inlet throat invert 0.00 ft El. inlet crest 0.00 ft

PRESS: <KEY> TO CONTINUE <W> FOR PROFILE TABLE
<P> TO PLOT <I> FOR IMPROVED INLET TABLE

1- Help 2 3 4-Type 5-End 6 7 8 9-DOS 10

HY8 PERFORMANCE CURVE PROMPT SCREEN
144 in SPAN BY 48 in RISE CULVERT

Figure 31-8J

PERFORMANCE CURVE FOR CULVERT 1 – One (84 in. by 48 in.) RCB

DIS-CHARGE FLOW (ft^3/s)	HEAD-WATER ELEV. (ft)	INLET CONTROL DEPTH (ft)	OUTLET CONTROL DEPTH (ft)	FLOW TYPE <F4>	NORMAL DEPTH (ft)	CRIT. DEPTH (ft)	OUTLET DEPTH (ft)	TW DEPTH (ft)	OUTLET VEL. (ft/s)	TW VEL. (ft/s)
0.00	190.7	0.00	-0.30	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
22.2	191.9	1.03	1.23	3-M1t	1.00	0.70	1.17	1.17	2.87	1.57
44.5	192.6	1.67	1.97	3-Mlt	1.60	1.10	1.73	1.73	3.83	1.97
66.7	193.2	2.17	2.57	3-Mlt	2.10	1.47	2.20	2.20	4.57	2.27
89.0	193.7	2.67	3.07	3-M2t	2.60	1.77	2.57	2.57	5.15	2.47
111.2	194.2	3.13	3.53	3-M2t	3.03	2.03	2.87	2.87	5.80	2.70
133.5	194.6	3.57	3.93	3-M2t	3.50	2.30	3.13	3.13	6.37	2.90
148.3	194.9	3.87	4.20	3-M2t	4.00	2.47	3.30	3.30	6.70	3.00
178.0	195.4	4.47	4.73	3-M2t	4.00	2.80	3.60	3.60	7.40	3.20
200.2	195.7	4.90	5.03	3-M2t	4.00	3.03	3.63	3.63	8.27	3.23
222.5	196.3	5.37	5.63	3-M2t	4.00	3.23	3.63	3.63	9.20	3.23

El. inlet face invert 190.7 ft   El. outlet invert 190.4 ft
El. inlet throat invert 0.00 ft   El. inlet crest 0.00 ft

PRESS: <KEY> TO CONTINUE <W> FOR PROFILE TABLE
<P> TO PLOT <I> FOR IMPROVED INLET TABLE

1- Help 2 3 4-Type 5-End 6 7 8 9-DOS 10

HY8 PERFORMANCE CURVE PROMPT SCREEN
84 in SPAN BY 48 in RISE CULVERT

Figure 31-8K

```
CULVERT FILE: HDS5EX1                                DATE: 10-10-1998
TAILWATER FILE:  AP-GDEX1                            CULVERT NO. 1 OF 1
                              FHWA CULVERT ANALYSIS
                               HY-8, VERSION 6.0
```

SUMMARY TABLE FOR FILE 2400

	<S> SITE DATA			<C> CULVERT SHAPE, MATERIAL, INLET				
CULV. NO.	INLET ELEV. (ft)	OUTLET ELEV. (ft)	CULVERT LENGTH (ft)	BARRELS SHAPE MATERIAL	SPAN (in.)	RISE (in.)	MANNING n	INLET TYPE
1	190.7	190.4	300	1 RCB	84	48	0.012	CONVENTIONAL
2								
3								
4								
5								
6								

```
HEADWATER ELEVATION (ft)
  ENTER ALLOWABLE    =   195.0
  CONTROLLING        =   194.9
  INLET CONTROL      =   194.5
  OUTLET CONTROL     =   194.9

FLOW VELOCITY (ft/s)
  V CULVERT          =   6.73
  V CHANNEL          =   3.00
  Q (ft3/s)          =   148.3
  SLOPE              =   0.0010

FLOW DEPTHS (ft)
  CULVERT            =   3.30
  CHANNEL            =   3.30
  NORMAL             =   4.00
  CRITICAL           =   2.47

MAXIMUM HEADWATER          <ENTER> TO RETURN
                           <H> TO CHANGE HEADWATER
                           <S> TO SAVE FILE

1       2-      3       4       5-End   6       7       8       9       10
```

HY8 MINIMIZATION ROUTINE TABLE PROMPT SCREEN

Figure 31-8L

HY8, VERSION 6.0

SUMMARY OF CULVERT FLOWS (ft3/s)   FILE: 31808   DATE: 10-12-1998

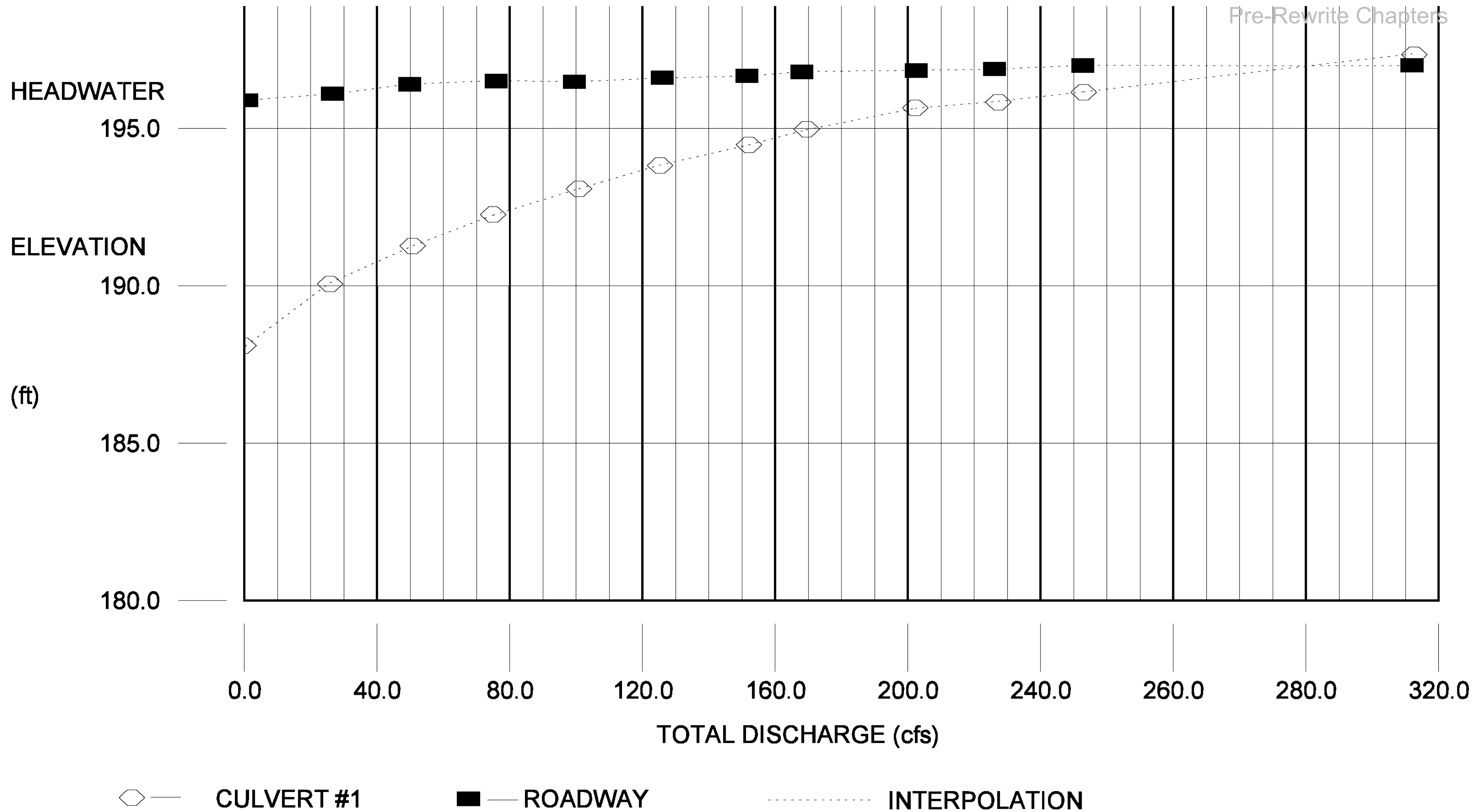
ELEV(ft)	TOTAL	1	2	3	4	5	6	ROADWAY	ITER
190.7	0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	1
191.7	21.2	21.2	0.00	0.00	0.00	0.00	0.00	0.00	1
192.3	45.9	45.9	0.00	0.00	0.00	0.00	0.00	0.00	1
192.8	67.1	67.1	0.00	0.00	0.00	0.00	0.00	0.00	1
193.2	88.3	88.3	0.00	0.00	0.00	0.00	0.00	0.00	1
193.6	113.0	109.5	0.00	0.00	0.00	0.00	0.00	0.00	1
193.9	134.2	134.2	0.00	0.00	0.00	0.00	0.00	0.00	1
194.1	148.3	148.3	0.00	0.00	0.00	0.00	0.00	0.00	1
194.5	176.6	176.6	0.00	0.00	0.00	0.00	0.00	0.00	1
194.6	201.3	201.3	0.00	0.00	0.00	0.00	0.00	0.00	4
194.7	222.5	215.4	0.00	0.00	0.00	0.00	0.00	7.42	6
194.6	201.3	201.3	0.00	0.00	0.00	0.00	0.00	OVERTOPPING	

PRESS: <P> TO PLOT TOTAL RATING CURVE
<T> TO DISPLAY TABLE FOR EACH CULVERT
<E> TO DISPLAY ERROR TABLE
<R> TO PRINT REPORT   Output stored in HDS5EX1.PC
<H> TO RETURN TO HEADWATER TABLE
<ENTER> TO RETURN TO OPTION MENU

1-Help   2-Progr   3-Time   4   5-End   6   7   8   9-DOS   10

HY8 SUMMARY OF CULVERT FLOWS PROMPT SCREEN

Figure 31-8M



TOTAL PERFORMANCE CURVE

Figure 31-8N

OVERTOPPING PERFORMANCE CURVE FOR CULVERT 1 – One (144 in. by 48 in.) RCB

DIS-CHARGE FLOW (ft3/s)	HEAD-WATER ELEV. (ft)	INLET CONTROL DEPTH (ft)	OUTLET CONTROL DEPTH (ft)	FLOW TYPE <F4>	NORMAL DEPTH (ft)	CRIT. DEPTH (ft)	OUTLET DEPTH (ft)	TW DEPTH (ft)	OUTLET VEL. (ft/s)	TW VEL. (ft/s)
0.00	190.7	0.00	-0.30	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
22.2	191.7	0.73	1.03	3-M1t	0.67	0.50	1.17	1.17	1.67	1.57
44.5	192.3	1.17	1.63	3-Mlt	1.03	0.77	1.73	1.73	2.23	1.97
66.7	192.8	1.50	2.13	3-Mlt	1.37	1.00	2.20	2.20	2.67	2.27
89.9	193.2	1.83	2.57	3-Mlt	1.67	1.23	2.57	2.57	3.00	2.47
111.2	193.6	2.13	2.90	3-Mlt	1.93	1.43	2.87	2.87	3.67	2.70
133.4	193.9	2.43	3.23	3-Mlt	2.17	1.60	3.13	3.13	3.73	2.90
148.3	194.1	2.63	3.43	3-Mlt	2.33	1.73	3.30	3.30	3.93	3.00
178.0	194.5	2.97	3.87	3-Mlt	2.63	1.97	3.60	3.60	4.33	3.20
200.0	194.6	3.23	3.93	3-Mlt	2.87	2.10	3.63	3.63	4.80	3.23
214.0	194.7	3.40	4.00	3-Mlt	3.00	2.20	3.63	3.63	5.17	3.23

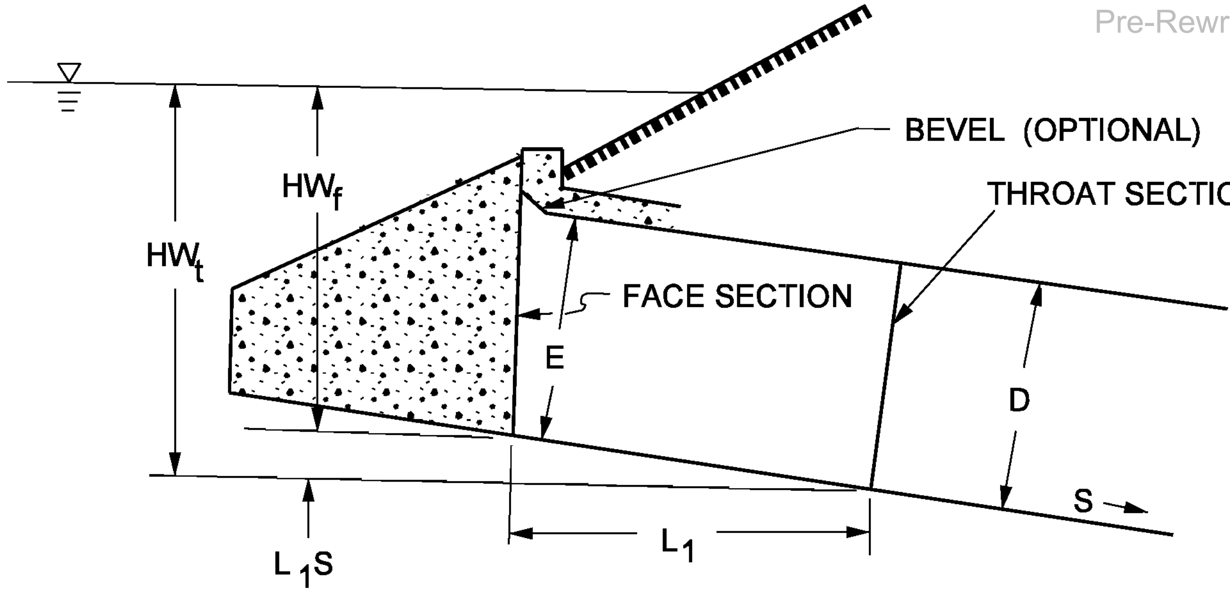
El. inlet face invert 190.7 ft El. outlet invert 190.4 ft
El. inlet throat invert 0.00 ft El. inlet crest 0.00 ft

PRESS: <KEY> TO CONTINUE <W> FOR PROFILE TABLE
<P> TO PLOT <I> FOR IMPROVED INLET TABLE

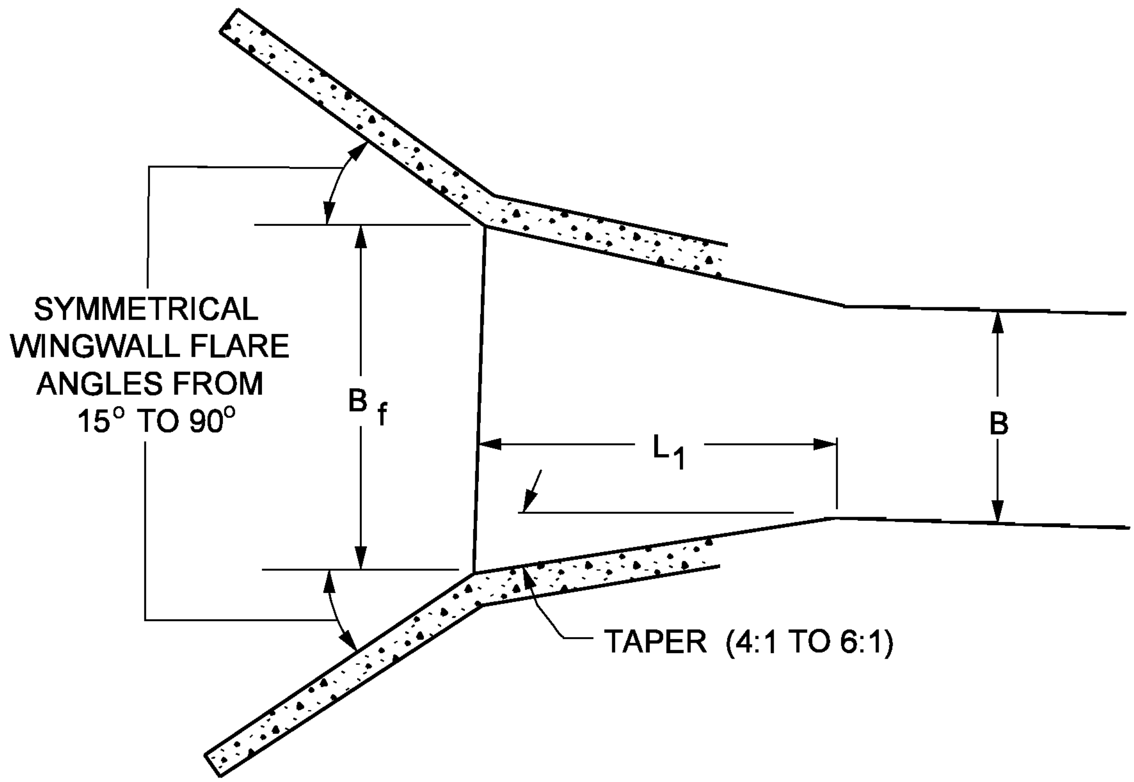
1- Help 2 3 4-Type 5-End 6 7 8 9-DOS 10

HY8 OVERTOPPING PERFORMANCE CURVE PROMPT SCREEN
144 in SPAN BY 48 in RISE CULVERT

Figure 31-8 O



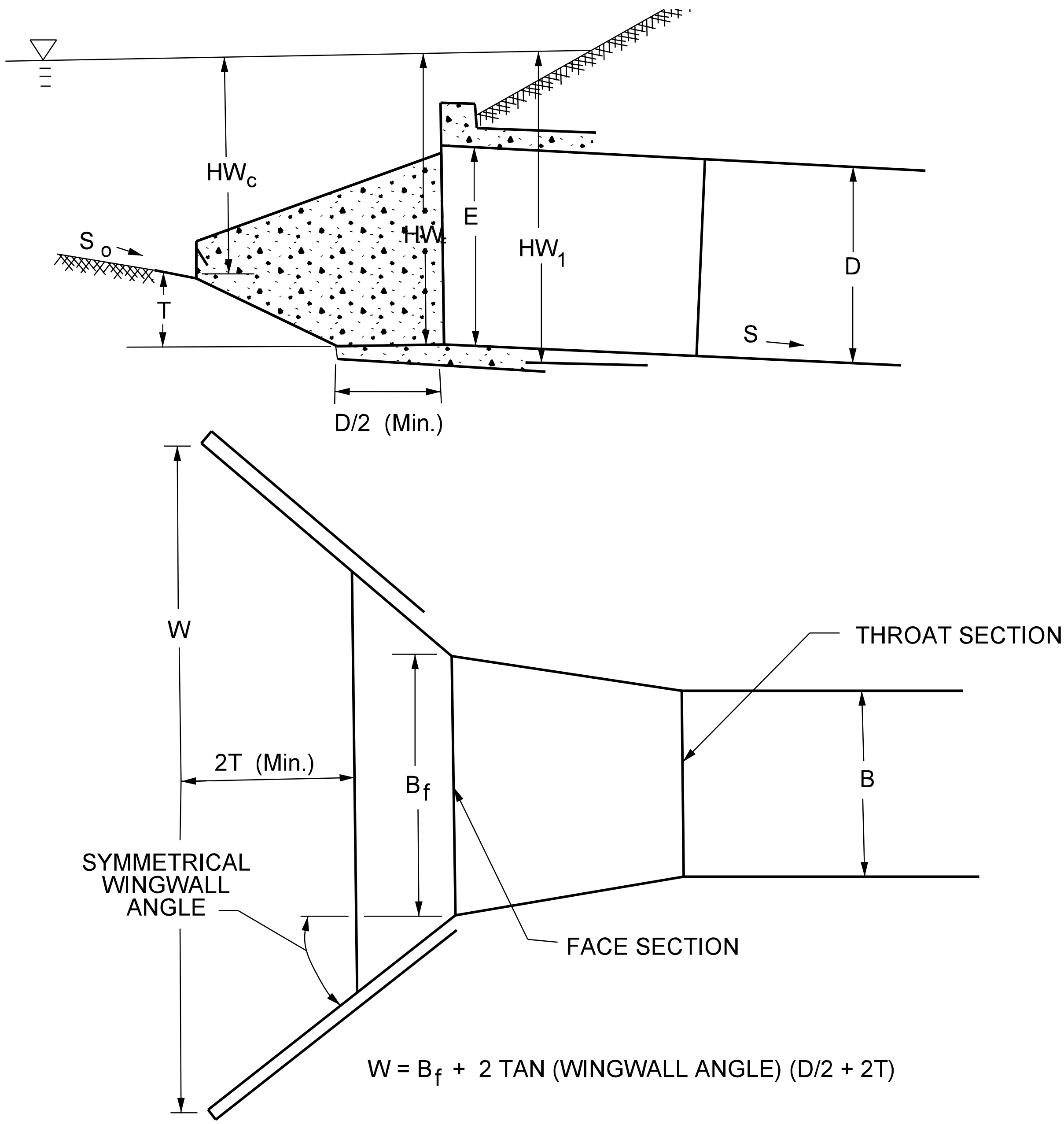
ELEVATION



PLAN

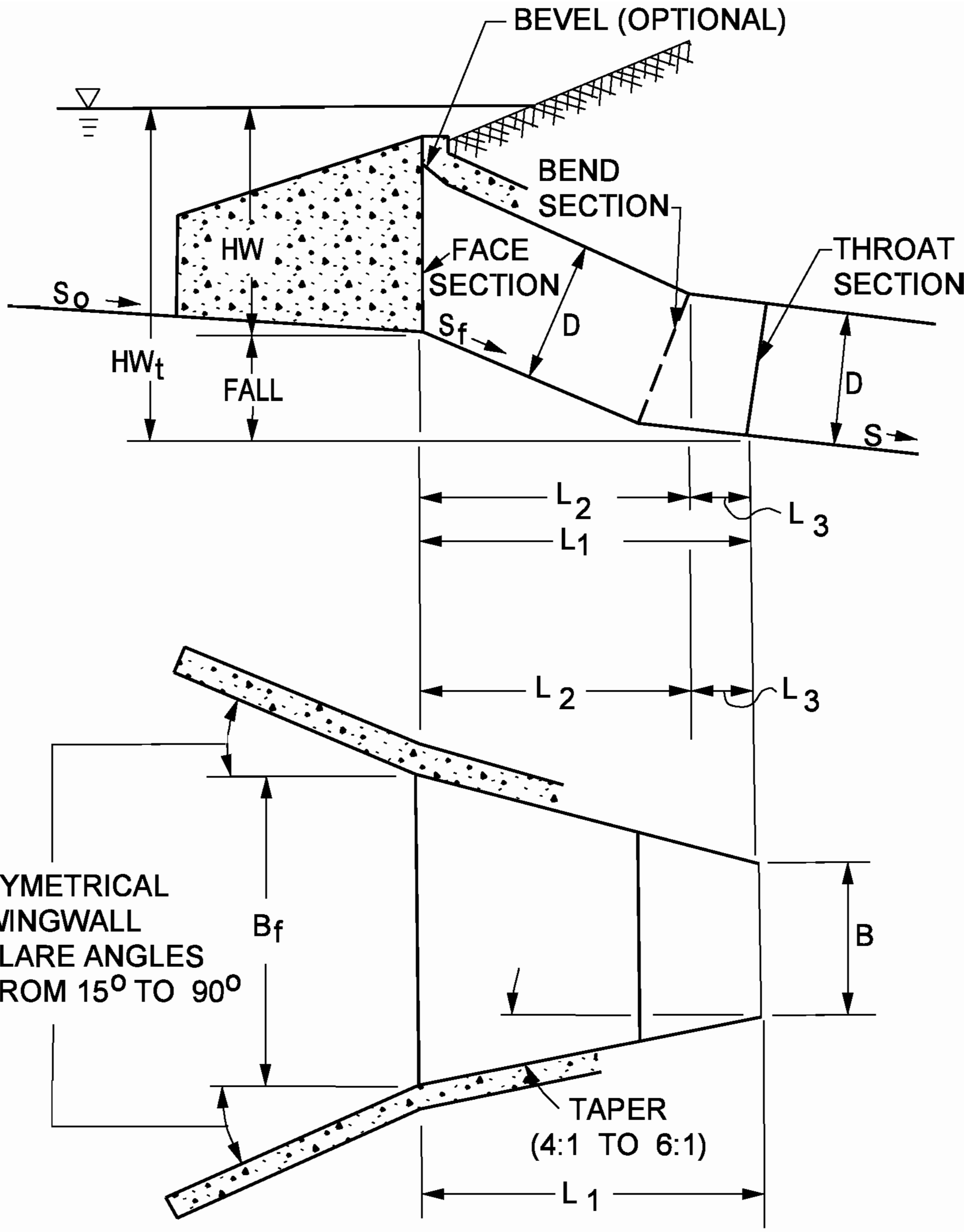
SIDE-TAPERED END TREATMENT

Figure 31-9A



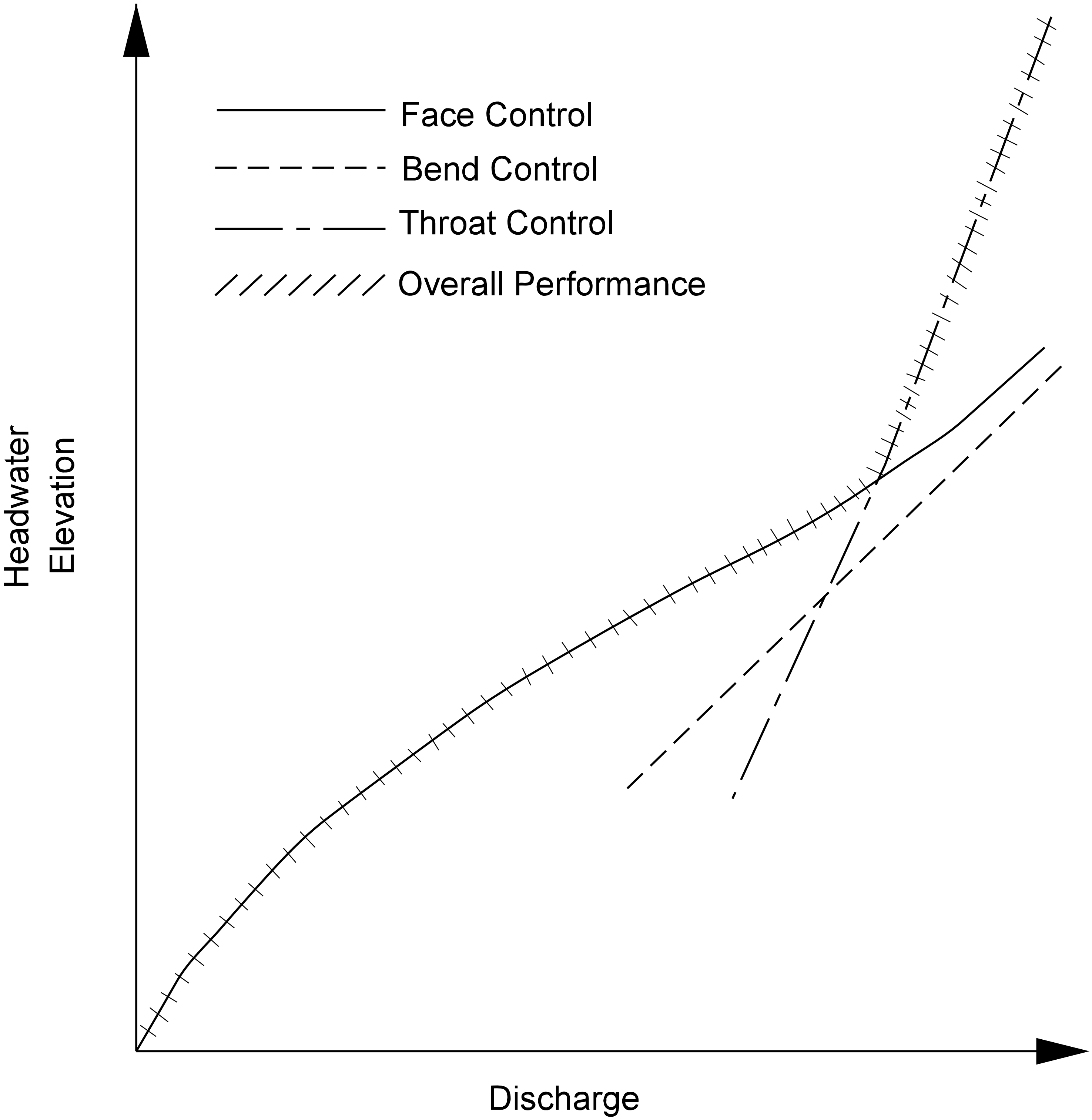
SIDE-TAPERED END TREATMENT
(Upstream Depression Contained Between Wingwalls)

Figure 31-9B



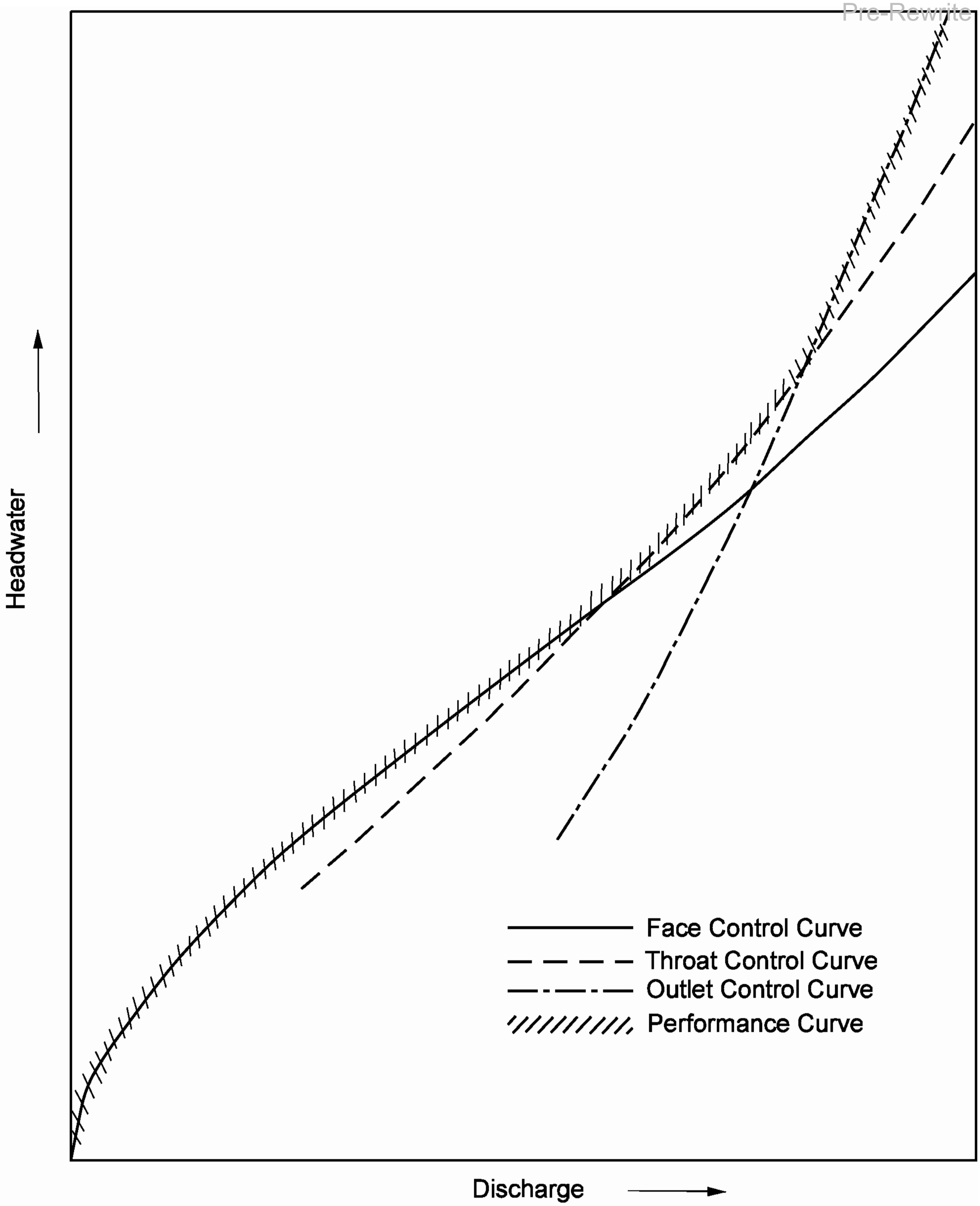
SLOPE-TAPERED END TREATMENT WITH VERTICAL FACE

Figure 31-9C



INLET CONTROL PERFORMANCE CURVES
(Schematic)

Figure 31-9D



CULVERT PERFORMANCE CURVE
(Schematic)

Figure 31-9E

Type of Conduit	Wall Description	Manning's *n*
Concrete Pipe	Smooth Interior	0.012
Concrete Box	Smooth Walls	0.012 - 0.015
Corrugated Metal Pipe or Box, Annular or Helical Pipe (see HDS #5)	2.75 in. x 0.5 in. corrugations 6 in. x 1 in. corrugations 5 in. x 1 in. corrugations 3 in. x 1 in. corrugations 6 in. x 2 in. structural plate 9.25 in. x 2.5 in. structural plate	0.024 0.024 0.024 0.024 0.033 - 0.035 0.033 - 0.037
Thermoplastic Pipe	Smooth Interior	0.012

Note 1: *The value indicated in this table is the recommended Manning's* n *design value. The actual field value for an older, existing pipeline may vary depending on the effects of abrasion, corrosion, deflection and joint conditions. A concrete pipe with poor joints and deteriorated walls may have an* n *value of 0.014 to 0.018. A corrugated metal pipe with joint and wall problems may also have a higher* n *value, and may experience shape changes which can adversely affect the general hydraulic characteristics of the culvert.*

Note 2: *For further information concerning Manning's* n *value for selected conduits, consult Hydraulic Design of Highway Culverts, Federal Highway Administration, HDS #5, p. 163.*

RECOMMENDED MANNING'S *n* VALUE

Figure 31-10A

$$H_e = K_e \left[\frac{V^2}{2g} \right]$$

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, square cut end	0.5
Headwall or headwall and wingwalls	
Square-edge	0.5
Rounded (radius = 1/12*D*)	0.2
Socket end of pipe (groove-end)	0.2
Projecting from fill, socket end (groove-end)	0.2
Beveled edges, 33.7-deg or 45-deg 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-deg or 45-deg 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 deg to 25 deg, or 30 deg to 75 deg 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 deg to 75 deg 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

* *An end section conforming to fill slope, made of either metal or concrete, is the section commonly available from manufacturers. From limited hydraulic tests, it is equivalent in operation to a headwall in both inlet and outlet control. An end section incorporating a closed taper in its design may have a superior hydraulic performance. Such a section can be designed using the information shown for the beveled inlet.*

ENTRANCE-LOSS COEFFICIENTS
(Outlet Control, Full or Partly Full)

Figure 31-10B

End-Treatment Type	Entrance Type	K_E
Grated Box End Section, Type 1	Concrete Pipe, headwall with square edge	0.5
Grated Box End Section, Type 2	Concrete Pipe, headwall with square edge	0.5
Multiple-Pipes Concrete Anchor	Concrete Pipe, projecting from fill, square cut end	0.5
Multiple-Pipes Concrete Anchor	Corrugated Metal Pipe, Projecting from fill	0.9
Metal Pipe End Section	Corrugated Metal Pipe, end section conforming to fill slope	0.5
Precast-Concrete End Section	Concrete Pipe, end section conforming to fill slope	0.5
Safety Metal End Section	Corrugated Metal Pipe, mitered to conform to fill slope	0.7
Safety Metal End Section	Corrugated Metal Pipe, end section conforming to fill slope	0.5
Safety Metal End Section	Corrugated Metal Pipe, mitered to conform to fill slope	0.7
Safety Metal End Section	Corrugated Metal Pipe, end section conforming to fill slope	0.5
Single-Pipe Concrete Anchor	Corrugated Metal Pipe, projecting from fill	0.9
Single-Pipe Concrete Anchor	Concrete Pipe, projecting from fill, square cut end	0.5
Single-Pipe Concrete Anchor	Corrugated Metal Pipe-Arch, projecting from fill	0.9
Multiple-Pipe Concrete Anchor	Concrete Pipe-Arch, projecting from fill, square cut end	0.5
Multiple-Pipe Concrete Anchor	Corrugated Metal Pipe-Arch, projecting from fill	0.9

ENTRANCE-LOSS COEFFICIENT, K_E,
FOR STANDARD INDOT CULVERT

Figure 31-10C

ROUTE: DES NO.:
PROJECT NO.:
PROJECT DESCRIPTION:

STATION:
SHEET OF

CULVERT DESIGN FORM

DESIGNER: DATE:
REVIEWER: DATE:

HYDROLOGICAL DATA

METHOD:
DRAINAGE AREA: mi^2 STREAM SLOPE:
CHANNEL SHAPE::
ROUTING: OTHER:

DESIGN FLOWS / TAILWATER

R.I. (years) FLOW (ft^3/s) TW (ft)

See Figure 31-10D(1) for culvert-design details

CULVERT DESCRIPTION MATERIAL-SHAPE-SIZE-ENTRANCE	TOT. FLOW Q (ft^3/s)	FLOW PER BAR-REL Q/N (1)	HEADWATER CALCULATIONS											CTRL. HEAD-WTR. ELEV.	OUTL. VELO-CITY	COMMENTS
			INLET CONTROL				OUTLET CONTROL									
			HW_i D (2)	HW_i	FALL (3)	EL_{hi} (4)	TW (5)	d_c	$\frac{d + D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)			

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERT

(2) HW_i /D = HW/D OR HW_f/D FROM DESIGN CHARTS

(3) FALL = HW_i - (EL_{hd} - EL_{sf}); FALL IS ZERO FOR CULVERT ON GRADE
EL_{hi} = HW_i + EL_i

(4) INVERT OF INLET CONTROL SECTION

(5) TW BASED ON DOWNSTREAM CONTROL OR FLOW DEPTH IN CHANNEL

(6) h_o = TW or (d_c + D)/2 (WHICHEVER IS GREATER)

(7) $$H = \frac{V}{2g}\left[1 + k_e + \left(\frac{19.63n^2 L}{R^{1.33}}\right)\right]$$

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

SUSCRIPT DEFINTIIONS
- a Approximate
- f Culvert Face
- h_d Design Headwater
- h_i Headwater in Inlet Control
- h_o Headwater in Outlet Control
- i Inlet Control Section
- o Outlet
- s_f Streambed at Culvert Face
- TW Tailwater

COMMENTS / DISCUSSION:

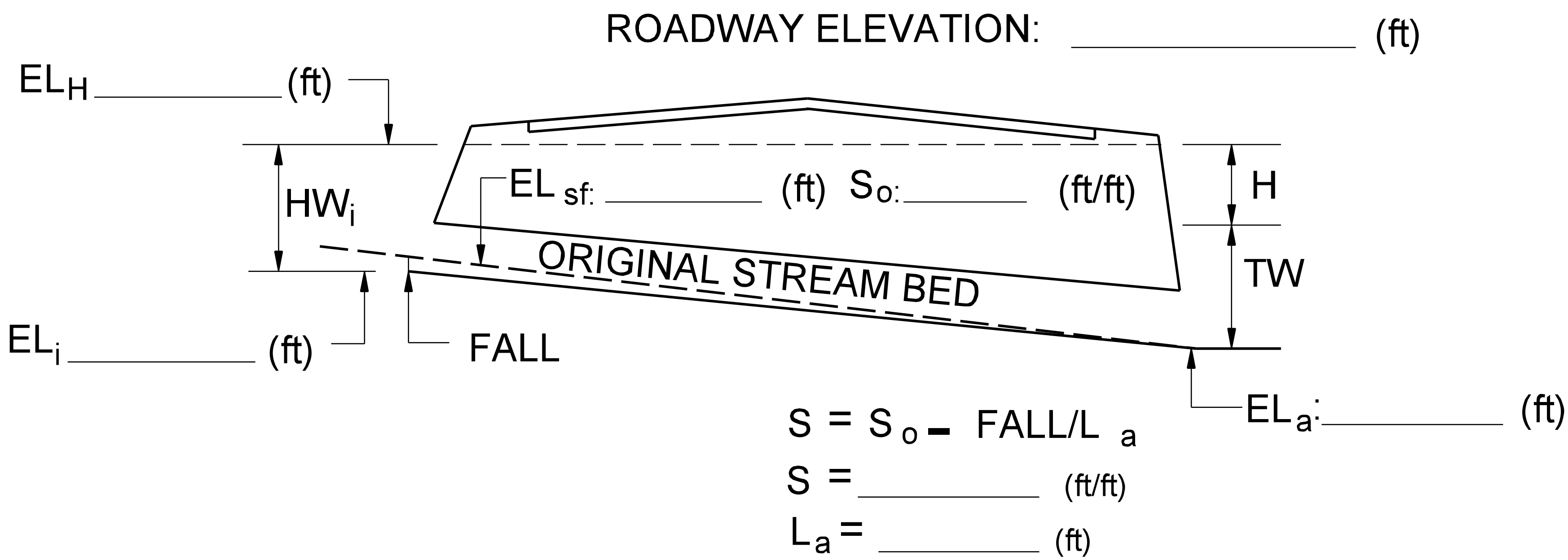
CULVERT BARREL SELECTED:

SIZE:

SHAPE:

MATERIAL: n:

ENTRANCE:



CULVERT DESIGN DETAILS
(Conventional End Treatment)

Figure 31-10D(1)

ROUTE: DES. NO.:
PROJECT NO.:
PROJECT DESCRIPTION:

STATION:

SHEET OF

CULVERT DESIGN FORM – TAPERED INLET

DESIGNER: DATE:
REVIEWER: DATE:

DESIGN DATA:

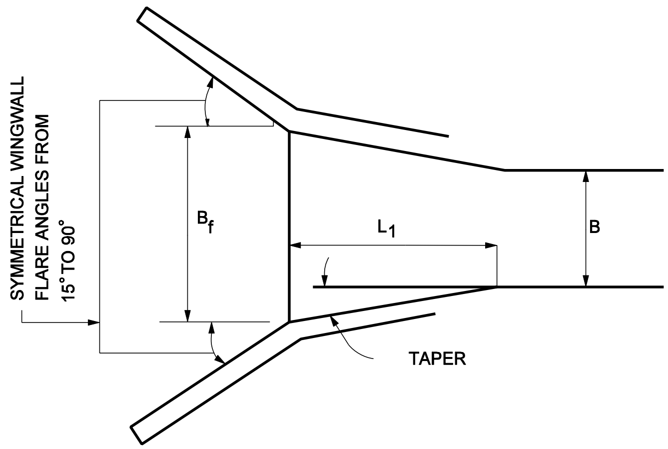
Q = ft³/s; $EL._{hl}$ (ft)
EL. Throat Invert (ft)
EL. Stream Bed at Face (ft)
FALL: (ft); TAPER :1 (4H:1V to 6H:1V)
STREAM SLOPE, S_c = (ft/ft)
SLOPE OF BARREL, S = (ft/ft)
S_f :1 (2H:1V to 3H:1V)
Barrel Shape and Material:
N = ; B = ; D =
Inlet Edge Description:

COMMENTS:

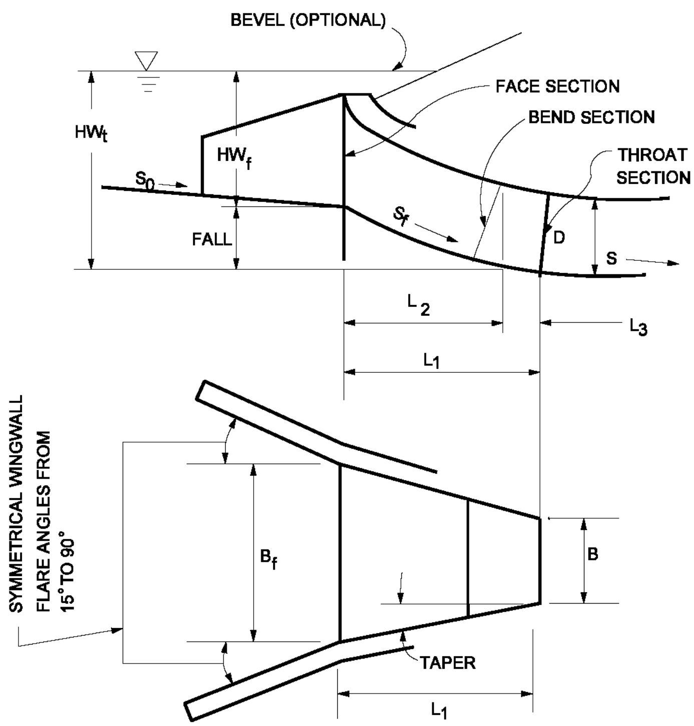
									SLOPE-TAPERED ONLY						SIDE-TAPERED w/ FALL		
Q (ft^3/s)	$EL._{hl}$	EL. Throat Invert	EL. Face Invert (1)	HW_f (2)	$\frac{HW_f}{E}$ (3)	$\frac{Q}{B_f}$ (4)	Min. B_f (5)	Sel-Ected B_f	Min. L_3 (6)	L_2 (7)	Check L_2 (8)	Adj. L_3 (9)	Adj. Taper (10)	L_1 (11)	EL. Crest Invert	HW_c (12)	Min. W (13)

TECHNICAL FOOTNOTES:
(1) Slope-Tapered: EL. Face Invert = EL. Stream Bed at Face
Side-Tapered: EL. Face Invert = EL. Throat Invert + 1 ft (approx.)
(2) $HW_f = EL._{hl}$ – EL. Face Invert
(3) $1.1D \geq E \geq D$
(4) From Design Charts
(5) Min. $B_f = Q / (Q/B_f)$
(6) Min. $L_3 = 0.5NB$
(7) $L_2 = S_f$(EL. Face Invert – EL. Throat Invert)
(8) Check $L_2 = 0.5(B_r - NB)Taper - L_3$
(9) If (8) > (7), Adj. $L_3 = 0.5(B_r - NB)Taper - L_3$
(10) If (7) ≤ (8), Adj. Taper = $\dfrac{L_2 + L_3}{0.5(B_r - NB)}$
(11) Slope-Tapered: $L_1 = L_2 + L_3$
Side-Tapered: $L = 0.5(B_r - NB)Taper$
(12) $HW_c = EL._{hl}$ – EL. Crest Invert
(13) Min. $W = \dfrac{0.634Q}{HW_c}$

SELECTED DESIGN:
B_f =
L_1 =
L_2 =
L_3 =
Bevels Angle = °
b =
d =
Taper = :1V
S_f = :1V



SIDE - TAPERED END TREATMENT
Figure 31-10E(1)



SLOPE - TAPERED END TREATMENT
Figure 31-10E(2)

ROUTE: DES. NO.:
PROJECT NO.:
PROJECT DESCRIPTION:

STATION:

SHEET OF

CULVERT DESIGN FORM – MITERED INLET

DESIGNER: DATE:
REVIEWER: DATE:

DESIGN DATA:

Q = ft^3/s; $EL._{hl}$ (ft)
EL. Throat Invert (ft)
EL. Stream Bed at Face (ft)
FALL: (ft); TAPER :1 (4H:1V to 6H:1V)
STREAM SLOPE, S_c = (ft/ft)
SLOPE OF BARREL, S = (ft/ft)
S_f :1 (2H:1V to 3H:1V)
Barrel Shape and Material:
N = ; B = ; D =
Inlet Edge Description:

COMMENTS:

Q (ft^3/s)	$EL._{hl}$	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)

TECHNICAL FOOTNOTES:

(1) $y = \left[\frac{(S_o - S_f) - 1}{(S_o - S_f)(S_f + 1)}\right] - D$

(2) EL. Face Invert = EL. Stream Bed at Crest – y
(3) $HW_f = EL._{hl}$ – EL. Face Invert
(4) $1.1D \geq E \geq D$
(5) From Design Charts
(6) Min. $B_f = Q / (Q/B_f)$
(7) Min. $L_3 = 0.5NB$
(8) $L^4 = yS_f + D/S_f$
(9) $L_2 = S_f$(EL. Crest Invert – EL. Throat Invert)
If negative, do not use mitered inlet.
(10) Check $L_2 = 0.5(B_r - NB)Taper - L_3$

(11) If (10) > (9), Adj. $L_3 = 0.5(B_r - NB)Taper - L_3$

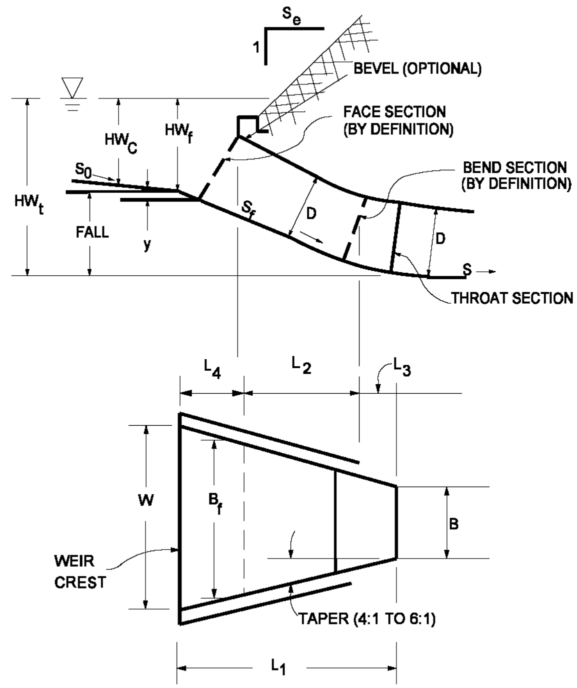
(12) If (9) ≥ (10), Adj. Taper = $\frac{L_2 + L_3}{0.5(B_r - NB)}$

(13) $L_1 = L_2 + L_3 + L_4$
(14) $HW_c = EL._{hl}$ – EL. Crest Invert

(15) Min. $W = \frac{0.634Q}{HW_c}$

(16) $W = NB + \left(\frac{2L_1}{Taper}\right)$ If W < Min. W, adjust taper.

SELECTED DESIGN:
B_f =
L_1 =
L_2 =
L_3 =
L_4 =
Bevels Angle = °
b =
d =
Taper = :1V
S_f = :1V



SLOPE-TAPERED INLET/MITERED FACE

Figure 31-10F(1)

TABLE OF CONTENTS

LIST OF FIGURES

CHAPTER THIRTY-TWO

BRIDGE HYDRAULICS

32-1.0 INTRODUCTION

32-1.01 Definition

A bridge is defined as follows:

1. a structure that transports traffic over a waterway, railroad, road or other obstruction;

2. a part of a stream-crossing system that includes the approach roadway over the floodplain, relief openings, and the bridge structure; and

3. legally, a structure with a total span of 20 ft or longer, measured along the centerline of the roadway. For a multiple-pipe structure, this includes the distance between the pipes. However, a structure designed hydraulically as a bridge as described above is treated in this Chapter, regardless of length.

See Figure 31-1A, Maximum Span Length for Culvert, for precise definitions on the measurement of span length to distinguish between a bridge and a culvert.

32-1.02 Analysis and Design

Proper hydraulic analysis and design is as vital as the structural design. A stream-crossing system should be designed for the following:

1. minimum cost subject to applicable criteria;
2. desired level of hydraulic performance up to an acceptable risk level;
3. mitigation of impacts on stream environment; and
4. accomplishment of social, economic, and environmental goals.

32-1.03 Purpose of Chapter

This Chapter includes the following:

1. guidance in the hydraulic design of a stream-crossing system through the following:

   a. appropriate policy and design criteria; and
   b. technical aspects of hydraulic design;

2. non-hydraulic factors that influence design including the following:

   a. environmental concerns;
   b. emergency access, traffic service; and
   c. consequences of catastrophic loss;

3. a design procedure which emphasizes hydraulic analysis using the computer programs WSPRO and HEC-2; and

4. a brief section on design philosophy. A more in-depth discussion is provided in the AASHTO *Highway Drainage Guidelines*, Chapter VII.

32-2.0 POLICY

32-2.01 General Policy

Policy is a set of goals or a plan of action. Federal and State policies that broadly apply to drainage design are provided in Chapter Twenty-eight. Policies that are unique to bridge crossings are provided herein.

The hydraulic analysis should consider a number of stream-crossing system designs to determine the most cost-effective proposal consistent with design constraints.

32-2.02 General INDOT Policy

The following general INDOT policy identifies specific areas for which quantifiable criteria can be developed.

1. The final design selection should consider the maximum backwater allowed by IDNR or INDOT. See Chapter Twenty-nine.

2. The final design should not significantly alter the flow distribution in the floodplain.

3. The crest-vertical curve profile should be considered as the preferred highway crossing profile in allowing for embankment overtopping at a lower discharge.

4. A 2-ft freeboard should be established to allow for passage of ice and debris. For a navigation channel, a vertical clearance in accordance with Federal or IDNR requirements should be established based on expected flows during the navigation season.

5. Degradation or aggradation of the river and contraction or local scour should be estimated. Appropriate positioning of the foundation, below the total scour depth if practical, should be included as part of the final design.

32-2.03 INDOT Bridge-Sizing Policy

The INDOT bridge-sizing policy is as follows.

1. Category 1. A bridge that does not require an Indiana Department of Natural Resources (IDNR) Permit.

2. Category 2. A bridge that does require an IDNR Permit.

A structure that requires an IDNR Permit (Certificate of Approval for Construction in a Floodway) includes the following:

1. structure with a drainage area greater than 50 mi^2 in a rural area; or
2. structure with a drainage area greater than 1 mi^2 in an urban area.

A rural area is defined in the Indiana Register, Volume 16, Number 6, March 1, 1993, p. 1527, as follows:

> *An area where the flood protection grade of each residential, commercial, or industrial building impacted by this project is higher than the regulatory flood elevation under the project control, and where the area lies outside:*
>
> *(i) the corporate boundaries of the consolidated city or an incorporated city or town; and*
>
> *(ii) the territorial authority for comprehensive planning established under IC 36-7-4-205(b).*

An area cannot be rural if it lies within a city or its planning zone, or if a finished floor elevation within the project backwater limits is below the Q_{100} elevation.

A project with a drainage area less than that listed above will not require a permit from IDNR. See Chapter Nine for more information on the IDNR Construction in a Floodway Permit.

For a new bridge on a new alignment, the maximum backwater should not exceed 1.5 in. The 1.5-in maximum may be modified as follows:

1. the backwater dissipates to 1.5 in or less at the right-of-way line;
2. the channel is sufficiently deep to contain the increased elevation without overtopping the banks; or
3. a flood easement may be purchased upstream of the bridge to allow for greater than 1.5 in. of backwater. The cost savings should be enough to offset the cost of the flood easement and the possible delay in constructing the project. Past experience has shown that this option may delay a project by one or more years.

The Hydraulics Engineer must approve an exception to the 1.5-in. backwater allowance for a new bridge on a new alignment.

For existing or baseline conditions, the IDNR limits surcharge to 1.25 in., urban or rural area. Existing conditions are defined as the water-surface profile that results from only those encroachments that have been in place since 1973. Although IDNR policy will allow for a slight increase over existing conditions, INDOT will not. INDOT policy for a bridge replacement or bridge rehabilitation is that the surcharge created by a proposed structure must be equal to or less than the existing surcharge, unless the existing surcharge is less than 1.25 in. This will allow future widening of the structure. If the surcharge created by an existing structure is greater than 12 in., the proposed surcharge for the bridge replacement or bridge rehabilitation project must not be greater than 12 in. above the natural channel flood profile.

FHWA does not require economic justification for a bridge that causes less than 12 in. of backwater. Therefore, a formal risk assessment will not be required.

32-3.0 DESIGN CRITERIA

Design criteria are the tangible means for placing accepted policies into action, and become the basis for the selection of the final design configuration of the stream-crossing system. Criteria are subject to change if conditions so dictate, or as approved by the Hydraulics Engineer.

32-3.01 General Criteria

The following are the AASHTO general criteria related to the hydraulic analysis for the location and design of a bridge, as stated in the *Highway Drainage Guidelines*.

1. Backwater will not significantly increase flood damage to property upstream of the crossing.

2. Velocity through the structure will not damage the highway facility nor increase damage to adjacent property.

3. Maintain the existing flow distribution as practical.

4. Provide pier spacing and orientation and abutments designed to minimize flow disruption and potential scour.

5. Provide foundation design or scour countermeasures to avoid failure due to scour.

6. Design pier spacing and freeboard at the structure to pass anticipated debris and ice.

7. Consider acceptable risks of damage or viable measures to counter the vagaries of an alluvial stream.

8. Consider minimal disruption of ecosystems and values unique to the floodplain and stream.

9. Provide a level of traffic service compatible with that expected for the class of highway and compatible with the projected traffic volume.

10. Design choices should support costs for construction, maintenance, and operation, including probable repair and reconstruction and potential liability that are affordable.

32-3.02 INDOT Criteria

The criteria described below and Figure 32-3A, Design-Storm Frequency, Bridge Waterway Opening, augment the general criteria described in Section 32-3.01. They provide specific, quantifiable values that relate to local site conditions. Evaluation of alternatives according to these criteria can be accomplished by using the water-surface profile programs WSPRO or HEC-2.

32-3.02(01) Roadway Serviceability

See Section 31-3.04(02).

32-3.02(02) Design Flood

A structure and its approach roadways should be designed, at a minimum, for the passage of the design-year flood specified for the required road serviceability for the highway classification system specified. By definition, a design flood will not overtop the roadway. As shown in Chapter Twenty-nine and Figure 32-3A, Design-Storm Frequency, Bridge Waterway Opening, the backwater must be calculated from the Q_{100} flood which may exceed the design flood.

32-3.02(03) Freeboard

Where practical, a minimum clearance of 2 ft should be provided between the design approach water-surface elevation and the low chord of the bridge for the final design alternative to allow for passage of ice and debris. Where this is not practical, the clearance should be established by the designer based on the type of stream and level of protection desired as approved by INDOT. For example, 1 ft should be adequate for a small stream that normally does not transport drift. An urban bridge with a grade limitation may provide no freeboard. A 3 ft freeboard is desirable for a major river which is known to carry large debris. The crest vertical curve profile is the preferred highway crossing profile in allowing for embankment overtopping at a lower discharge.

32-3.02(04) Span Lengths

The minimum span length for a bridge with more than 3 spans should be 100 ft for those spans over the main channel. A three-span bridge should have the center span length maximized at a site where debris may be a problem. For a two-span bridge, span lengths are subject to approval by the Hydraulics Engineer.

32-3.02(05) Flow Distribution

The conveyance of the proposed stream-crossing location should be calculated to determine the flow distribution and to establish the location of a bridge opening. The proposed facility should not cause a significant change in the existing flow distribution.

32-3.02(06) Scour

Design for bridge foundation scour considering the magnitude of flood, including the 100-year (1%) event, which generates the maximum scour depth. The design should use a geotechnical design practice safety factor of 2 to 3. The resulting design should then be checked using a super flood, Q_{500}. Use 1.7 times the magnitude of the 100-year (1%) event if no other source for Q_{500} is available and a geotechnical design practice safety factor of at least 1.0. See Section 32-6.08.

32-3.02(07) Temporary-Runaround Structure

A temporary-runaround structure is operational for three months to two years. Therefore, the serviceability criteria are greatly reduced. At a minimum, a temporary runaround should be serviceable during a Q_2 discharge, and should be checked for allowable backwater at a Q_{100} discharge. The best way to achieve this objective is to set the grade of the temporary runaround as close as possible to the elevations corresponding with the serviceability levels shown in Figure 32-3B, Design-Storm Frequency for Temporary Structure.

Backwater must be computed for the design storm and Q_{100} discharges. For a structure requiring an IDNR permit, the backwater at Q_{100} must not exceed 1.5 in. over the base condition (i.e., existing backwater elevation). Base condition is the condition of the floodplain on January 1, 1973, but without an unauthorized dam or levee. If an activity after December 31, 1972, lowered the regulatory-flood profile, the floodplain under the lower profile is the base condition. For a structure not requiring an IDNR permit, the backwater from the Q_{100} event should not exceed the finished-floor elevations of nearby buildings or residences.

32-3.02(08) Channel Clearing

Channel clearing consists of the removal of sediment to enlarge the waterway opening. Channel clearing should not occur below 1 ft above the Ordinary High Water elevation. For an intermittent stream, where the Ordinary High Water elevation is very near the flowline elevation, channel clearing should not occur below 2 ft above the flowline elevation.

32-4.0 DESIGN PROCEDURE

32-4.01 Survey Accuracy (Computation Method)

The design for a stream-crossing system requires a comprehensive engineering approach that includes formulation of alternatives, data collection, selection of the most cost-effective alternative according to established criteria, and documentation of the final design.

Water-surface profiles are computed for technical uses including the following:

1. flood insurance study;
2. flood hazard mitigation investigation;
3. drainage crossing analysis; and
4. longitudinal encroachment.

The completed profile can affect the highway bridge design and is the mechanism for determining the effect of a bridge opening on upstream water levels. Errors associated with computing water-surface profiles with the step-backwater profile method can be classified as follows:

1. data estimation errors resulting from incomplete or inaccurate data collection and inaccurate data estimation;

2. 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 (Manning's *n* value is the coefficient measuring boundary friction);

3. inadequate length of stream reach investigated; and

4. significant computational errors resulting from using cross-sectional spacings which are incorrectly considered to be adequate. The errors are due to inaccurate integration of the energy-loss-distance relationship which is the basis for profile computations. These errors may be reduced by adding interpolated or actual sections (more calculation steps).

32-4.02 Design-Procedure Outline

The following design-procedure outline should be used. Although the scope of the project and individual site characteristics make each design unique, this procedure should be applied unless indicated otherwise by INDOT.

1. Data Collection.

    a. Survey
        (1) Topography
        (2) Geology
        (3) High-water marks
        (4) History of debris accumulation, ice, and scour
        (5) Maps and aerial photographs
        (6) Field reconnaissance

b. Influences on Hydraulic Performance of Site
   (1) Other streams, reservoirs, water intakes
   (2) Structures upstream or downstream
   (3) Natural features of stream and floodplain
   (4) Channel modifications upstream or downstream
   (5) Floodplain encroachments
   (6) Sediment types and bed forms (Also see FHWA HEC 20, Appendix C, Scour, Site Data, Level I Qualitative Analysis, 1991)

c. Environmental Impact
   (1) Existing bed or bank instability (Level I)
   (2) Floodplain land use and flow distribution
   (3) Environmentally-sensitive areas (fisheries, wetlands, etc.)
   (4) Level I Qualitative Analysis (FHWA HEC 20, 1991)

c. Site-Specific Design Criteria
   (1) Road serviceability (design frequency)
   (2) Flood damage potential
   (3) Freeboard

2. <u>Hydrologic Analysis</u>.

a. Studies by Other Agencies
   (1) Federal Flood Insurance Studies
   (2) Federal Floodplain Studies by the COE, NRCS, etc.
   (3) IDNR and Local Floodplain Studies
   (4) Hydraulic performance of existing bridge

b. Watershed Morphology
   (1) Drainage area (attach map)
   (2) Watershed and stream slope
   (3) Channel geometry

c. Hydrologic Computations
   (1) Discharge and frequency for historical flood that complements the high-water marks used for calibration
   (2) Discharges for specified frequencies

3. <u>Hydraulic Analysis</u>.

a. Computer Model Calibration and Verification
b. Hydraulic Performance for Existing Conditions
c. Hydraulic Performance of Proposed Designs
d. Scour Computations

4. Selection of Final Design.

   a. Measure of compliance with established hydraulic criteria
   b. Compare proposed bridge size and backwater to the existing bridge
   c. Consider environmental and social criteria
   d. Make final selection
   e. Design of riprap, scour abatement, river training, etc.

5. Documentation. A bridge-waterway study is required at the Hydraulics Grade Review plan submittal. The study requires the following.

   a. Drainage-Area Determination. The drainage area can be obtained by planimeter from USGS Quad Maps using the USGS manual *Drainage Areas of Indiana Streams* for supplemental data as needed. For a large stream, the drainage area can be obtained from the USGS Manual *Drainage Areas of Indiana Streams*. The sources used must be photocopied and attached to the bridge waterway study.

   b. Hydrologic Analysis. Complete documentation of the method used and all relevant variables. See Chapter Twenty-nine for allowable methods.

   c. Hydraulic Analysis of Existing and Proposed Conditions. See Section 32-4.04. A hardcopy of all input and output files and a site plan that illustrates the location of all cross sections used in the hydraulic analysis is required. A disk with all input files must also be submitted.

   d. Justification of Selected Bridge. A summary of the results of the hydraulic analysis of bridge types is required.

   e. Summary of Hydraulic Parameters. Summarize the bridge-waterway study by listing the parameters as follows:

      (1) drainage area;
      (2) Q_{100} discharge;
      (3) Q_{100} elevation;
      (4) backwater;
      (5) velocity;
      (6) waterway area;
      (7) low-structure elevation;
      (8) skew;
      (9) existing waterway opening;
      (10) existing low-structure elevation; and

(11) existing backwater.

f. Files. Provide one diskette containing all input and output files for WSPRO or HEC-2.

g. Layout Sheet for LPA Project. This must show the following:

(1) profile grade (existing and proposed);
(2) waterway opening;
(3) pier placement;
(4) superstructure;
(5) hydraulic data; and
(6) Q_{100} elevation;

If road overflow is expected, the Plan and Profile sheet must show the limits of road overflow. A checklist form is provided in Section 32-7.0.

32-4.03 Hydraulic Performance of Bridge

The stream-crossing system is subject to either free-surface flow or pressure flow through one or more bridge openings with possible embankment overtopping. These hydraulic complexities should be analyzed using WSPRO or HEC-2 unless indicated otherwise by INDOT. Alternative methods of analysis of bridge hydraulics are discussed below, but emphasis is placed on the use of WSPRO.

It is impractical to perform the hydraulic analysis for a bridge by manual calculations due to the interactive and complex nature of those computations. However, an example of the basic manual calculations is included in the AASHTO *Model Drainage Manual*, Chapter 10, Appendix D, as an explanation of the aspects of bridge hydraulics.

The hydraulic variables and flow types are defined in Figure 32-4A, Bridge-Hydraulics Definitions Sketch, and Figure 32-4B, Bridge Flow Types. The following applies.

1. Backwater, h_1, 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 32-4B.

2. Type I flow consists of subcritical flow throughout the approach, bridge, and exit cross sections, and is the most common condition encountered in practice.

3. Type IIA and IIB flows both represent subcritical approach flows which have been choked by the contraction, resulting in the occurrence of critical depth in the bridge opening. In Type IIA, the critical water-surface elevation in the bridge opening is lower than the undisturbed normal water-surface elevation. In Type IIB, it is higher than the normal water surface elevation. A weak hydraulic jump immediately downstream of the bridge contraction is possible.

4. Type III flow is supercritical approach flow and remains supercritical through the bridge contraction. Such a flow condition is not subject to backwater unless it chokes and forces the occurrence of a hydraulic jump upstream of the contraction.

32-4.04 Methodologies

No single method is ideally suited for every situation. If a satisfactory computation cannot be achieved with a given method, an alternative method should be attempted. However, it has been found that, with attention to the setup requirements of each method, essentially duplicative results can usually be achieved using both momentum and energy methods.

32-4.04(01) Momentum

The Corps of Engineers' HEC-2 model uses a variation of the momentum method in the bridge routine where there are bridge piers. The momentum equation between cross sections 1 and 3 is used to detect Type II flow and solve for the upstream depth with critical depth in the bridge contraction.

This model has been used for the majority of the flood-insurance studies performed under the NFIP. However, some believe that the bridge-analysis routines in HDS-1 and WSPRO may yield a better definition of actual hydraulic performance.

32-4.04(02) HEC-RAS

The Corps of Engineers' Hydrologic Engineering Center (HEC) has developed the HEC-RAS (River Analysis System) program package. It operates under WINDOWS and has full graphic support. The package includes all the features inherent to HEC-2 and WSPRO, plus program selected friction slope methods, mixed flow regime capability, automatic *n* value calibration, ice cover, quasi 2-D velocity distribution, superelevation around bends, bank erosion, riprap design, stable channel design, sediment transport calculations, and scour.

HEC-RAS Version 2.2 will provide the outputs necessary to evaluate bridge hydraulics for all flow types. However, earlier versions will not be accepted for bridge hydraulics computations.

32-4.04(03) Energy (HDS-1)

The method developed by FHWA described in HDS-1 is an energy approach with the energy equation written between cross sections 1 and 4 as shown in Figure 32-4B, Bridge Flow Types, for Type I flow. The backwater is defined as the increase in the approach water-surface elevation relative to the normal water-surface elevation without the bridge.

This model utilizes a single typical cross section to represent the stream reach from points 1 to 4 on Figure 32-4B. It also requires the use of a single energy gradient. This method is no longer allowed by INDOT for final design analysis of a bridge due to its inherent limitations, but it may be useful for preliminary analysis and training. Studies performed by the Corps of Engineers for the FHWA show the need to utilize a multiple cross-section method of analysis to achieve reasonable stage-discharge relationships at a bridge.

32-4.04(04) Energy (WSPRO)

WSPRO combines step-backwater analysis with bridge backwater calculations. This method allows for pressure flow through the bridge, embankment overtopping, and flow through multiple openings and culverts. It also includes an improved technique for determining approach flow lengths and the introduction of an expansion loss coefficient. The flow-length improvement was found necessary where approach flows occur on a very wide, heavily vegetated floodplain. The program also greatly facilitates the hydraulic analysis required to determine the least-cost alternative.

WSPRO is suggested for both preliminary and final analyses of bridge hydraulics. If only a single surveyed cross section is available, the input-data propagation features of WSPRO make it easy to apply with more comprehensive output available than with HDS-1.

32-4.04(05) Two-Dimensional Modeling

The water-surface profile and velocities in a section of river are often predicted using a computer model. In practice, most analysis is performed using one-dimensional methods such as the standard step method found in WSPRO or HEC-2. Although one-dimensional methods are adequate for many applications, these methods cannot provide a detailed determination of the cross-stream water surface elevations, flow velocities, or flow distribution.

Two-dimensional models are more complex and require more time to set up and calibrate. They require essentially the same field data as a one-dimensional model and, depending on complexity, may require a little more computer time.

Bri-Stars is a semi-two-dimensional model capable of computing alluvial scour or deposition through subcritical, supercritical, and a combination of both flow conditions involving hydraulic jumps. It is capable of simulating channel widening or narrowing and local scour due to highway encroachments.

It has a bridge component which allows the computation of hydraulic flow variables and the resulting scour. Bri-Stars also includes a companion expert system program which allows classifying a stream by its morphological properties. See Section 30-5.0 for a more complete discussion of Bri-Stars.

The USGS has developed a two-dimensional finite element model for the FHWA that is designated FESWMS. This model has been developed to analyze flow at a bridge crossing where complicated hydraulic conditions exist. This two-dimensional modeling system is flexible and may be applied to many types of steady and unsteady flow problems, including multiple-opening bridge crossing, spur dike, floodplain encroachment, multiple channels, flow around an island, and flow in an estuary. Where the flow is essentially two-dimensional in the horizontal plane, a one-dimensional analysis may lead to costly over-design or possibly improper design of hydraulic structures and improvements.

32-4.04(06) Physical Modeling

A complex hydrodynamic situation defies accurate or practical mathematical modeling. Physical models should be considered as follows:

1. hydraulic performance data are needed that cannot be reliably obtained from mathematical modeling;
2. risk of failure or excessive over-design is unacceptable; and
3. research is needed.

The constraints on physical modeling are as follows:

1. size (scale);

2. cost; and
3. time.

32-4.05 WSPRO Modeling

In theory, the water-surface profile used in the hydraulic analysis of a bridge should extend from a point downstream of the bridge that is beyond the influence of the constriction, to a point upstream that is beyond the extent of the bridge backwater. In practice, all of the cross sections that are actually necessary for the energy analysis through the bridge opening for a single-opening bridge without spur dikes are shown in Figure 32-4C, Cross-Section Locations for Stream Crossing with a Single Waterway Opening. The additional cross sections that are necessary for computing the entire profile are not shown in this Figure. Cross sections 1, 3, and 4 are required for a Type I flow analysis and are referred to as the approach section, bridge section, and exit section, respectively. Cross section 3F, which is identified as the full-valley section, is needed for the water-surface profile computation without the presence of the bridge contraction. Cross section 2 is used as a control point in Type II flow but requires no input data. Two more cross sections must be defined if spur dikes and a roadway profile are specified.

Pressure flow through the bridge opening is assumed to occur where the depth just upstream of the bridge opening exceeds 1.1 times the hydraulic depth of the opening. The flow is then 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. WSPRO can also simultaneously consider embankment overflow as a weir discharge. This leads to flow classes 1 through 6 as shown in Figure 32-4D, Flow Classification According to Submergence Conditions (WSPRO User Instructors Manual - 1990).

In free-surface flow, there is no contact between the water surface and the low-girder elevation of the bridge. In orifice flow, only the upstream girder is submerged. In submerged-orifice flow, both the upstream and downstream girders are submerged. A total of four different bridge types can be analyzed.

The user's instruction manual for WSPRO should serve as a source for more-detailed information on using the computer model. Some specific example problems are provided in *Model Drainage Manual*, Chapter 10, Appendix B, with sample computer input and output data provided. Only the information required to understand the examples is included.

32-4.06 HEC-2 Modeling

In theory, the water surface profile used in the hydraulic analysis of a bridge should extend from a point downstream of the bridge that is beyond the influence of the constriction, to a point upstream that is beyond the extent of the bridge backwater. The cross sections that are necessary in practice for the analysis of a single-opening bridge using the special bridge option are shown in Figure 32-4E, Cross Section Locations in the Vicinity of a Bridge.

Energy losses caused by a structure such as a bridge or culvert are computed in two parts. First, the losses due to expansion and contraction of the cross section on the upstream and downstream sides of the structure are computed in the standard step calculations. Secondly, the loss through the structure itself is computed by either the normal-bridge or the special-bridge method.

The user's instructional manual for HEC-2 should serve as a source for more-detailed information for using the computer model. Input/output examples are provided. HEC-2 has its own data creation package, COED, that assists the user with preparing and editing input data and includes online help features. A separate, stand-alone data-editing program which checks for input/modeling errors is also provided. Cross-section, water-surface profile, and rating-curve viewing/plotting/printing are provided using the PLOT2 program.

The normal-bridge method handles the cross section at the bridge just as it can at a river cross section with the exception that the area of the bridge below the water surface is subtracted from the total area. The wetted perimeter is increased where the water-surface elevation exceeds the low chord. The normal-bridge method is particularly applicable for a bridge without piers, a bridge under high submergence, or for low flow through a circular or arch culvert. Where flow crosses critical depth in a structure, the special-bridge method should be used. The normal-bridge method is automatically used by the computer, though data were prepared for the special-bridge method, for a bridge without piers and under low-flow control.

The special-bridge method can always be used, but it should be used for a bridge with piers where low flow or pressure flow controls, or where flow passes through critical depth in passing through the structure. The special-bridge method computes losses through the structure for low flow, weir flow, and pressure flow, or for a combination of these.

A series of program capabilities are available to restrict flow to the effective flow areas of cross sections. Among these capabilities are options to simulate sediment deposition, to confine flow to a levied channel, to block out roadway fill or a bridge deck, and to analyze floodplain encroachments.

Cross sections with low overbank areas or levees require consideration in computing water-surface profiles because of possible overflow into areas outside the main channel. The computations are based on the assumption that all of the 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 elevation, and if the water cannot enter or leave the overbanks upstream of that cross section, the flow areas in these overbanks should not be used in the computations. Variable IEARA on the X3 card and the bank stations coded on the X1 card in fields 3 and 4 are used for this condition. By setting IEARA equal to 10, the program will consider only flow confined by the levees, unless the water-surface elevation is above the top of one or both levees, such that flow areas outside the levees will be included. If this option is used and the water-surface elevation is close to the top of a levee, it may not be possible to balance the assumed and computed water surface elevations due to the changing flow areas just above and below the levee top. Where this condition occurs, a statement will be printed that states that the assumed and computed water-surface elevations for the cross section cannot be balanced. A water-surface elevation equal to the elevation which came closest to balancing will be adopted. The program user should determine the appropriateness of the assumed water-surface elevation and restart the computation at that cross section if required.

The user should study the flow pattern of the river where levees exist. If, for example, a levee is open at both ends and flow passes behind the levee without overtopping it, IEARA equals 0, or a blank should be input. 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, it may be necessary to set IEARA equal to 10 to confine the flow to the channel.

A user’s instruction manual for HEC-2 is available and should serve as a source for more-detailed information on using this computer model.

32-5.0 BRIDGE SCOUR OR AGGRADATION

32-5.01 Introduction

Hydraulic analysis of a bridge design requires that an assessment be made of the proposed bridge’s vulnerability to undermining due to potential scour. Because of the extreme hazard and economic hardships posed by a rapid bridge collapse, considerations must be made in selecting appropriate flood magnitudes for use in the analysis. The hydraulics engineer must endeavor to always be aware of and use the most current scour-forecasting technology.

The FHWA has issued Technical Advisory TA 5140.20 on bridge scour. The document, *Interim Procedures for Evaluating Scour at Bridges* is an attachment to the Technical Advisory. The interim procedures were replaced by HEC 18 (1991, 1993, 1995). Users of this *Manual* should see HEC 18 for a more thorough treatise on scour and scour prediction methodology. A companion FHWA document to HEC 18 is HEC 20, *Stream Stability at Highway Structures*.

The inherent complexities of stream stability, further complicated by highway-stream crossings, require a multilevel solution 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 analyses using basic hydrologic, hydraulic, and sediment-transport engineering concepts. Such analyses can include evaluation of flood history, channel hydraulic conditions (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 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 or physical hydraulic models should be considered. This multilevel approach is provided in HEC 20.

Less hazardous are problems associated with aggradation. Where freeboard is limited, problems associated with increased flood hazards to upstream property or to the traveling public due to more frequent overtopping may occur. Where aggradation is expected, it may be necessary to evaluate these consequences. Aggradation in a stream reach may serve to moderate potential scour depths. Aggradation can be referred to as negative scour.

32-5.02 Scour Types

Present technology dictates that bridge scour should be evaluated as the interrelated components as follows:

1. long-term profile changes (aggradation or degradation);
2. plan form change (lateral channel movement);
3. contraction scour or deposition; and
4. local scour.

32-5.02(01) Long-Term Profile Changes

Long-term profile changes can result from stream-bed profile changes that occur from aggradation or degradation.

1. Aggradation is the deposition of bed load due to a decrease in the energy gradient.

2. Degradation is the scouring of bed material due to increased stream sediment transport capacity which results from an increase in the energy gradient.

Forms of degradation and aggradation should be considered as imposing a permanent future change for the stream-bed elevation at a bridge site if they can be identified.

32-5.02(02) Plan-Form Change

A plan-form change is a morphological change such as meander migration or bank widening. The lateral movement of a meander can threaten bridge approaches and 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.

32-5.02(03) Contraction

Channel contraction scour results from a constriction of the channel which may, in part, be caused by bridge piers in the waterway. Deposition results from an expansion of the channel or from the bridge site being positioned immediately downstream of a steeper reach of stream. A highway, bridge, or natural-channel contraction is the most commonly encountered cause of contraction scour. The practices provided herein for estimating deposition or contraction scour are as follows.

1. Sediment Routing Practice. This practice should be considered if either bed armoring or aggradation from an expanding reach is expected to cause an unacceptable hazard.

2. Empirical Practice. This practice is adapted from laboratory investigations of bridge contractions in non-armoring soils and, as such, must be used considering this qualification. This practice does not consider bed armoring and its application for aggradation may be technically weak.

The same empirical practice algorithms used in herein to evaluate a naturally contracting reach may also be used to evaluate deposition in an expanding reach provided armoring is not expected to occur. With deposition, the practice of applying the empirical equations in reverse is required. For example, the narrower cross section is upstream which results in the need to manipulate the use of the empirical contraction scour equation. This need to manipulate the intended use of an equation does not occur with the sediment-routing practice which is why it may be more reliable in an expanding reach.

32-5.02(04) Local Scour

Exacerbating the potential scour hazard at a bridge site are 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. However, the importance of these geometric variables will vary. As an example, increasing the pier or cofferdam width either through design or debris accumulation will increase the amount of local scour, but only up to a point in a subcritical-flow stream. After reaching this point, pier scour should not be expected to measurably increase with increased stream velocity or depth. This threshold has not been defined for a rarer, supercritical-flowing stream.

Where fendering or other pier-protection systems are used, their effect on pier scour and collection of debris should be considered in design. The stability of abutments in an area of turbulent flow should be investigated. Exposed embankment slopes should be protected with appropriate scour countermeasures.

32-5.03 Armoring

Armoring occurs because a stream or river is unable, during a particular flood, to move the more-coarse material comprising either the bed or, if some bed scour occurs, its underlying material. Scour may occur initially but later become arrested by armoring before the full scour potential is reached for a given flood magnitude. If armoring does occur, the coarser bed material will tend to remain in place or quickly redeposit to form a layer of riprap-like armor on the stream bed or in the scour holes and thus limit further scour for a particular discharge. This armoring effect can decrease scour-hole depths which were predicted based on formulas developed for sand or other fine material channels for a particular flood magnitude. If a larger flood occurs than used to define the probable scour-hole depths, scour will likely penetrate deeper until armoring again occurs at a lower threshold.

Armoring may also cause bank widening. Bank widening encourages a river or stream to seek a more unstable, braided regime. Such instabilities may pose serious problems for a bridge as they encourage further, difficult-to-assess plan-form changes. Bank widening also spreads the approach-flow distribution which in turn results in a more-severe bridge-opening contraction.

32-5.04 Scour Resistant Materials

Caution is necessary in determining the scour resistance of bed materials and the underlying strata. With sand-sized 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 may not be realized during the passage of a particular flood. However, some scour-resistant material may be lost. This material is replaced with more-easily scoured material. Thus, another flood may later reach the predicted scour depth. Serious scour has been observed to occur in materials commonly perceived to be scour-resistant, such as consolidated soils and glacial till, bedrock streams, and streams with gravel and boulder beds.

32-5.05 Scour-Analysis Methods

Before the scour-forecasting methods for contraction and local scour can be applied, it is first necessary to obtain the fixed-bed channel hydraulics, estimate the profile and plan form scour or aggradation, adjust the fixed-bed hydraulics to reflect these changes, and compute the bridge hydraulics. Two methods are provided in the AASHTO *Model Drainage Manual*, Chapter 10, for combining the contraction and local scour components to obtain total scour. Method 1 has application where armoring is not a concern or insufficient information is available to permit its evaluation, or where more-precise scour estimates are not deemed necessary. Method 2 may be used where stream bed armoring is of concern, more-precise contraction scour estimates are deemed necessary, or deposition is expected and is a primary concern. INDOT uses Method 1, which is described below.

Method 1 is considered a conservative practice because it assumes that the scour components develop independently. Thus, the potential local scour to be calculated using this method should be added to the contraction scour without considering the effects of contraction scour on the channel and bridge hydraulics. The approach with Method 1 is as follows.

1. Estimate the natural channel's hydraulics for a fixed-bed condition based on existing conditions.

2. Assess the expected profile and plan-form changes.

3. Adjust the fixed-bed hydraulics to reflect expected profile or plan-form changes.

4. Estimate contraction scour using the empirical contraction formula and the adjusted fixed-bed hydraulics assuming no bed armoring.

5. Estimate local scour using the adjusted fixed-bed channel and bridge hydraulics assuming no bed armoring.

6. Add the local scour to the contraction scour to obtain the total scour. If contraction scour is negative, then use zero for contraction scour.

32-5.06 Scour-Assessment Procedure

Bridge scour assessment should be accomplished by collecting the data and applying the procedure outlined herein. An example problem demonstrating the scour computations is included in the AASHTO *Model Drainage Manual*, Chapter 10, Appendix B.

32-5.06(01) Site Data

1. Bed Material. Obtain bed-material samples for all channel cross sections where armoring will be evaluated. If armoring is not being evaluated, this information need only be obtained at the site. From these samples, try to identify historical scour and associate it with a discharge. Also, determine the bed-material size distribution in the bridge reach and from this distribution determine d16, d50, d84, and d90. This will be accomplished by using the appropriate soil borings.

2. Geometry. Obtain existing stream and floodplain cross sections, stream profile, site plan, and the streams present, and where possible, historic geomorphic plan form. Also, locate the bridge site with respect to other bridges in the area, tributaries to the stream or close to the site, bedrock controls, man-made controls (dam, old check structure, river-training works, etc.), and downstream confluence with other streams. Locate (distance and height) headcuts due to natural causes or, for example, gravel-mining operations. Upstream gravel-mining operations may absorb the bed-material discharge resulting in the more-adverse clear-water scour situation discussed below. Data related to plan-form changes such as meander migration and the rate at which they may be occurring are useful.

3. Historic Scour. Obtain scour data for other bridges or similar facilities along the stream.

4. Hydrology. Identify the character of the stream hydrology; i.e., perennial, ephemeral, intermittent, and whether it is flashy, or subject to broad hydrograph peaks resulting from a gradual flow increase such as that which occurs with a thunderstorm or snowmelt.

5. Geomorphology. Classify the geomorphology of the site, whether it is a floodplain stream, crosses a delta, or crosses a youthful, mature, or old-age alluvial fan.

32-5.06(02) General

1. Step 1. Decide which analysis method is applicable. Method 1 should be used to evaluate existing bridges to identify significant potential scour hazards or, where armoring is obviously not of concern, a proposed bridge. Method 2 should be used to evaluate bridges where significant armoring may occur.

2. Step 2. Determine the magnitude of the 100-year flood and the 500-year super flood.

3. Step 3. Develop a water-surface profile through the site's reach for fixed-bed conditions using WSPRO or HEC-2.

4. Step 4. Obtain the variables necessary to determine contraction and local scour.

5. Step 5. Compute the predicted scour depths using the equations in HEC 18 for contraction and pier scour for the 100-year and 500-year floods or an overtopping flood of a lesser recurrence interval.

6. Step 6. Once an acceptable scour threshold is determined, the Office of Geotechnical Services can prepare a foundation recommendation for the bridge based on the scour information obtained from the foregoing procedure and using commonly accepted safety factors. The structural engineer should evaluate the lateral stability of the bridge based on the aforementioned scour.

   Spread footings on soil or erodible rock should be located so that the bottom of each footing is below the scour depth determined for the check flood for scour. A spread footing on scour-resistant rock should be designed and constructed to maintain the integrity of the supporting rock.

7. Step 7. Repeat the aforementioned assessment procedures using the greatest bridge opening flood discharge associated with the selected 500-year superflood, or an overtopping flood of a lesser recurrence interval. These findings are again for the Office of Geotechnical Services to use in evaluating the foundation recommendation obtained in Step 6. A foundation design safety factor of 1.0 is used to ensure that the bridge is marginally stable for a flood associated with the 500-year superflood.

32-5.07 Pressure-Flow Scour

The following is extracted from the FHWA Publication HEC 18 *Evaluating Scour at Bridges*, April, 1993.

Pressure flow, which is also denoted as orifice flow, occurs where the water surface elevation at the upstream face of the bridge is greater than or equal to the low chord of the bridge superstructure. Pressure flow under the bridge results from a pile-up of water on the upstream bridge face, and a plunging of the flow downward and under the bridge. At a higher approach-flow depth, 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 intensity of the horseshoe vortex. The vertical contraction of the flow is a more significant cause of the increase in scour depth. However, where a bridge becomes submerged, the average velocity under it is reduced due to a combination of additional backwater caused by the bridge superstructure 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, an increase in local scour due to pressure flow may be offset by a lesser velocity through the bridge opening due to increased backwater, and a reduction in discharge due to overtopping.

WSPRO or HEC-2 can be used to determine the discharge through the bridge and the velocity of approach and depth upstream of the piers where flow impacts the bridge superstructure. These values should be used to calculate local pier scour. Engineering judgment should then be used 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 number (Fr = 0.1) to 1.6 for a high-approach Froude number (Fr = 0.6). If the bridge is overtopped, 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.

32-6.0 DESIGN PHILOSOPHY

32-6.01 Introduction

A stream is a dynamic natural system which, as a result of the encroachment caused by elements of a stream-crossing system, will respond so as to challenge an experienced hydraulics engineer. The complexities of the stream response to encroachment demand that the hydraulics engineer must be involved from the outset in the choice of alternative stream-crossing locations, and at the design team must have experience in the hydraulic design of a stream-crossing system. The hydraulics engineer should also be involved in the solution of stream-stability problems at an existing structure.

The design issues which contribute to the overall complexity of spanning a stream with a stream-crossing system are discussed below. A more thorough discussion of design philosophy and design considerations is provided in the AASHTO *Highway Drainage Guidelines* Hydraulic Analyses for the Location and Design of Bridges.

32-6.02 Location of Stream Crossing

Although many factors, including non-technical ones, enter into the final location of a stream-crossing system, the hydraulics of the proposed location must have a high priority. Hydraulic considerations in selecting the location include floodplain width and roughness, flow distribution and direction, stream type (braided, straight, or meandering), stream regime (aggrading, degrading, or equilibrium), and stream controls. The hydraulics of a proposed location also affects environmental considerations such as aquatic life, wetlands, sedimentation, and stream stability. The hydraulics of a particular site determines whether or not certain national objectives such as wise use of floodplains, reduction of flooding losses, and preservation of wetlands can be satisfied.

32-6.03 Coordination, Permits, and Approvals

The interests of other government agencies must be considered in the evaluation of a proposed stream-crossing system. Cooperation and coordination with these agencies, especially water-resources-planning agencies, must be undertaken.

The designer of a stream-crossing system must be cognizant of relevant local, State, and Federal laws and permit requirements. Federal and State permits are required for construction of a bridge over a navigable waterway and are issued by the U.S. Coast Guard and IDNR. Permits for other construction activities in a navigable waterway are under the jurisdiction of the U.S. Army Corps of Engineers. Applications for Federal permits may require environmental-impact assessments under the National Environmental Policy Act of 1969. IDNR issues permits for construction in a floodway or lake preservation. IDEM issues 401 permits to accompany 404 permits if required. If more than 5 ac of land are disturbed, a Rule 5 (NPDES) permit will be required. See Chapter Nine for more information on permits and certifications.

32-6.04 Environmental Considerations

Environmental criteria which must be satisfied in the design of a stream-crossing system include the preservation of wetlands and protection of aquatic habitat. Such considerations often require the expertise of a biologist. The hydraulic design criteria related to scour, degradation, aggradation, flow velocities, and lateral distribution of flow, for example, are criteria for evaluation of environmental impacts and the safety of the stream-crossing structure.

32-6.05 Stream Morphology

The form and shape of the stream path created by its erosion and deposition characteristics comprise its morphology. A stream can be braided, straight, or meandering, or it can be in the process of changing from one form to another as a result of natural or man-made influences. A historical study of the stream morphology at a proposed stream-crossing site is mandatory (FHWA HEC 20 Level I Analysis). This study should also include an assessment of long-term trends in aggradation or degradation. A braided stream or alluvial fans should be avoided at a stream-crossing site if possible.

32-6.06 Data Collection

The purpose of data collection is to gather all necessary site information. This should include such information as topography and other physical features, land use and culture, flood data, basin characteristics precipitation data, historical high-water marks, existing structures, channel characteristics, and environmental data. A site plan should be developed on which much of the data can be shown.

32-6.07 Scour

The extreme hazard posed by a bridge subject to scour failure dictates a different philosophy in selecting suitable flood magnitudes to use in the scour analysis. With bridge-flood hazards other than scour, such as those caused by roadway overtopping or property damage from inundation, a prudent and reasonable practice is to first select a design flood to determine a trial bridge opening geometry. This geometry is either subjectively or objectively selected based on the initial cost of the bridge along with the potential future costs for flood hazards. Following the selection of this trial bridge geometry, the base flood (100-year) is used to evaluate this selected opening. This two-step evaluation process is used to ensure that the selected bridge opening, based on the design flood, includes no unexpected increase in existing flood hazards other than those from scour or aggradation. Bridge scour or aggradation should be considered from the base flood (100-year) as well as the superflood (500-year).

Prediction technology is steadily developing but lacks the reliability associated with other facets of hydraulics engineering. Formulas for predicting scour depths are currently available and others will be developed in the future. The designer should strive to be acquainted with the state of practice at the time of a given analysis and is encouraged to be conservative in the resulting scour predictions.

The determination should be made as to what constitutes the greatest discharge passing through the bridge opening during a particular flood. Where there are relief structures on the floodplain, or if overtopping occurs, a flood other than the base flood or superflood may cause the worst 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 floodplain relief opening. A discharge relief at the bridge due to overtopping or a relief opening may not result in reduction in the bridge opening discharge.

With potential bridge scour hazards, a different flood selection and analysis philosophy is considered reasonable. The aforementioned trial bridge opening, which is selected by considering initial costs and future flood hazard costs, should be evaluated for two possible scour conditions, with the worse condition dictating the foundation design and possibly indicating a change in the selected trial bridge opening.

First, evaluate the proposed bridge and road geometry for scour using the base flood. Once the expected scour geometry has been assessed, the Office of Geotechnical Services will recommend the foundation. This foundation design should use the conventional foundation safety factors, and eliminate consideration of stream-bed or bank material displaced by scour for foundation support.

Second, impose a superflood on the proposed bridge and road geometry. This event should be used to evaluate the proposed bridge opening to ensure that the resulting potential scour will produce no unexpected scour hazards. The superflood is defined as the 500-year flood or a designated ratio (e.g., 1.7) times the 100-year flood. Similar to the manner for the base flood, evaluate the selected bridge opening using the superflood. The foundation design based on the base flood should then be reviewed by the Office of Geotechnical Services using a safety factor of 1.0 and again, considering stream-bed or bank material displaced by scour from the super flood.

32-6.08 Preventive or Protection Measures

Based on an assessment of potential scour provided by the Hydraulics Engineer, the structural designer can incorporate design features that will prevent or mitigate scour damage at the piers. Circular piers or elongated piers with circular noses and an alignment parallel to the flood flow direction are a possible alternative. Spread footings should be used only where the stream bed is extremely stable below the footings, or where the spread footings are founded at a depth below the maximum scour computed in Section 32-6.07. Footings may be founded above the scour elevation where they are keyed into non-erodible rock. Drilled shafts or drilled piers are possible where piles cannot be driven. Protection against stream-bed degradation can be provided with a drop structure or grade-control structure in, or downstream of, the bridge opening.

Rock riprap is used where stone of sufficient size is available, to armor abutment fill slopes and the area around the bases of piers. Riprap design information is provided in Chapter Thirty-eight.

If possible, clearing of vegetation upstream or downstream of the toe of the embankment slope should be avoided. Embankment overtopping may be incorporated into the design but should be located away from the bridge abutments and superstructure. Spur dikes are recommended to align the approach flow with the bridge opening and to prevent scour around the abutments. They are usually elliptically shaped with a major to minor axis ratio of 2.5 to 1. A length of approximately 150 ft provides a satisfactory design. Their length can be determined according to HDS 1. Spur dikes, embankments, and abutments should be protected with rock riprap with a filter blanket or other revetments approved by INDOT.

32-6.09 Deck Drainage

The designer is responsible to provide for the safety of the traveling public. There is a greater risk of someone being injured or killed in an accident on the bridge as the result of wet pavement than there is of injury or death due to the catastrophic collapse of the bridge due to a flood or structural failure.

An improperly-drained bridge deck can cause problems including corrosion, icing, or hydroplaning. If possible, a bridge deck should be watertight, and deck drainage should be carried to the ends of the bridge. Drains at the ends of the bridge should have sufficient inlet capacity to carry all bridge deck drainage.

The design of pavement drainage on the bridge should use the same criteria as the approach roadway. However, an approach roadway with a rural typical section will be freer draining than a bridge deck with parapets where the deck confines the runoff in a manner similar to a curbed-roadway section. Consideration must be given to spread on the bridge deck.

Where it is necessary to intercept deck drainage at intermediate points along the bridge, the design of the interceptors should be in accordance with the procedure described in Chapter Thirty-three.

Where deck-drainage interceptors are needed, a collection system will be necessary to discharge the runoff. Some considerations for this system are as follows:

1. environmental concerns for discharging pavement runoff directly into a waterway;
2. design and maintenance of an extensive drain system attached to the superstructure;
3. free drops from deck interceptors;
4. 6-in. minimum projection beyond the lowest adjacent superstructure component; and
5. provide erosion control under free drops unless the outlet from the bridge superstructure is more than 40 ft above the ground.

32-6.10 Construction Maintenance

A temporary structure or crossing used during construction should be designed for a specified risk of failure due to flooding during the construction period. The impacts on normal water levels and normal flow distribution must be considered.

Each borrow area within the floodplain should be chosen to minimize the potential for scour and adverse environmental effects within the limits of the bridge and its approaches on the floodplain.

The stream-crossing design should incorporate measures which reduce maintenance costs if possible. These measures include spur dikes, retards, guide dikes, jetties, riprap protection of abutments and embankments, embankment overflow at lower elevations than the bridge deck, and alignment of piers with the flow.

32-6.11 Waterway Enlargement

Roadway and structural constraints can dictate the vertical positioning of a bridge, resulting in a small vertical clearance between the low chord and the ground. Significant increases in span length provide small increases in effective waterway opening.

It is possible to increase the effective area by excavating a flood channel through the reach affecting the hydraulic performance of the bridge. The factors that must be accommodated if this action is taken are as follows.

1. The flow line of the flood channel should be set above the stage elevation of the dominant discharge. See the AASHTO *Highway Drainage Guidelines*.

2. The flood channel must extend far enough up- and downstream of the bridge to establish the desired flow regime through the affected reach.

3. The flood channel must be stabilized to prevent erosion and scour.

32-6.12 Auxiliary Openings

The need for auxiliary waterway openings, or relief openings, arises on a stream with a wide floodplain. The purpose of openings in the floodplain is to pass a portion of the flood flow in the floodplain once 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 it has predictable capacity during a flood event.

The objectives in choosing the location of auxiliary openings include the following:

1. maintenance of flow distribution and flow patterns;
2. accommodation of relatively large flow concentrations on the floodplain;

3. avoidance of floodplain flow along the roadway embankment for long distances; and
4. crossing of significant tributary channels.

The technological weakness in modeling auxiliary openings is in the use of one-dimensional models to analyze two-dimensional flow. The development of two-dimensional models, such as FESWMS, is a step toward more-adequate analysis of a complex stream-crossing system.

The most complex factor in designing an auxiliary opening is determining the division of flow between two or more structures. If incorrectly proportioned, one or more of the structures may be overtaxed during a flood event. The design of an auxiliary opening should be generous to guard against that possibility.

32-7.0 DESIGN PROCEDURE FORM

A checklist to document the design procedures, studies, decisions, criteria, calculations, etc., for bridge hydraulics is shown as Figure 32-7A, Design Procedure Checklist.

32-8.0 REFERENCES

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2. AASHTO, *Model Drainage Manual*, Chapter Ten, Bridges.

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15. U.S. Army Corps of Engineers, *HEC 2, Water Surface Profiles, User's Manual*, Version 4.6, September 1990.

16. U.S. Army Corps of Engineers, *HEC-RAS, River Analysis System, User's Manual*, Draft Version 2.0, April 1997.

Functional Classification	Allowable Backwater	Roadway Serviceability	Allowable Velocity
Freeway	Q_{100}	Q_{100}	Q_{100}
Multilane Non-Freeway	Q_{100}	Q_{100}	Q_{100}
Two-Lane Facility			
AADT ≥ 3000	Q_{100}	Q_{100}	Q_{100}
3000 > AADT ≥ 1000	Q_{100}	Q_{25}	Q_{100}
AADT < 1000	Q_{100}	Q_{10}	Q_{100}
Ramp	Q_{100}	Q_{100}	Q_{100}

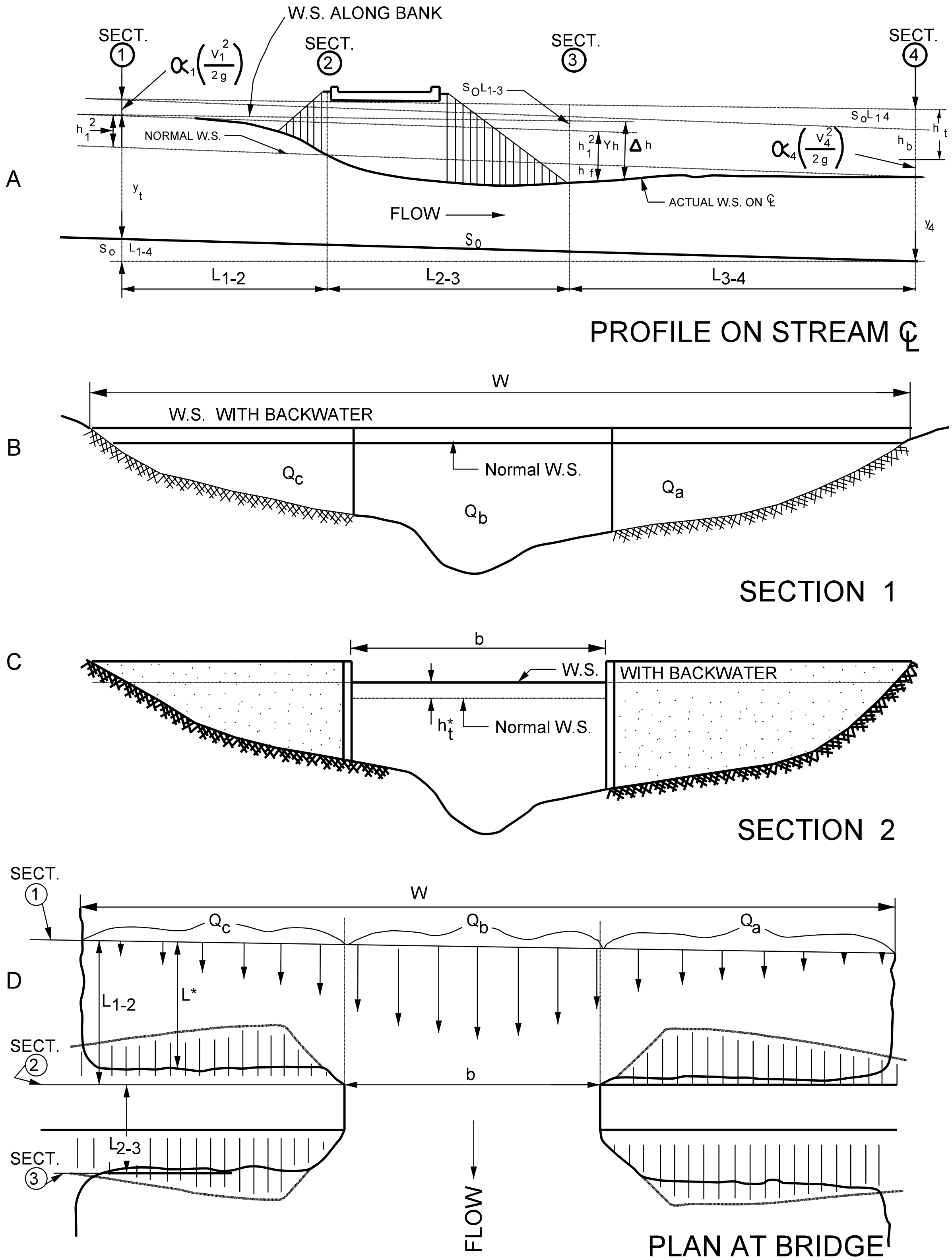
DESIGN-STORM FREQUENCY
(Bridge Waterway Opening)

Figure 32-3A

Facility Type	Road Serviceability	Allowable Velocity
Freeway	Q_{25}	Q_{10}
Non-Freeway, ≥ 4 Lanes	Q_{10}	Q_{10}
Two-Lane Facility		
ADT ≥ 3000	Q_{10}	Q_{10}
3000 > ADT ≥ 1000	Q_2	Q_2
ADT <1000	Q_2	Q_2

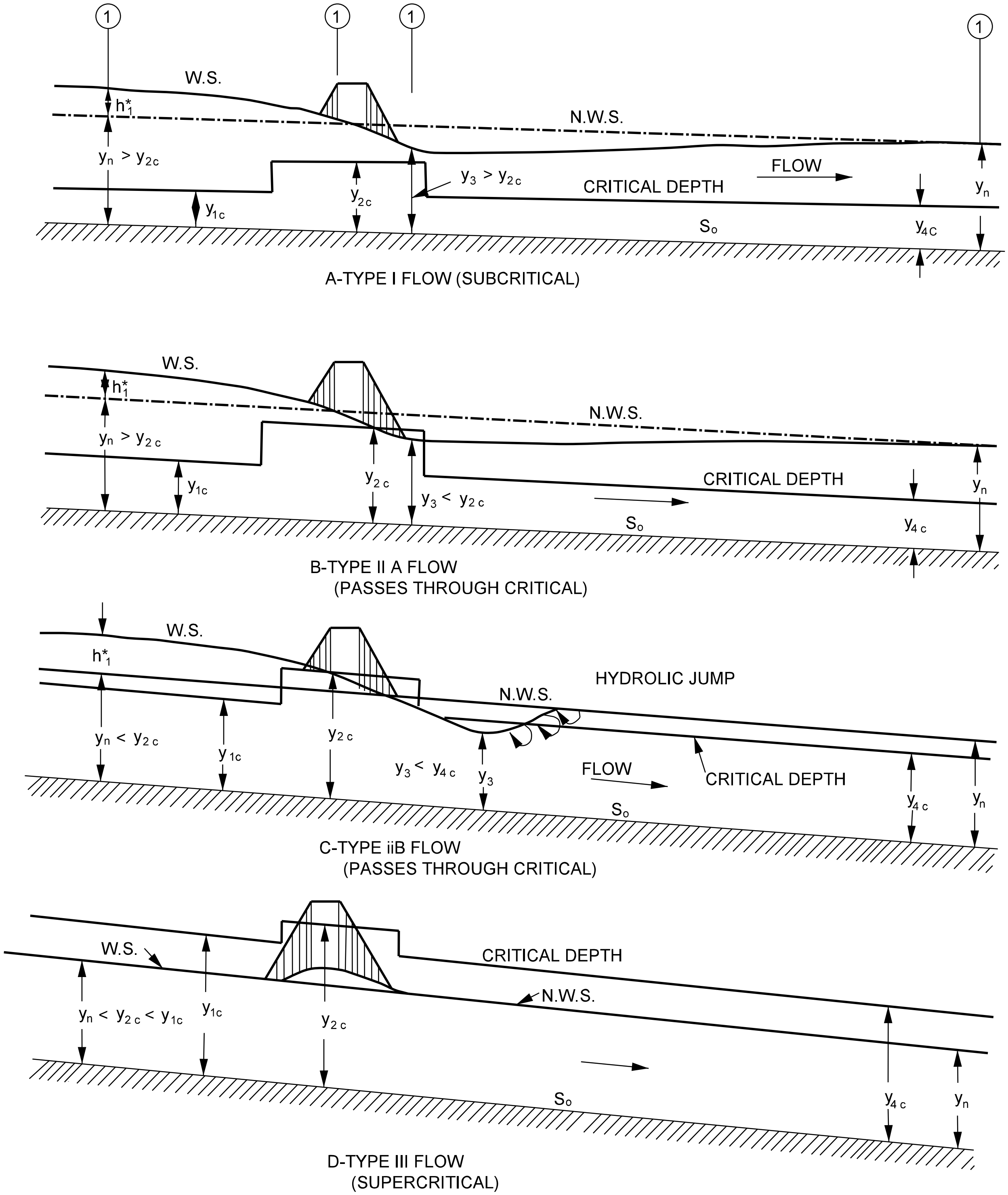
DESIGN-STORM FREQUENCY, TEMPORARY STRUCTURE

Figure 32-3B



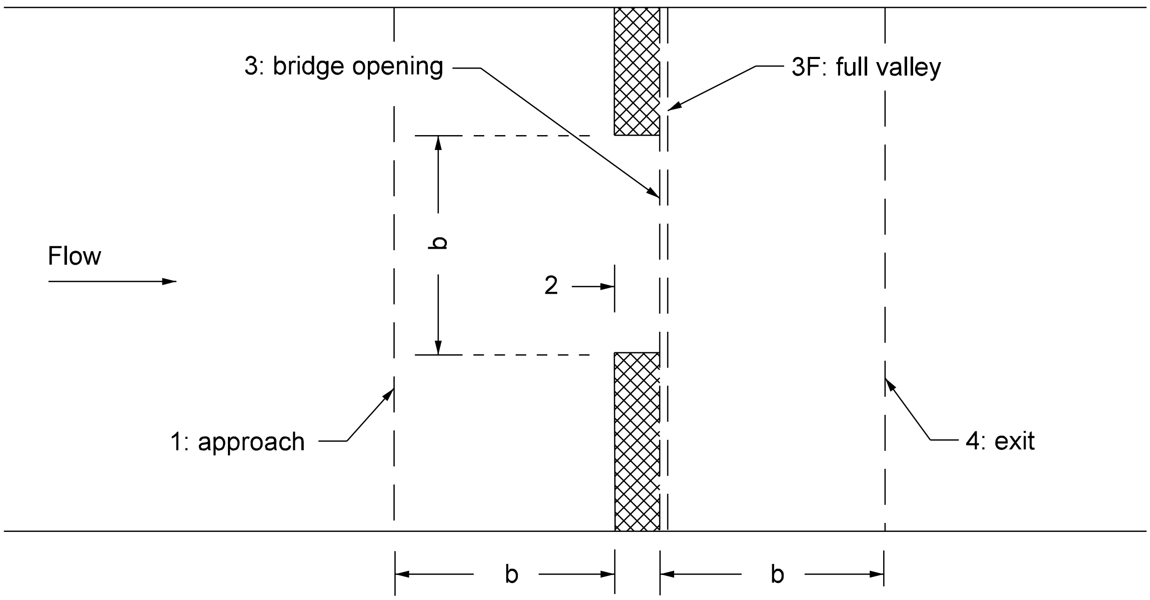
BRIDGE HYDRAULICS DEFINITIONS SKETCH

Figure 32-4A

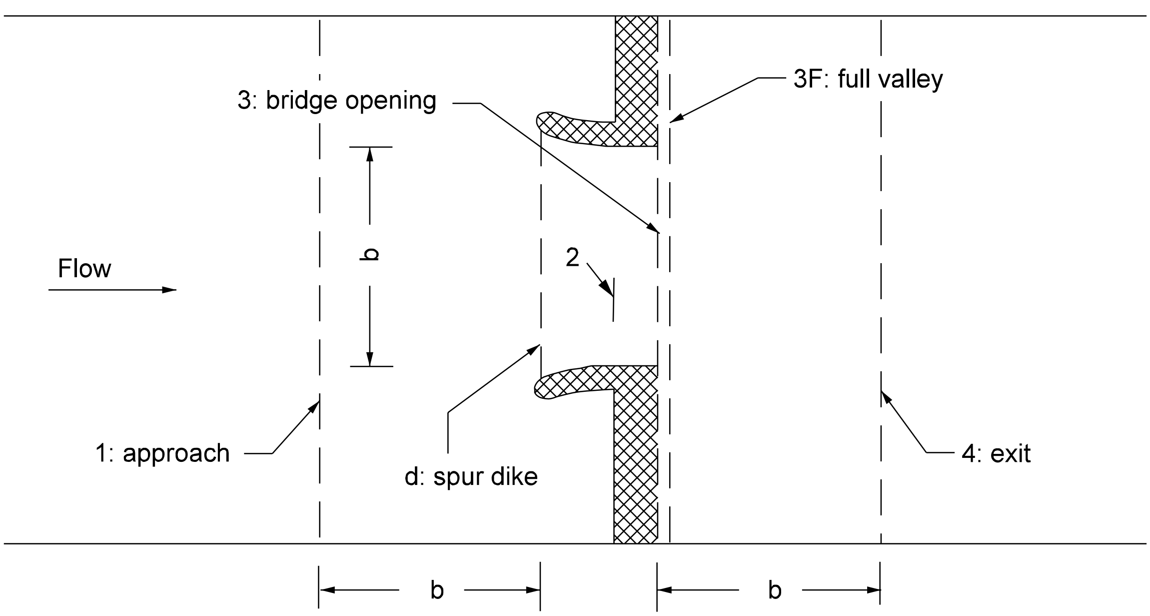


BRIDGE FLOW TYPES

Figure 32-4B



a) without spur dikes



b) with spur dikes

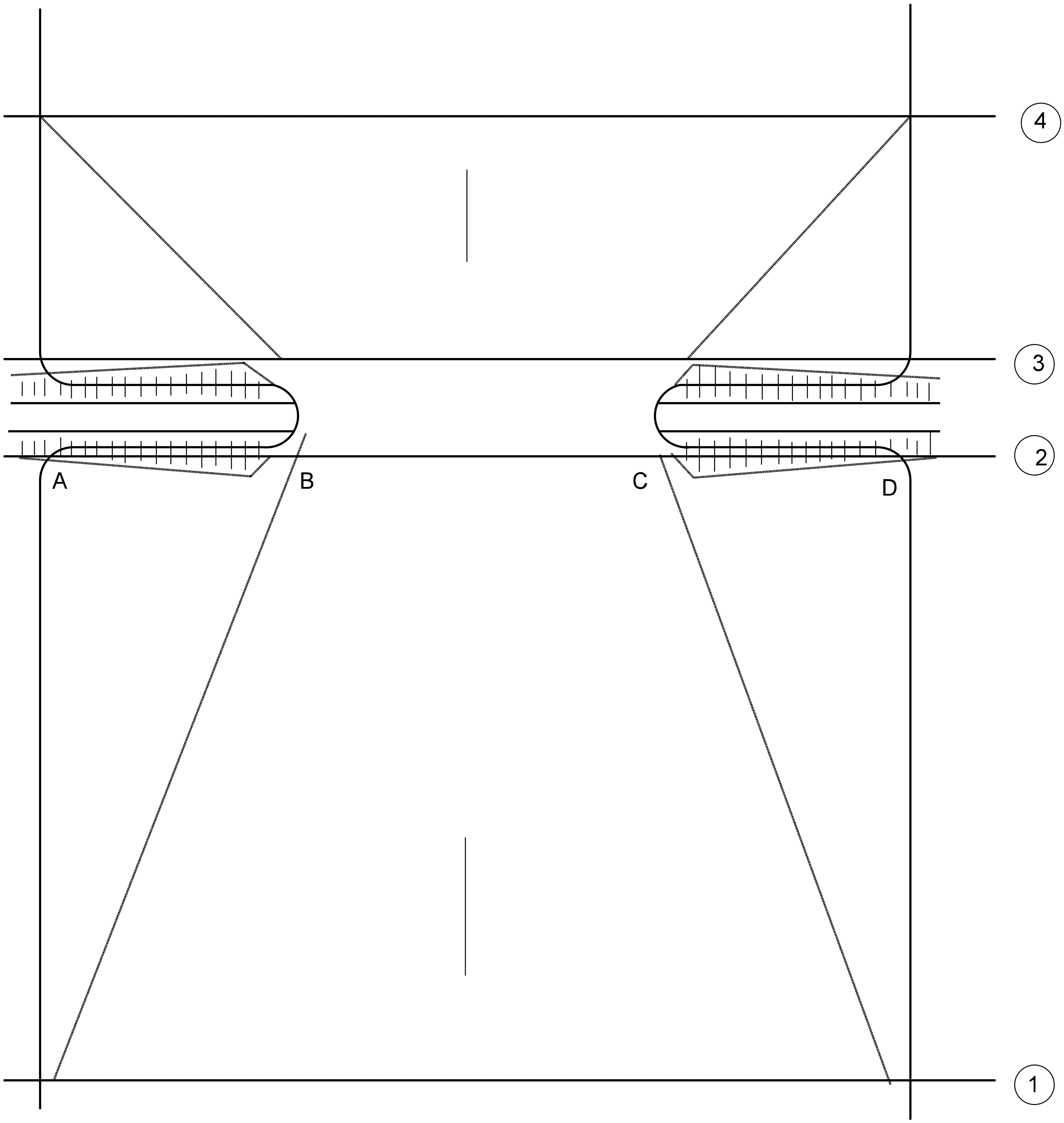
CROSS SECTION LOCATIONS FOR STREAM CROSSING
WITH A SINGLE WATERWAY OPENING

Figure 32-4C

Flow Through Bridge Opening Only	Flow Through Bridge Opening, and Over Roadway Grade
Class 1 – Free-surface flow	Class 4 – Free-surface flow
Class 2 - Orifice flow	Class 5 - Orifice flow
Class 3 – Submerged-orifice flow	Class 6 – Submerged-orifice flow

FLOW CLASSIFICATION ACCORDING TO SUBMERGENCE CONDITIONS
(WSPRO User Instructors Manual, 1990)

Figure 32-4D



CROSS SECTION LOCATIONS IN THE VICINTY OF BRIDGES

Figure 32-4E

Design Procedure Checklist

Route: Des No. Project No.
County: City or Town:
Description: Designer:

MAPS

- ☐ USGS Quad. Scale Date
- ☐ Flood-Hazard Delineation (Quad.)
- ☐ Floodplain Delineation (HUD)
- ☐ Flood-Insurance Firm and FHBM
- ☐ Local Land Use
- ☐ Soils Map
- ☐ Geologic Maps
- ☐ Aerial PhotosScale Date

STUDIES BY EXTERNAL AGENCIES

- ☐ USACE Floodplain Information Report
- ☐ NRCS Watershed Studies, PFP-HYDRO
- ☐ Local Watershed Management
- ☐ USGS Gages and Studies
- ☐ Interim Floodplain Studies
- ☐ Water Resources Data
- ☐ Regional Planning Data
- ☐ Forestry Service Gaging Da
- ☐ FEMA Flood-Insurance Studies

STUDIES BY INTERNAL SOURCES

- ☐ Hydraulics-Team Records
- ☐ Flood Record (High Water, Newspaper)

☐ **BEIDGE INSPECTION REPORTS**

CALIBRATION OF HIGH-WATER DATA

- ☐ Discharge and Frequency of H.W. el.
- ☐ Influences Responsible for H.W. el.
- ☐ Analyze Hydraulic Performance of Existing Facility for 100-Year Flood
- ☐ Analyze Hydraulic Performance of Proposed Facility for 100-Year Flood
- ☐ Reconnaissance Revisions Report

DESIGN APPURTENANCES

- ☐ Dissipators, Riprap
- ☐ Scour Analysis/Evaluation
- ☐ Utility-Company Plans

TECHNICAL RESOURCES

- ☐ *Indiana Design Manual*, Part IV
- ☐ Technical Library

DISCHARGE CALCULATIONS

- ☐ Drainage Areas
- ☐ Formula
- ☐ HEC-1 / TR-20
- ☐ NRCS
- ☐ Regional Analysis
- ☐ Regression Equations
- ☐ Area-Discharge Curves
- ☐ Log-Pearson Type III Gage Rating

HIGH-WATER ELEVATIONS

- ☐ INDOT Survey
- ☐ External Sources
- ☐ Personal Reconnaissance

FLOOD HISTORY

- ☐ Erosion and Sediment Control
- ☐ External Sources
- ☐ Personal Reconnaissance
- ☐ Maintenance Records

DESIGN PROCEDURE CHECKLIST

Figure 32-7A

DATA REPORTS

- ☐ INDOT Data

ENVIRONMENTAL REPORTS

- ☐ INDOT

TECHNICAL AIDS

- ☐ *Indiana Design Manual*, Part IV
- ☐ INDOT and FHWA Directives
- ☐ Technical Library

COMPUTER PROGRAMS

- ☐ HY8, CDS
- ☐ Direct-Step Water-Surface Profile
- ☐ USACE HEC-2 Water-Surface Profile
- ☐ FHWA Bridge Backwater
- ☐ Log-Pearson Type III Analysis
- ☐ WSPRO Water-Surface Profile
- ☐ PFP-HYDRA
- ☐ FESWMS
- ☐ HEC-1 / TR 20
- ☐ HY-9 Scour Analysis
- ☐ USACE HEC-RAS River Analysis System
- ☐ BRISTARS

DESIGN PROCEDURE CHECKLIST

Figure 32-7A (Continued)

TABLE OF CONTENTS

LIST OF FIGURES

Figure Title

CHAPTER THIRTY-THREE

BRIDGE DECK DRAINAGE

33-1.0 INTRODUCTION

The objective of this Chapter is to present a sound, economic, and low-maintenance design for bridge-deck and bridge-end drainage facilities. The detrimental effects of runoff emphasize the importance of removing water from the bridge deck as quickly as practical. This highlights the need for an efficient drainage system that always works properly. Proper design provides benefits for traffic safety, maintenance, structural integrity, and aesthetics.

33-1.01 Scope

This Chapter provides guidelines and procedures for designing a bridge-deck drainage system, including illustrative and practical examples. It incorporates hydraulic capacity, traffic safety, structural integrity, and practical maintenance. System hardware components, such as inlets, pipes, and downspouts, are described. It provides guidance for selecting a design gutter spread and flood frequency, and a design methodology for inlet spacing including example problems.

33-1.02 Objectives

The objectives in developing a design that will control the spread of water into traffic lanes, will function properly with a minimum of maintenance, and will not interfere with the architectural beauty or structural integrity of the bridge are discussed below.

33-1.02(01) Minimization of Maintenance

A bridge may not require a drainage system. This is ideal because of the high maintenance required to keep deck drains and other types of inlets free from debris discarded from vehicles and sand deposited during winter-maintenance activities. A clogged deck drain can cause more encroachment of runoff onto the shoulder or travel lane than no drainage structure. Bridge deck drains may be necessary in addition to riprap turnouts to capture runoff from the bridge and convey it to an appropriate outlet.

33-1.02(02) Minimization of Spread

As water accumulates and spreads across the width of the gutter and into the travel lane, it can reduce service levels and cause safety problems. Where inlets are required, they must be adequately sized and spaced to remove runoff from the bridge deck before it encroaches onto the traveled way to the limit of the design spread.

This Chapter provides a procedure to determine the maximum length of deck allowable without exceeding the design spread. See Chapter Thirty-six for more information.

33-1.02(03) Avoidance of Hydroplaning

Precipitation produces sheet flow on pavement and gutter flow. If sheet flow or spread has sufficient depth, vehicle tires can separate from the pavement surface and produce hydroplaning. To reduce this risk, the drainage system must be designed to prevent the accumulation of significant depths of water.

33-1.02(04) Reduction of Icing Potential on Deck

A bridge deck is usually the first segment of a highway to become icy in cold weather. Adequate deck drainage through use of minimum grades and cross slopes is essential to prevent the accumulation and spreading of icy spots. Icing on a bridge deck caused by frost is difficult to prevent except through surface texture and maintenance practices. Proper signing to warn the motoring public of the potential of ice on a bridge may be appropriate.

33-1.02(05) Integration into Structural Dimensions

The deck-drainage system must satisfy the structural requirements of the bridge. Drainage details affect structural design. For example, inlets for a reinforced-concrete bridge deck must fit within the reinforcing-bars layout. If deck drainage is not needed, structural design is free of inlet details. The drainage system should prevent water, road salt, or other corrosives from contacting the structural components.

33-1.02(06) Aesthetics

A pipe system conveying water from deck inlets to natural ground can be affixed to the exterior surfaces of a bridge. However, encasing the piping in structural members may not be advisable because of potential freezing damage. Pipes affixed to exterior surfaces of a structure, running at odd angles, can present an unpleasant silhouette and detract from the bridge's architectural aesthetics. To avoid this, pipes can be located in slots up the back of the columns or can be hidden behind decorative pilasters.

33-1.02(07) Protection of Water Quality [Rev. Jan. 2011]

The number of drains that discharge directly into a waterway should be reduced. Where possible and practicable, stormwater treatment measures should be included to treat the stormwater runoff prior to entering the waterway. Additional treatment or additional measures may be required as a result of a commitment during early coordination or as a regulatory-agency-permit condition.

33-1.03 Concept Definitions

The following are definitions of concepts used in bridge-deck-drainage analysis and design.

1. Cleanout plug. A removable plug in the piping system that provides access to a run of piping for cleaning. It is located near a bend or Y-shaped intersection.

2. Cross Slope. The slope of the pavement cross section from the curb to the crown.

3. Drop Inlet. A drain that is used away from a bridge or at a bridge end. It is larger than an inlet chamber and is set in earth in the subgrade or shoulder of an approach embankment. It has a horizontal or near-horizontal opening.

4. Drain. A receptacle that receives and conveys water.

5. Drainage System. The entire arrangement of grates, drains, inlet chambers, pipes, gutters, ditches, outfalls, and energy dissipators necessary to collect water and convey it to a disposal point.

6. Grate. The ribbed or perforated cover of an inlet chamber that admits water and supports traffic loads. Grates are removable to allow access for maintenance.

7. Inlet Chamber. The small cast-iron, welded steel, or formed concrete compartment that is beneath a grate. Although set into the bridge deck, it is sometimes only an open hole in the deck.

8. Outlet Pipe. The pipe that leads the water away from an inlet chamber or drop inlet.

9. Runoff, Drainage. A liquid that can run off the roadway surface. Although the liquid is most-often water, it includes other liquids and dissolved solids that can make their way into the drainage system.

10. Scupper. A small opening in a curb or barrier through which water can flow from the bridge deck. The term is nautical and by analogy relates bridge-deck drains to openings in the sides of a ship at deck level to allow water to exit.

11. Spread. The top of water measured laterally from the bridge curb.

12. Storm Drain. A beneath-the-bridge and underground piping system that may connect to a municipal storm-drain system or may be a separate collection system for highway and bridge drainage.

33-2.0 SYSTEM COMPONENTS [REV. JAN. 2011]

A bridge-deck drainage system includes the bridge deck itself, bridge gutters, inlets, pipes, downspouts, and stormwater-pollution-prevention controls. The system is designed by the bridge designer and coordinated with the Hydraulics Team. Coordination of efforts is essential in designing the components of the system to satisfy the objectives described in Section 33-1.0.

33-2.01 System Requirements

The primary requirement of the drainage system is to remove rainfall-generated runoff from the bridge deck before it collects and spreads in the gutter to encroach onto the traveled way and exceed the limit of a design spread. To satisfy this objective, the drainage system must satisfy the design criteria described below.

33-2.01(01) Structural Considerations

The primary structural considerations in bridge-deck drainage-system design are as follows.

1. Inlet sizing and placement must be compatible with the structural reinforcement and components of a bridge.

2. The drainage system should be designed to deter flow from contacting vulnerable structural members and to minimize the potential of eroding embankments.

The structural and hydraulics engineer should work together to design a system that has the necessary hydraulic capacity and is compatible with structural elements. To avoid corrosion and erosion, the design must include proper placement of outfalls, including prevention of flow from splashing or being blown back onto support members. Water should be prevented from running down the joint between the pavement and the bridge and thereby undermining an abutment or wingwall.

33-2.01(02) Deck and Gutters

The bridge deck and gutters are surfaces that initially receive precipitation. If grades, super-elevations, and cross slopes are properly designed, water and debris are efficiently conveyed to the inlets or riprap turnouts. A bridge deck with a level profile grade or in a sag vertical curve has poor hydraulics and may cause ponded water and hydroplaning problems. Superelevation transitions through a level grade may cause water and ice problems as well. The cross slope is 2% on a tangent section. A level grade or a sag vertical curve is not allowed for a bridge on a new alignment. The desirable longitudinal grade for bridge-deck drainage is 0.5% or steeper. A flatter grade is permissible where it is not physically possible or economically desirable to satisfy the above criteria.

33-2.01(03) Drainage Appurtenances

These include inlets, grates, pipes, and downspouts. From the deck and gutters, water and debris flow to the inlets, through pipes and downspouts, and finally to the outfall. A number of grate and inlet box designs are available to discourage clogging. Collector pipes and downspouts with smooth Y-connections and bends help prevent clogging in mid-system. T-connections are discouraged because of their propensity to plug. Collector pipes need a slope of 2% to sustain a self-cleaning velocity. Cleanout plugs located near curves and Y-connections should be included to provide access to the pipe to facilitate cleaning. A storm-drainage system beneath the bridge may be necessary to transport runoff to side-ditch, storm-drain or stormwater detention facilities.

33-2.01(04) Riprap Turnout

See Figure 33-2A, Slopewall, Riprap, and Sodding Limits for Grade Separation Structure, for treatment for a grade separation. See Figures 17-4 I and 17-4J for treatment for a stream crossing.

33-2.01(05) Bridge-End Collector on Curbed Roadway

A drainage inlet placed at the end of the bridge is essential for proper drainage. A grate inlet, curb-opening inlet, or combination inlet may be used at a bridge end. The hydraulic characteristics of the inlet should be considered in selecting the type. An inlet placed on the upslope end of the bridge should be designed to collect all of the runoff upslope of the bridge. This will prevent the bridge-deck drains and inlets from being overtaxed from runoff entering onto the bridge from the approaches. A collector at the downslope end of the bridge should be designed to collect all of the flow not intercepted by the bridge inlet. A conservative design approach is to assume that 50% of the inlets on the bridge are plugged and to size the end collectors accordingly. If there are no bridge inlets, downslope inlets should be designed to intercept all of the bridge drainage. From the inlet structure, there must be a pipe, paved channel, or trough to transport the water down the face of the embankment.

33-2.01(06) Stormwater-Pollution-Prevention Controls [Added Jan. 2011]

Stormwater-pollution-prevention controls (SWPPP) should be designed to provide an average annual post-development TSS removal rate of 80%. All stormwater controls should satisfy the other design criteria described herein. Controls should not increase the spread of water into traffic lanes, should function properly with minimum maintenance, and should not interfere with the structural integrity of the bridge. Downspouts should not be allowed to discharge untreated flow, unless such discharge is collected and treated on the receiving body beneath the bridge. Direct discharge of untreated stormwater runoff is not allowed. Flow consolidation should be considered to reduce the number of multiple BMP installations and maintenance requirements.

33-2.02 Bridge-Deck Inlet

33-2.02(01) Types

A bridge-deck inlet must remove water from a deck within the limits of the allowable spread. Considering hydraulics, inlets should be large and widely separated. Considering the structure, inlets should be avoided or be as small and as few as practical. This Section describes the inlet designs used by INDOT and discusses the factors that affect inlet interception capacity. This Section discusses design features to help prevent clogging, and it provides guidance for determining inlet locations. The following inlets are currently in use.

1. Grate A. This grate fits onto roadway drain type SQ. It is a parallel bar grate and the most hydraulically efficient grate in use. The grate is 19 in. square. Because the width of the openings is 1 in., the grate is not considered bicycle-safe if placed with the bars parallel to the direction of traffic. However, it is feasible to use this grate where bicycle traffic is allowed on the bridge if the bars are placed perpendicular to the direction of travel. The perpendicular arrangement may substantially reduce the hydraulic capacity of

the grate, as shown on Figure 33-5C, Grate Inlet Frontal Flow Interception Efficiency. The outlet fitting is circular pipe, 6 in. diameter.

2. Grate D. This grate fits onto roadway drain type OS. This is a type C grate with parallel bars but has two transverse bars which prevent bicycle wheels from dropping into the inlet; therefore, it is considered bicycle-safe. The transverse bars reduce the hydraulic capacity of the grate. The grate dimensions are 19 in. wide by 20 in. long. The outlet fitting is circular pipe, 6 in. diameter.

3. Slab-Bridge Floor Drain. This deck drain is to be used with a reinforced-concrete slab bridge. The drain is a PVC pipe, 6 in. diameter, set into the deck. This drain has limited hydraulic capacity; therefore, the spacing will be much closer than that for grate type A or D. The standard spacing is approximately 72 in. A ½-in. depression, which extends 1 ft transversely from the face of the curb, slightly increases the capacity.

4. Curved-Vane Grate. This grate is to be used on a curbed roadway where the inlets are located off the bridge deck.

5. Concrete-Barrier-Railing Scupper. This device should be used only on a local public agency bridge with concrete-barrier railings.

33-2.02(02) Interception Capacity and Efficiency

Inlet interception capacity is the flow intercepted by a bridge deck inlet under a given set of conditions. The efficiency of an inlet is the percent of total flow that the inlet will intercept. The efficiency of an inlet varies with cross slope, longitudinal slope, total gutter flow and, to a lesser extent, pavement roughness. Efficiency, *E*, is defined as follows:

$$E = \frac{Q_i}{Q} \qquad \text{(Equation 33-2.1)}$$

Where: Q = Total gutter flow, ft^3/s
Q_i = Intercepted flow, ft^3/s

The intercepted flow consists of frontal flow entering the inlet parallel to the gutter and flow entering from the side of the inlet. For a small, rectangular inlet, side flow is assumed to be small. The ratio of side flow intercepted to total side flow, R_s, is defined as follows::

$$R_s = \frac{1}{1 + \left(\dfrac{0.15 V^{1.8}}{S_x L_g{}^{2.3}} \right)} \qquad \text{(Equation 33-2.2)}$$

Where: L_g = Length of the inlet parallel to the flow, ft
V = Average velocity in the gutter, ft/s
S_X = Pavement cross slope, ft/ft

Because the side flow is small compared to the total flow, the inclusion of side flow is at the discretion of the designer. Equation 33-2.3 describes the ratio of frontal flow to total gutter flow as follows:

$$E_O = 1 - \left[1 - \left(\frac{W}{T} \right) \right]^{2.67} \qquad \text{(Equation 33-2.3)}$$

Where: W = Width of the inlet, ft
T = Width of the design spread, ft

Figure 33-5B provides a solution to Equation 33-2.3. The fraction of frontal flow that actually enters the inlet can be expressed as follows:

$$R_f = 1 - 0.09 (V - V_O) \qquad \text{(Equation 33-2.4)}$$

Where: R_f = Frontal flow capture fraction
V = Gutter velocity, ft/s
V_O = Grate splashover velocity, ft/s

See Figure 33-5C, Grate Inlet Frontal Flow Interception Efficiency, for a nomograph solution.

33-2.03 Inlet Location

33-2.03(01) Factors

The deck-spread criteria and geometric controls will determine the optimal hydraulic location of inlets. See Section 33-4.0. However, structural constraints, maintenance considerations, and other factors will influence the actual locations of inlets. For a pavement on grade, design spread will determine the distance between grates. Gutter flow should be intercepted on a horizontal curve or within a superelevation transition to minimize water flow across the bridge deck.

33-2.03(02) Spacing

Determining the maximum spacing between inlets is straightforward if the drainage area consists of pavement only, or has reasonably uniform runoff characteristics and is regularly shaped. This assumes that the time of concentration, t_C, is the same for all evenly-spaced inlets. See Section 33-5.0 for inlet-spacing calculations.

33-2.03(03) Sag Vertical Curve

Where ponding can occur in a sag vertical curve, flanking inlets should be placed on each side of the inlet at the low point of the sag. The flanking inlets should be placed so that they will limit spread on a low-gradient approach to the level point and act in relief of the inlet at the low point if it becomes clogged, or if the design storm is exceeded. See Section 36-10.0.

33-2.03(04) Minimum Number and Location

The following applies to inlet location.

1. Structure with Curbs.

    a. For a structure of less than 170 ft length and on grade, or a structure of less than 250 ft length and on a crest vertical curve, inlets are not required. However, hydraulic calculations for deck drains are required.

    b. For a structure on grade, provide roadway drains at the low end of the structure and riprap turnouts at the ends of the railing transitions.

    c. For a structure on a crest vertical curve, provide a circular roadway drain on each corner of the bridge and riprap turnouts at the end of the railing transitions.

    d. For a structure on a sag vertical curve, a minimum of two roadway inlets are required.

2. Structure Without Curbs. Inlets or riprap turnouts are not required.

3. Grade-Separation Structure. If deck drainage is required at the ends of the bridge, deck drains should discharge into inlets located in the berm or on the slopewall under the bridge as shown on the INDOT *Standard Drawings*.

33-2.03(05) Coordination with Deck Reinforcement

Inlet locations must be considered in the design and layout of the reinforcement spacing within the deck and must not promote corrosion of the structural members. Additional reinforcement should be provided around the deck drain to maintain the continuity of the deck reinforcement.

33-2.03(06) Maintenance

An inlet should be placed where it can be serviced easily and safely. A difficult-to-reach inlet will be neglected and inevitably become plugged. An inlet placed in a traffic lane may plug due to vehicles forcing debris into it.

33-2.04 Underdeck Collectors and Discharge System

33-2.04(01) Design

The following applies to the design of the underdeck drainage system.

1. General. A bridge-drainage pipe beneath the deck is sized larger than needed for hydraulic purposes to facilitate maintenance. The minimum pipe diameter is 6 in. The inlet conditions will control the flow capacity. Entrances, bends, and junctions in the underdeck pipe system provide opportunities for debris to snag and collect. Provide smooth transitions and smooth interior surfaces. Avoid sharp bends, corner joints, or bevel joints.

2. Velocity. The recommended minimum velocity for storm drainage is 2.67 ft/s. Because vertical fall for a pipe beneath a bridge is available, an 8% slope is the minimum required to transport sand and silt through the pipe at over 2.67 ft/s.

3. Standard Details. The details for cast-iron roadway drains used by the Department are shown on the INDOT *Standard Drawings*.

4. Alternative Design. Figure 33-2B, Ratio of Frontal Flow to Total Gutter Flow, Rectangular Inlet, illustrates two alternatives to drains. Figure 33-2B(a) shows a traditional arrangement including a short overhang and a steel beam, which permits the drain pipe to be located internally with reference to the external beam. Figure 33-2B(b) shows another arrangement including a large overhang and a bulb-tee beam, which locates the drain pipe to the outside. This is aesthetically less pleasing, therefore emphasizing the desirability of keeping the number of drains to a minimum.

5. Drain Location, Exterior Beam. A drainage casting should be positioned such that the outlet pipe is located inside the exterior beam, if practical. See Figure 33-2B(a). If not, the casting type and position should be selected to locate the drainage pipe as close as practical to the exterior beam. The plans should show the drain location, positioning, and attachment details.

6. Longitudinal Pipe, Closed Drainage. This pipe must not extend below the superstructure. The minimum slope is 1% for a longitudinal pipe between drains or from a drain to the point of discharge.

7. Overpass. An open deck drain should not be located over a roadway, sidewalk, or railroad. If a drain must be located in one of these areas, a closed drainage system should be provided.

33-2.04(02) Free-Fall

The following applies to free-fall, where used beneath a bridge.

1. The downspout should be extended 6 in. below the beam soffit. The downspout should be placed approximately 10 ft from the face of a substructure unit, unless a closed drainage system is used. A downspout should not interfere with the required horizontal or vertical clearances. A pipe system designed to bring water down to ground level may become clogged with debris and ice and should only be used as the last option.

2. A downspout should not discharge water where such water can be windblown and flow down a column or pier.

3. Water should not be discharged openly over a traveled way (vehicular, railroad, or pedestrian), unpaved embankment, or unprotected ground where it can cause erosion or undermine a structural element. An energy dissipater or riprap should be provided to prevent erosion.

4. A free fall exceeding 25 ft will sufficiently disperse the falling water so that erosion damage will not occur beneath the bridge. Where the water freefalls onto riprap or flowing water a lesser freefall will be permissible.

33-2.04(03) Cleanout

A cleanouts for maintenance access should be provided at key points within the system to facilitate the removal of obstructions. A downspout should be located so that a maintenance crew can access it from underneath the bridge and preferably from the ground. The most convenient arrangement should be made, as a cleanout that is inaccessible or difficult to reach will not be cleaned.

33-3.0 CALCULATION OF RUNOFF

33-3.01 Rational Method

The conventional method for estimating runoff for bridge-deck drainage is the Rational Method. This hydrologic method is discussed in detail in Chapter Twenty-nine. The Rational Method is discussed below as it pertains to bridge -deck drainage.

The Rational formula is as follows:

$$Q = CiA \qquad \text{(Equation 33-3.1)}$$

Where: Q = Peak runoff rate, ft^3

C = A dimensionless runoff coefficient that represents characteristics of the drainage area. C = 0.9 for a bridge deck.

i = Rainfall intensity, in./h

A = Drainage area, ac

33-3.02 Rainfall Intensity

For a bridge deck, the determination of the design rainfall intensity for use in the Rational Method can be based on the following:

1. the time of concentration, which is a function of the design spread;
2. the specific intensity at which a motorist's vision will be impaired; or
3. the condition for which hydroplaning will occur.

The source of rainfall intensity data is from Intensity-Duration-Frequency (IDF) curves which can be plotted for the location in question. Other sources of rainfall intensity data are available and will be discussed below.

The design rainfall intensity can be obtained from IDF Curves for the exact location (latitude and longitude) of the project. This is readily available from the HYDRO module of the computer program HYDRAIN. Regional IDF curves are included in Chapter Twenty-nine. The rainfall intensity obtained from the IDF curve is a conservative design value if compared with values obtained from hydroplaning or motorist-vision criteria. Because these latter criteria involve variable factors, such as tire tread wear and driver behavior, one approach is to use the IDF curve as the governing criteria for selecting rainfall intensity and then compare that intensity to the other two criteria. The most conservative rainfall intensity value should be used.

For non-freeway bridge-deck drainage, the design flood for determining the spacing and location of inlets is based on a 10-year return period. For a freeway, the return period is 50-years. Rainfall intensity of 5 in./h to 6 ½ in./h for time of concentration of 5 to 10 min during a 10-year frequency storm is not unusual.

33-3.03 Time of Concentration

HEC 12 *Drainage of Highway Pavements* assumes that inlets are independent drainage elements that collect runoff from their small contributing drainage areas. This assumption yields a conservative and constant time of concentration for equally-spaced deck inlets or scuppers, and equals the time of concentration to the first inlet.

The time of concentration for bridge-deck inlets consists of overland-flow time and gutter-flow time. The overland flow is sheet flow from the deck high point to the gutter. Gutter-flow time is the time of flow in the gutter.

33-3.03(01) Overland Flow Time of Concentration

The following equations should be used for determining time of concentration for overland flow, t_O.

$$t_O = \frac{k_W (n l_O)^{0.6}}{(Ci)^{0.4} S^{0.3}} = \frac{0.93 (n W_p)^{0.6}}{(Ci)^{0.4} S^{0.3}} \qquad \text{(Equation 33-3.2)}$$

Where:

- t_O = Time of overland flow, min
- l_O = W_p = Overland, flow length, ft
- n = Manning's roughness coefficient
- C = Runoff coefficient
- i = Rainfall intensity, in./h
- S = Average slope of the overland area
- k_W = 0.93, a constant

33-3.03(02) Gutter Flow Time of Concentration

The following equation should be used for determining time of concentration for gutter flow, t_g.

$$t_g = k_g \left(\frac{S_X T^2}{CiW_p} \right) \qquad \text{(Equation 33-3.3)}$$

Where:

t_g	=	Time of gutter flow, min
S_X	=	Cross slope of gutter
T	=	Spread, ft
C	=	Runoff coefficient
i	=	Rainfall intensity, in./h
W_P	=	Width of pavement contributing runoff, ft
k_g	=	484, a constant

Figure 33-3A illustrates the cross-section elements which apply to Equation 33-3.3.

33-3.03(03) Total Time of Concentration

The total time of concentration, t_C, is the sum of t_O and t_g.

33-4.0 FLOW IN A GUTTER

A bridge-deck gutter is defined as the section of pavement adjacent to the curb or parapet that conveys water during a storm-runoff event. It may include a portion or all of a travel lane. A gutter cross section has a triangular shape with the curb forming the near-vertical leg of the triangle. A bridge-deck gutter has a straight cross slope.

Chapter Thirty-six discusses gutter flow in detail relative to pavement drainage. Gutter flow is discussed below as it pertains to bridge-deck drainage.

For a bridge deck, a modification of the Manning equation is necessary for use in computing flow in a triangular channel because the hydraulic radius in the equation does not adequately describe the gutter cross section, especially where the top width of the water surface may be more than 40 times the depth at the curb. The resulting equation is as follows:

$$Q = \left(\frac{k_g}{n} \right) S_x^{\,1.67} S^{0.5} T^{2.67} \qquad \text{(Equation 33-4.1)}$$

Where:

Q	=	Flow rate, ft^3/s
k_g	=	0.56, a constant
T	=	Width of flow (spread), ft

S_X = Cross slope
S = Longitudinal slope
n = Manning's roughness coefficient

Figure 33-4A illustrates the cross section which applies to Equation 33-4.1.

See Figure 36-7A for the criteria for design frequency and allowable water spread, *T*.

Gutter velocity is determined by dividing the gutter-flow equation by the cross-sectional area of the gutter. The resulting relation is as follows:

$$V = \left(\frac{2kg}{n}\right) S^{0.5} S_x^{\;0.67} T^{0.67} \qquad \text{(Equation 33-4.2)}$$

Where: V = Gutter velocity, ft/s

33-5.0 INLET SPACING

Chapter Thirty-six discusses inlet spacing in detail for pavement. This Section provides methods specifically for determining inlet spacing for a constant-grade, flat, or vertically-curved bridge. Example problems are also included.

33-5.01 Constant-Grade Bridge

33-5.01(01) Procedure

An IDF curve for the appropriate location will be necessary. HYDRO can be used to generate an IDF curve for a known latitude and longitude, or the designer may use the closest regional IDF curve shown in Chapter Twenty-nine. The designer must select a return period (10 years) and design spread (see Section 33-4.0 and Chapter Thirty-six). If the bridge slope is nearly flat (less than 0.3%), the procedure for a flat bridge should also be followed as a check. The hydraulic procedure is to start at the high end of the bridge and work downslope from inlet to inlet. Use the following procedure.

1. Step 1. An iterative process is necessary to determine rainfall intensity, *i*, because this value is necessary to solve both Equations 33-3.2 and 33-3.3. Assume a value of *i* and solve for overland time of concentration using Equation 33-3.2, and gutter-flow time of concentration using Equation 33-3.3. Add the times together for the total time. If it is less than 5 min, use 5 min. Compare with the assumed value and repeat the process if not sufficiently close. Select a design rainfall intensity from the IDF data.

2. Step 2. Find the flow on the deck, Q, at design spread, T, using Equation 33-4.1.

3. Step 3. Starting at the high end of the bridge, the inlet spacing, L_C, can be computed using Equation 33-5.1 as follows:

$$L_C = \frac{(4.356 \, x \, 10^4)EQ}{CiW_p} \quad \text{(Equation 33-5.1)}$$

Where:
- L_C = Distance to the first inlet or between inlets, ft
- i = Design rainfall intensity, in/h (Step 1)
- Q = Gutter flow, ft^3/s (Step 2)
- C = Rational runoff coefficient
- W_p = Width of pavement contributing to gutter flow, ft
- E = Constant equal to 1 for first inlet, equal to capture efficiency for subsequent inlets

$L_O = L_C$ for the first inlet; $L_C = L_C$ for the others.

2. *For vertical curve, $E = K$; $K = 1$, $L_O = L_C$ for the first inlet; $L_O = L_C$, for others.*

4. Step 4. Compare L_C with the length of the bridge. If L_C is greater than the length of the bridge, inlets are not needed and only bridge end treatment is needed. If L_C is less than the bridge length, go to Step 5.

5. Step 5. If inlets are required, the designer should calculate the constant inlet spacing, L_C, for the subsequent inlets. To do this, it will be necessary to determine the capture efficiency, E, for the type of inlet that is proposed for use. For type SQ inlets where the grate is transverse to the direction of travel for a bicyclist, a reduction in capture efficiency will be required as indicated in Figure 33-5C.

   For a circular drain, such as a slab-bridge floor drain, Figure 33-5A, Efficiency Curves for Circular Drain, summarizes results from a laboratory study. To use the figure, calculate the ratio of inlet diameter, D, to gutter spread, and enter the graph at the appropriate value along the *x*-axis. Upon interception with the applicable curve (or appropriate interpolated curve), read efficiency, E, from the *y*-axis.

   For a rectangular inlet, the steps necessary are shown below to calculate flow interception efficiency, E, which is the ratio of intercepted to total deck flow.

a. Find the ratio of frontal flow, E_O, bound by the width of grate, W, to total deck flow, using Figure 33-5B, Ratio of Frontal Flow to Total Gutter Flow (Rectangular Inlet), or the following:

$$E_O = 1 - \left(1 - \frac{W}{T}\right)^{2.67} \qquad \text{(Equation 33-5.2)}$$

b. Find the flow intercepted by the inlet as a percent of the frontal flow. The gutter velocity is needed and is provided by Equation 33-4.2.

c. Identify the grate type, and, using Figure 33-5C, Grate Inlet Frontal Flow Interception Efficiency, determine the portion of frontal flow, R_f, the total flow within a grate width from the curb, which is intercepted by a grate. This will be less than 100% if the gutter velocity exceeds the splashover velocity.

d. The interception efficiency, E, is then computed as follows:

$$E = R_f E_O \qquad \text{(Equation 33-5.3)}$$

If side flow is to be considered, see Chapter Thirty-six.

e. The flow intercepted by an inlet is as follows:

$$Q_i = EQ_w \qquad \text{(Equation 33-5.4)}$$

Where $Q_w = QE_o$

f. The flow bypassing an inlet is as follows:

$$Q_b = Q[E_o(1 - E) + (1 - E_o)] \qquad \text{(Equation 33-5.5)}$$

6. Step 6. After E is determined, solve for L_C, spacing of all inlets using Equation 33-5.1. Because bridge-deck grade and time of concentration are assumed to be constant, the spacing between inlets will be constant.

7. Step 7. Continue to space inlets until the end of the bridge is reached. Once L_O and L_C have been determined analytically, these values may need to be adapted to accommodate structural and aesthetic constraints.

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33-5.01(02) Example 33-5.1

Given: Two-lane urban arterial
120-m bridge with 0.5% grade (S_O = 0.005)
Latitude 40 deg 0 min, and longitude 86 deg 0 min
W_p = 20 ft from the centerline crown to the gutter edge
C = 0.9
T = 8-ft shoulder + 4-ft allowable encroachment = 12 ft
n = 0.016
S_X = 0.02
10-year return period

Inlets, if provided, will be deck drain type SQ with grate type A. The bridge has a waterproof expansion joint. All upslope pavement drainage is intercepted by bridge end collectors.

Find: Inlet spacing, L_O, L_C.

Solution: Use the procedure for a constant-grade bridge.

1. Step 1. Compute intensity, i, for time of concentration, t_C, to first inlet. Figure 33-5D, IDF Data from HYDRO (40 deg latitude and 86 deg longitude), reproduces the IDF data from HYDRO for the given latitude and longitude for the 10-year return period. Use the IDF data and the time-of-concentration equations shown in Section 33-3.0 in the iterative process.

   a. Select a trial value for t_C of 10 min and verify this assumption.

   b. From the IDF data, i_{10} for 10 min duration is 5.44 in./h.

   c. Compute the overland flow time of concentration using Equation 33-3.2 as follows:

$$t_O = 0.93 \frac{(20)^{0.6}(0.016)^{0.6}}{[(0.9)(5.44)]^{0.4}(0.02)^{0.3}} = 80 \text{ min}$$

   d. Compute gutter-flow time of concentration using Equation 33-3.3. Because the upslope bridge end inlet intercepts all approach flow, E = 1.

$$t_g = 484 \frac{(0.02)\,(12)^2}{(0.9)(5.44)(20)} = 14.2 \text{ minutes}$$

e. Compute total t_C and compare with selected value as follows:

$$t_c = 0.80 + 14.2 = 15.0 \text{ minutes}$$

f. Because the trial value of 10 min and the computed value of 15 min are not equal, select another trial t_C value of 19 min and repeat steps c through e. The interpolated value of i from Figure 33-5D, IDF Data from HYDRO (40 deg latitude and 86 deg longitude), for a duration of 19 min is 4.2 in./h.

g. The computed time of concentration for the second trial duration is 19.02 min, which is very close to 19 min. Therefore, use i = 4.2 in./h as the design rainfall intensity.

2. Step 2. Compute full-gutter flow based on the design spread of 12 ft. Use Equation 33-4.1 as follows:

$$Q_f = \left(\frac{0.56}{0.016}\right)(0.02)^{1.67}(0.005)^{0.5}(12)^{2.67} = 2.74\ ft^3/s$$

3. Step 3. Starting at the upslope end of the bridge, compute the distance to the first inlet, L_O, using Equation 33-5.1 with E = 1.

$$L_O = \frac{(46250)(2.74)}{(0.9)(4.2)(20)} = 1579\ ft$$

4. Step 4. Compare L_O = 1579 ft with the bridge length of 400 ft. Because L_O is greater than the total bridge length of 400 ft, drainage inlets are not required on the bridge.

5. Step 5. Not applicable.

6. Step 6. Not applicable.

7. Step 7. Not applicable.

* * * * * * * *

33-5.01(03) Example 33-5.2

Given: 6-lane rural freeway
2200-ft bridge on 1% grade (S_O = 0.01)
Latitude 40 deg 0 min, and longitude 86 deg 0 min
W_P = 34 ft from the centerline crown to the gutter edge

$n = 0.016$
$C = 0.9$
$S_X = 0.02$
T = 10 ft to edge of traveled way
10-year return period

Bicycle traffic is not allowed on the bridge. Therefore, inlets will be deck drain type SQ with grate type A.

The bridge has a waterproof expansion joint. All upslope pavement drainage is intercepted by a bridge-end collector.

Find: Inlet spacing, L_O, L_C.

Solution: Use the procedure for a constant-grade bridge.

1. Step 1. Compute intensity, i, for time of concentration, t_c, to the first inlet. Use the IDF data in Figure 33-5E, Parameters for Equation 33-5.6, and the time-of-concentration equations in Section 33-3.0 in an iterative procedure.

   a. Select trial value for t_C of 6 minutes and verify the assumption.

   b. The i for duration of 6 min is 6.48 in./h from the IDF data.

   c. Compute overland flow time of concentration using Equation 33-3.2 as follows:

$$t_O = 0.93 \frac{(34)^{0.6} (0.016)^{0.6}}{[(0.9)(6.48)]^{0.4} (0.02)^{0.3}} = 1.03 \text{ minutes}$$

   d. Compute gutter-flow time of concentration using Equation 33-3.3. Because the upslope bridge-end inlet intercepts all approach flow, E = 1.

$$t_g = 484 \frac{(0.02)(10)^2}{(0.9)(6.48)(34)} = 4.88 \text{ minutes}$$

   e. Compute total t_C and compare it with the selected trial value as follows:

$$t_C = 1.03 + 4.88 = 5.91 \text{ minutes}$$

   f. Because the trial value and the computed value are approximately equal, use i = 6.48 in./h as the design rainfall intensity.

2. Step 2. Compute full-gutter flow for the design spread of 10 ft. Use Equation 33-4.1 as follows:

$$Q_f = \left(\frac{0.56}{0.016}\right)(0.02)^{1.67}(0.01)^{0.5}(10)^{2.67} = 2.38\ ft^3/s$$

3. Step 3. Starting at the upslope end of the bridge, compute the distance to the first inlet, L_O, using Equation 33-5.1 as follows:

$$L_O = \frac{(43560)(2.38)(1)}{(0.9)(6.48)(34)} = 522\ ft$$

4. Step 4. Because L_O of 522 ft is less than the total bridge length of 2250 ft, inlets are needed.

5. Step 5. Determine the inlet efficiency, E, for the rectangular inlets:

   a. Using Equation 33-5.2, compute the frontal flow ratio, E_O, as follows:

   $$E_O = 1 - \left(1 - \frac{1.6}{10}\right)^{2.67} = 0.37$$

   b. Using Equation 33-4.2, compute gutter velocity as follows:

   $$V = \frac{(2)(0.56)}{0.016}(0.01)^{0.5}(0.02)^{0.67}(10)^{0.67} = 2.38\ ft/s$$

   c. Using Figure 33-5C, find the frontal-flow intercept efficiency, R_f, for a parallel bar grate, L = 1.6 ft. The splash over velocity is approximately 4.3 ft/sec, which is greater than the gutter velocity which implies R_f = 1.0.

   d. Therefore, the interception efficiency from Equation 33-5.3 is as follows:

   $$E = (0.37)(1) = 0.37$$

6. Step 6. Compute constant spacing for the remainder of the inlets using Equation 33-5.1 as follows:

$$L_C = \frac{(43560)(2.38)}{(0.9)(6.48)(34)}(0.37) = 193\ ft$$

7. Step 7. Adapt spacing to structural constraints. For example, if bent spacing is 183 ft to 200 ft, place the first inlet at the second bent approximately 366 ft to 400 ft from the high

end of the bridge. Place an inlet at each bent thereafter for a total of 10 or 11 inlets, depending on the bent spacing.

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33-5.02 Flat Bridge

33-5.02(01) Procedure

A bridge in a vertical curve, either sag or crest, is nearly flat at its low- or high-point station. A bridge with a grade of less than about 0.3% is nearly flat. For a nearly-flat station on a vertically-curved bridge, or a bridge with a constant grade, the designer should check spacing assuming that the bridge is flat. If the flat-bridge spacing is less than the spacing determined using a nearly flat grade, the flat-grade spacing is warranted. Use the following procedure.

1. Step 1. The time of concentration, t_C, to each inlet is assumed to be 5 min. Frequency, design spread, T, pavement width, W_P, bridge length, L_B, Manning's n, Rational runoff coefficient, C, and gutter slope, S_X, are assumed to be known. Using a time of concentration of 5 min and the selected frequency, rainfall intensity is determined from HYDRO or from the regional IDF curves shown in Chapter Twenty-nine.

2. Step 2. Constant inlet spacing, L_C, can then be computed using Equation 33-5.6, as follows:

$$L_C = \left[\frac{1312}{\left(nCiW_p\right)^{0.67}} \right] S_x^{\,1.44} T^{\,2.11} \qquad \text{(Equation 33-5.6)}$$

   Where variables are as previously defined. Figure 33-5E illustrates the parameters for Equation 33-5.6.

3. Step 3. Compare the computed spacing with the known bridge length. If L_C is greater than the length of the bridge, there is no need for inlets and the designer need only be concerned with the design of bridge end treatments (Step 6). If L_C is less than the bridge length, the total needed inlet perimeter, P, can be computed using Equation 33-5.7, which is based on critical depth along the perimeter of the inlet (weir flow), as follows:

$$\mathrm{P} = \frac{(\mathrm{CiW}_p)^{0.33}\,\mathrm{T}^{0.61}}{102.5\,\mathrm{S}_x^{0.06}\,\mathrm{n}^{0.67}} \qquad \text{(Equation 33-5.7)}$$

Where variables as previously defined. Figure 33-5F illustrates the parameters for Equation 33-5.7.

4. Step 4. Select inlet based on perimeter requirements.

5. Step 5. Adapt spacing to accommodate structural and aesthetic constraints.

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33-5.02(02) Example 33-5.3

Given: 6-lane rural freeway
1000-ft bridge
Latitude 40 deg 0 min, and longitude 86 deg 0 min
W_p = 34 ft from centerline crown to the gutter edge
n = 0.016
C = 0.9
S_X = 0.02
T = 10 ft to edge of traveled way
10-year return period

Find: Inlet spacing, L_C, and total inlet perimeter, P.

Solution: Use the procedure for a flat bridge.

1. Step 1. Compute intensity for time of concentration of 5 min from the IDF data shown in Figure 33-5D, therefore, i = 6.72 in./h.

2. Step 2. Compute inlet spacing, L_C, using Equation 33-5.6 as follows:

$$\mathrm{L}_c = \left[\frac{1312}{[(0.016)(0.9)(6.72)(34)]^{0.67}}\right](0.02)^{1.44}(10)^{2.11} = 272\ ft$$

3. Step 3. Because L_C < 1000 ft, the length of the bridge, inlets are needed. Compute total inlet perimeter using Equation 33-5.7 as follows:

$$\mathrm{P} = \frac{[(0.9)(6.72)(34)]^{0.33}(10)^{0.61}}{102.5(0.02)^{0.06}(0.016)^{0.67}} = 4.65\,ft$$

4. Step 4. An inlet configuration may be used if the total inlet perimeter is at least 56 in. If a square or rectangular grate is adjacent to the curb, the sum of the other three sides

should equal 56 in. Thus, 56 / 3 = 18¾ in. Either deck drain type OS or SQ is adequate. A circular floor drain, such as the slab-bridge floor drain, can be used. Three 6-in. diameter drains are necessary at 267-ft intervals.

5. Step 5. Adapt spacing and type of inlet to structural constraints.

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33-5.03 Vertically-Curved Bridge

33-5.03(01) Procedure

The methodology for spacing inlets on a vertically-curved bridge is similar to that for a constant-grade bridge, except that a trial-and-error approach is incorporated into the inlet-spacing computations to reflect the estimated grade at the next inlet. The designer first selects a design frequency and design spread. Bridge dimensions, bridge-end grades, roughness and runoff coefficients, and inlet specifications are assumed to be given. Using basic geometry, the designer computes the distance from the high point to each end of the bridge, L_{E1}, L_{E2}. Use the following procedure.

1. Step 1. Compute the lengths of the short and long ends of the bridge, L_{E1}, L_{E2}, by solving Equation 33-5.8 as follows:

$$S = \left(\frac{g_2 - g_1}{L_B} \right) X + g_1 \qquad \text{(Equation 33-5.8)}$$

Where: L_B = length of bridge, ft
g_1, g_2 = slopes of the tangents of the vertical curve, decimals

Solving for X with $S = 0$ provides the distance from the left edge to the high point, L_{E1}.

2. Step 2. Determine the rainfall intensity based on the computed time of concentration to the first inlet as follows.

   a. Select trial time of concentration and determine rainfall intensity from the IDF data.

   b. Compute overland travel time, t_O, using Equation 33-3.2.

   c. Compute gutter travel time, t_g, using Equation 33-3.3.

d. Compute time of concentration by summing the gutter- and overland-travel times.

e. Compare computed time with trial time in Step 2.a. and repeat if necessary.

3. Step 3. Select a trial distance from the high point to the first inlet on the long end of the bridge, L_O, and compute the local slope using Equation 33-5.8.

4. Step 4. Compute gutter flow, Q_f, corresponding to the design spread using Equation 33-4.1.

5. Step 5. Compute the distance to the first inlet, L_O, letting K = 1 for the first inlet, and using Equation 33-5.1. Substitute K for E and L_O for L_C in the equation.

6. Step 6. Determine spacing to the next inlet on the long end of the bridge as follows.

   a. Select a trial L_1.

   b. Compute the local slope, S, using Equation 33-5.3.

   c. Calculate the gutter flow, Q, using Equation 33-4.1.

   d. Compute inlet efficiency, E, using Figure 33-5A for circular scuppers, Figure 33-5B, Equation 33-4.2, Figure 33-5C, and Equation 33-5.3 for rectangular inlets.

   e. Compute the interception, K. K is less than 1 for the inlets following the first one. Use Equation 33-5.9 as follows:

$$K = 1 - (1 - E)\left[\frac{S_u}{S}\right]^{0.5} \qquad \text{(Equation 33-5.9)}$$

   Where: $E = E_O$. R_f = interception efficiency
   S_u = Longitudinal grade for upstream inlet.

   f. Compute inlet spacing, L_1, using Equation 33-5.1 by substituting K for E and L_1 for L_C, and compare to the trial L_1 in Step 6.a. If the computed L_1 value does not equal the trial L_1 value, repeat Step 6.

7. Step 7. Repeat Step 6 for the next inlet. Inlet spacing is determined one at a time until the sum of the inlet spacings exceeds the length of the long side of the bridge. Spacing on the short side of the bridge, starting from the high point and working down, will be the same as that determined for the long side until, of course, the length of the short side is

exceeded. The spacing of the vertically-curved-deck inlets is symmetrical with respect to the high point of the bridge.

8. Step 8. Adapt spacing of inlets to accommodate structural constraints.

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33-5.03(02) Example 33-5.4

Given: 6-lane rural freeway
Bridge on vertical curve
Latitude 40 deg 0 min, and longitude 86 deg 0 min
W_p = 34 ft from centerline crown to gutter edge
n = 0.016
S_X = 0.02
T = 10 ft to edge of traveled way
C = 0.9
L_B = 2000 ft
10-year return period
g_1 = +0.01
g_2 = -0.02

Inlets will be deck drain type SQ. Bicycle traffic is not allowed.

Find: Inlet spacing L_O, L_1, L_2, L_3, etc.

Solution: Use the procedure for a vertically-curved bridge.

1. Step 1. Compute L_{E1} and L_{E2}, assuming a parabolic vertical curve. Locate the high point, X_{HP}, using Equation 33-5.30 with S = 0, as follows:

$$X_{HP} = (-0.01)\left[\frac{(2000)}{(-0.02 - 0.01)}\right] = 667 \ ft$$

Thus: L_{E1} = 667 ft and L_{E2} = 2000 – 667 = 1333 ft

2. Step 2. Compute intensity for time of concentration to the first inlet. Use the IDF data shown in Figure 33-5D as follows:

a. Select trial time of concentration of 6 min. Then i_{10} = 6.48 in./h for a 10-year return period.

b. Compute t_O from Equation 33-3.2 as follows:

$$t_O = 0.93 \frac{[(34)(0.016)]^{0.6}}{[(0.9)(6.48)]^{0.4}(0.02)^{0.3}} = 1.03 \text{ minutes}$$

c. Compute t_g from Equation 33-3.3 as follows:

$$t_g = 484 \frac{(0.02)(10)^2}{(0.9)(6.48)(34)} = 4.88 \text{ minutes}$$

d. Compute t_C as follows:

$$t_C = 1.03 + 4.88 = 5.91 \text{ minutes}$$

e. The calculated t_C is sufficiently close to the assumed t_C. Therefore, use 6.48 in./h as the design rainfall intensity.

3. Step 3. Select a trial value for the distance from the bridge's high point to the first inlet, working down the long side of the bridge, and compute the local slope.

a. Select L_O = 333 ft (1st trial)

$$S = \frac{(-0.02 - 0.01)}{2000}(1000) + 0.01 = -0.005$$

X = 667 + 333 = 1000 ft (distance from the left end)

b. Use Equation 33-5.8 to determine S as follows:

Use $|S|$ = 0.005

4. Step 4. Compute full gutter flow, Q_f, at design spread of 10 ft for Equation 33-4.1 as follows:

$$Q_f = \left(\frac{0.56}{0.016}\right)(0.02)^{1.67}(0.005)^{0.5}(10)^{2.67} = 1.68 \; ft^3/\text{s}$$

5. Step 5. Compute distance to first inlet, L_O. K = 1 for the first inlet. Use Equation 33-5.1 as follows:

$$L_O = \frac{(43560)(1.68)}{(0.9)(6.48)(34)} = 370 \; ft$$

Use L_O = 333 ft. Inlets are needed because L_O is less than the length of the long side of the bridge.

6. Step 6. Determine spacing to next inlet as follows.

a. Select L_1 = 100 ft (1st trial).

X = 667 + 333 + 100 = 1100 ft (distance from left end)

b. Use Equation 33-5.30 to determine S as follows:

$$S = \frac{(-0.02 - 0.01)}{2000}(1100) + 0.01 = -0.0065 \qquad \text{(Use } |S| = 0.0065\text{)}$$

S_u = prior S = 0.0050, the slope at immediately-upstream inlet.

c. Compute full gutter flow, Q_f, using Equation 33-4.1 as follows:

$$Q_f = \left(\frac{0.56}{0.016}\right)(0.02)^{1.67}(0.0065)^{0.5}(10)^{2.67} = 1.92\ ft^3/s$$

d. Compute inlet efficiency, E, for rectangular inlets using Equation 33-5.2 or Figure 33-5B.

$$E_O = 1 - [1 - 1.6/10]^{2.67} = 0.37$$

$$V = \left(\frac{1.12}{0.016}\right)(0.02)^{0.67}(0.0065)^{0.5}(10)^{0.67} = 1.92\ ft/s$$

R_f = 1.0 from Figure 33-5C, Grate Inlet Frontal Flow Interception Efficiency

$E = E_O R_f$ = 0.37 (1.0) = 0.37

From Figure 33-5C, splash over does not occur for parallel grates until a gutter velocity of 4.3 ft/s is reached corresponding to a slope of 1.9%. Thus, R_f will equal 1.0 and E will equal 0.37 for the remainder of this example. For less-efficient grates on a steep slope, E can change from inlet to inlet.

e. Compute interception coefficient, K, using Equation 33-5.9. K does not equal 1 for the second inlet.

$$K = 1 - (1 - 0.37)\left(\frac{0.005}{0.0065}\right)^{0.5} = 0.45$$

f. Compute inlet spacing, L_1, using Equation 33-5.1 as follows:

$$L_1 = \frac{(43\,560)(1.92)}{(0.9)(6.48)(34)}(0.45) = 190 \text{ ft (not equal to 100 ft as assumed)}$$

7. Repeat Step 6. Because the computed value for L_1 does not equal the trial value, select a new trial value for L_1 and repeat Step 6 as follows:

a. Select L_1 = 250 ft (2nd trial).

X = 667 + 333 + 250 = 1250 ft (Equation 33-5.10)

b. Use Equation 33-5.8 to redetermine S as follows:

$$S = \frac{(-0.02 - 0.01)}{2000}(1250) + 0.01 = -0.00875 \text{ (use } |S| = 0.00875)$$

S_u is still equal to 0.005.

c. Recompute Q as follows:

$$Q = \left(\frac{0.56}{0.016}\right)(0.02)^{1.67}(0.00875)^{0.5}(10)^{2.67} = 2.23\, ft^3/\text{s} \qquad \text{(Equation 33-5.11)}$$

d. Recompute inlet efficiency, E, as follows:

E = 0.37 (no change as noted) (Equation 33-5.12)

e. Recompute interception coefficient, K_1, as follows:

$$K = 1 - (1 - 0.37)\left(\frac{0.005}{0.00875}\right)^{0.5} = 0.524 \qquad \text{(Equation 33-5.13)}$$

f. Recompute inlet spacing, L_1, as follows:

$$L_1 = \frac{(43560)(2.23)}{(0.9)(6.48)(34)}(0.524) = 256 \text{ ft} \qquad \text{(Equation 33-5.14)}$$

This is sufficiently close to 250 ft.

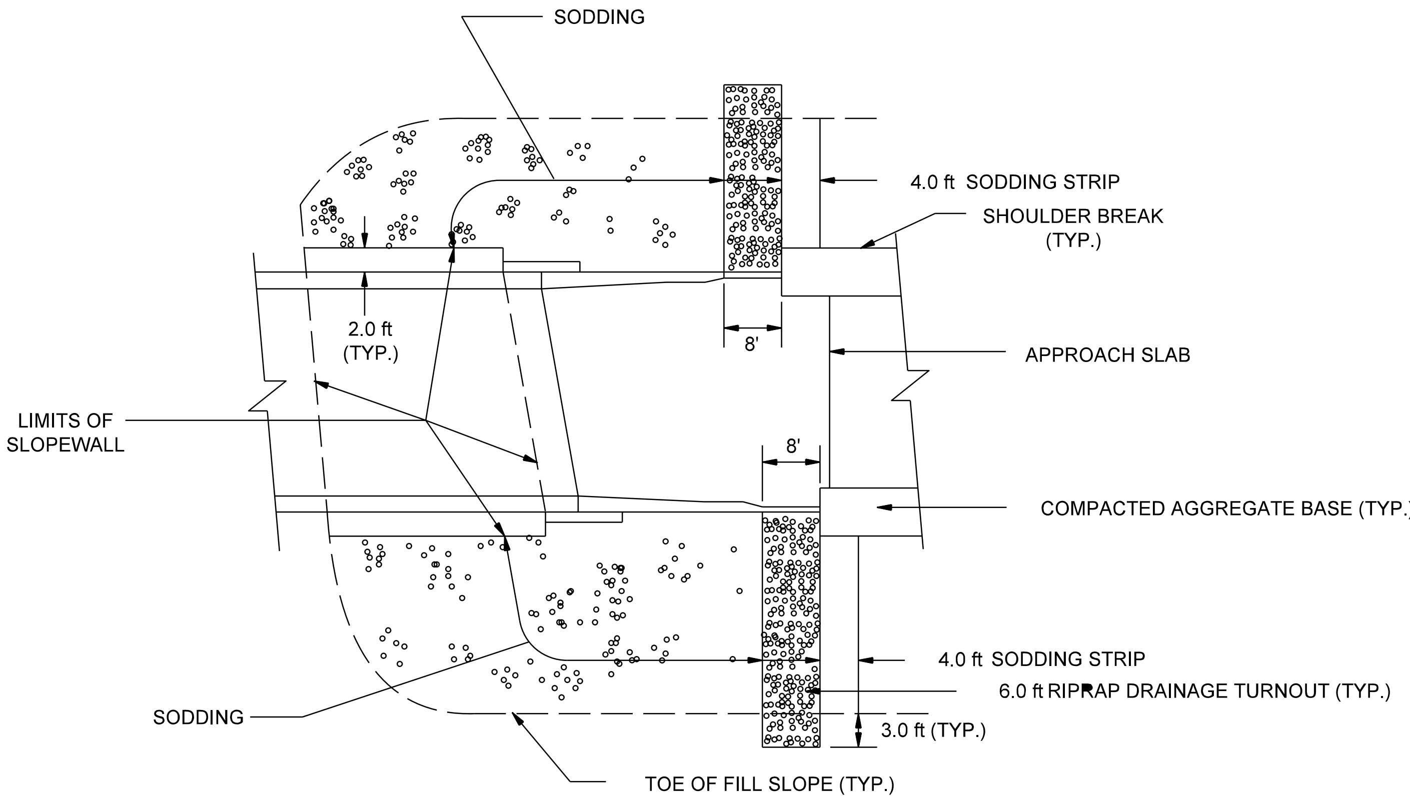
8. Step 7. Determine spacing to next and subsequent inlets. Select the values as follows:

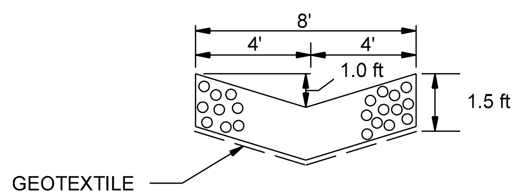
L_2 = 250 ft
X = 1500 ft
S = 0.0125
Q = 2.58 ft^3/s
E = 0.37
K_1 = 0.473
L_2 = 280 ft, therefore OK

Inlet Spacing From High Point			**Slope, *S***	**Gutter Flow, *Q* (ft³/s)**
L_i	**Long Side**	**Short Side**		
L_O	333 ft	333 ft	0.0050	1.68
L_1	250 ft	250 ft	0.00875	2.23
L_2	250 ft	N/A	0.0125	2.58
L_3	250 ft	N/A	0.0163	2.93

9. Step 8. Adapt spacing to accommodate structural constraints.

* * * * * * * *



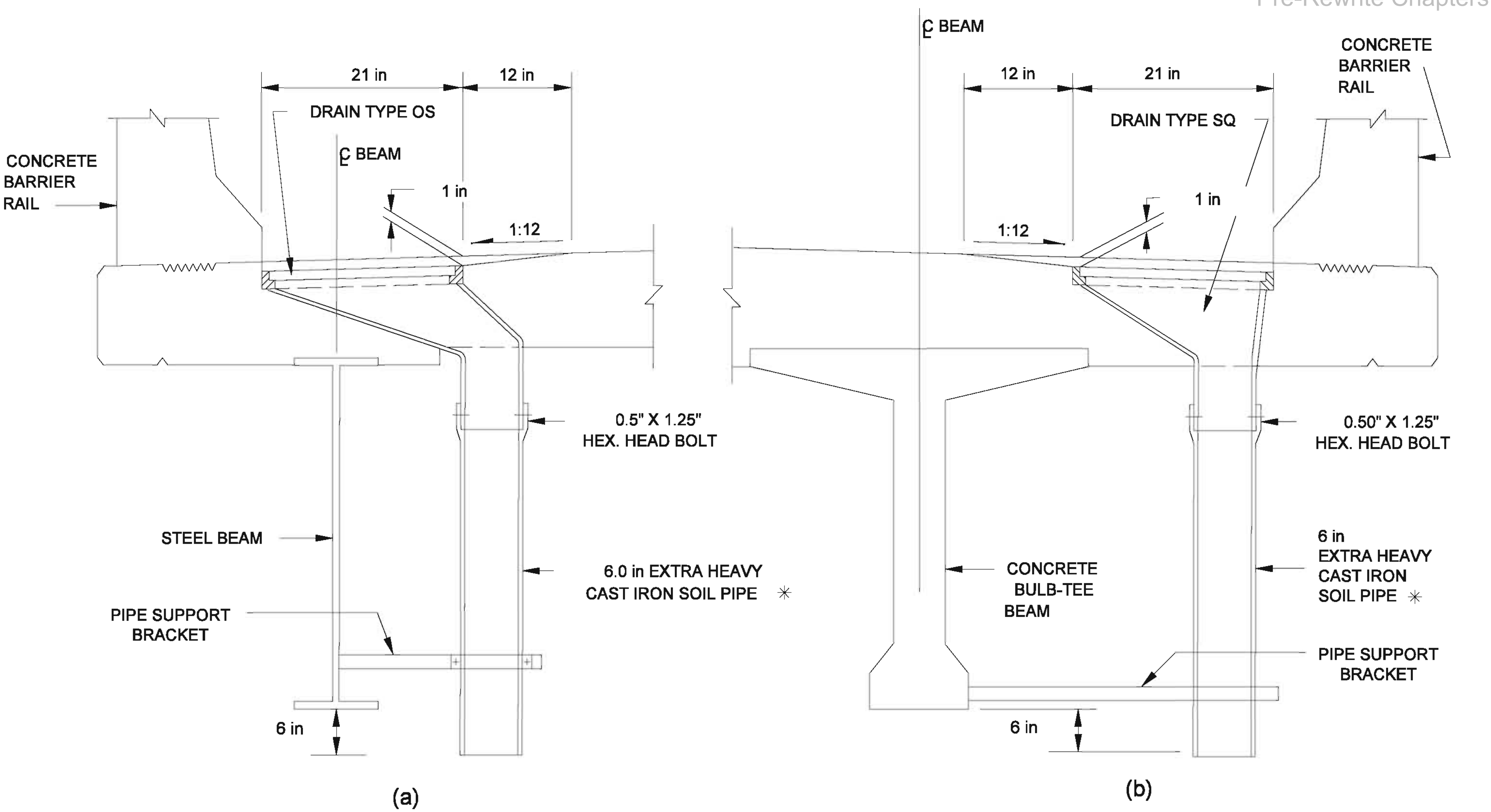


SECTION THROUGH
RIPRAP DRAINAGE TURNOUT

Note: This Figure shows the limits for where the barrier transitions are on the approach slab. See Figure 17-4I for details for where the barrier transitions are on the bridge.

SLOPE WALL, RIPRAP, AND SODDING LIMITS FOR GRADE-SEPARATION STRUCTURE

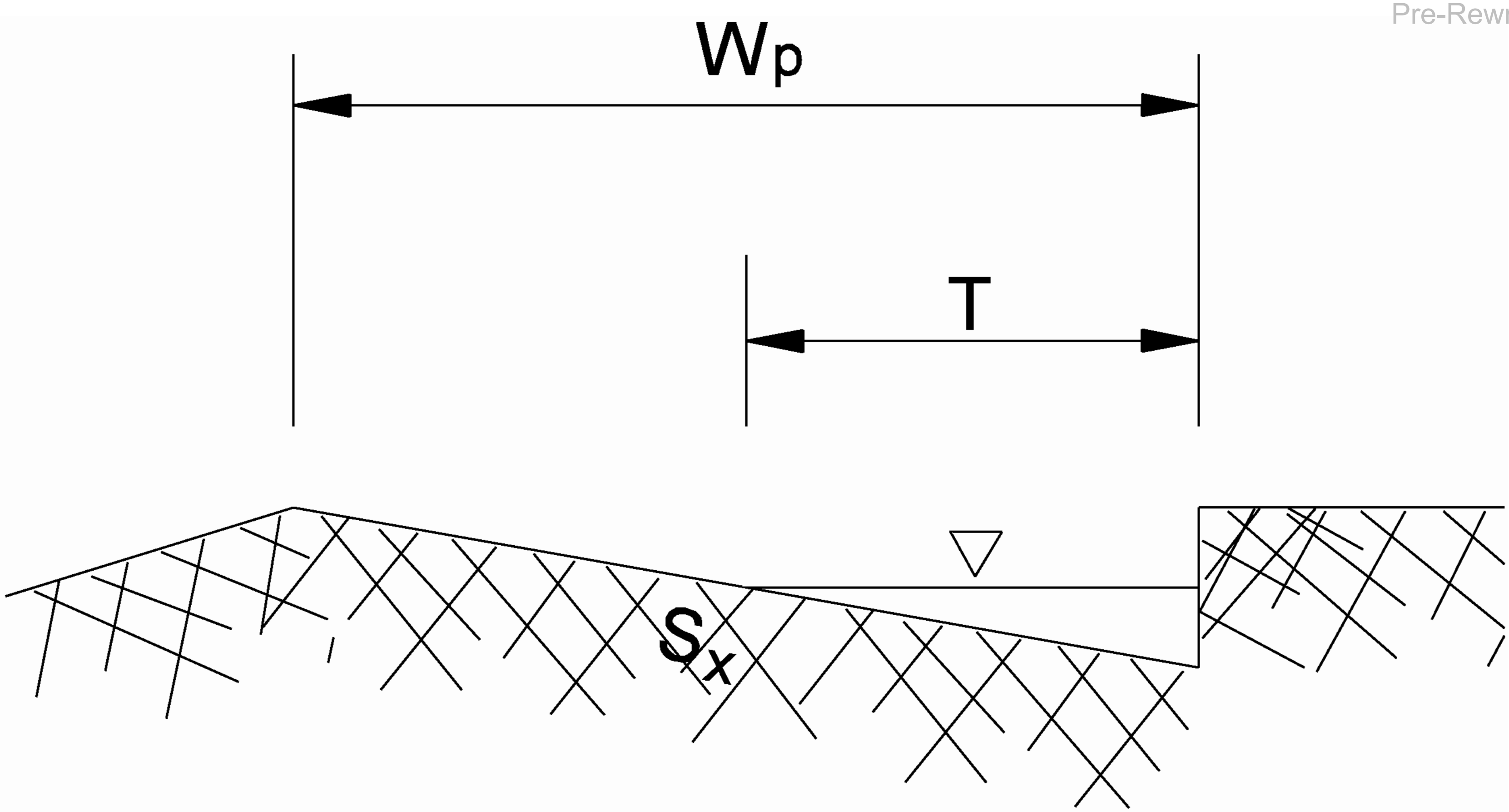
Figure 33-2A



✳ OR 6 in PVC (DESIGNER MUST INCLUDE 704-BDCG-05 TO SHOW ATTACHMENT TO ROADWAY DRAIN).

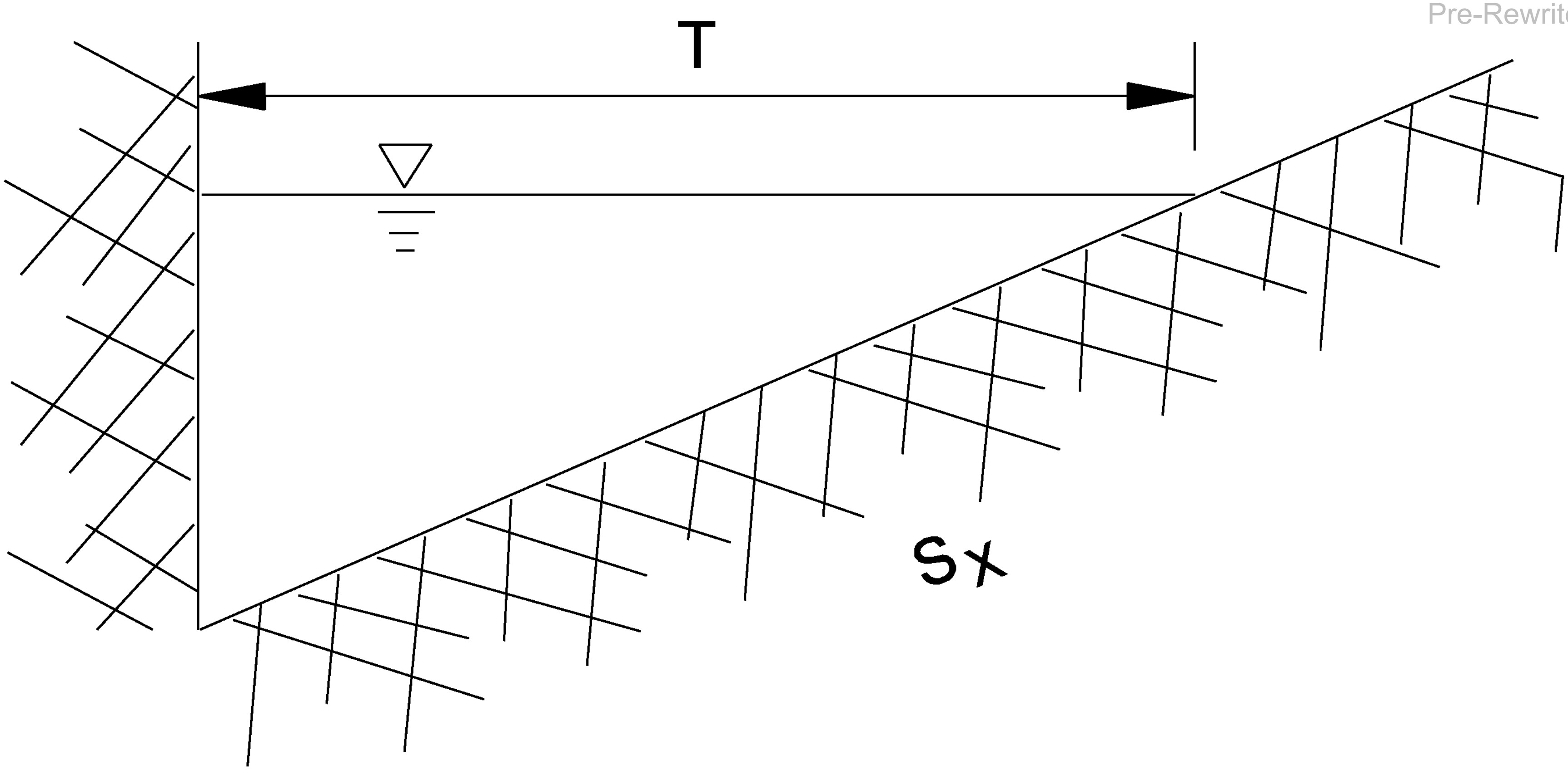
TYPICAL FLOOR DRAIN SECTIONS

Figure 33-2B



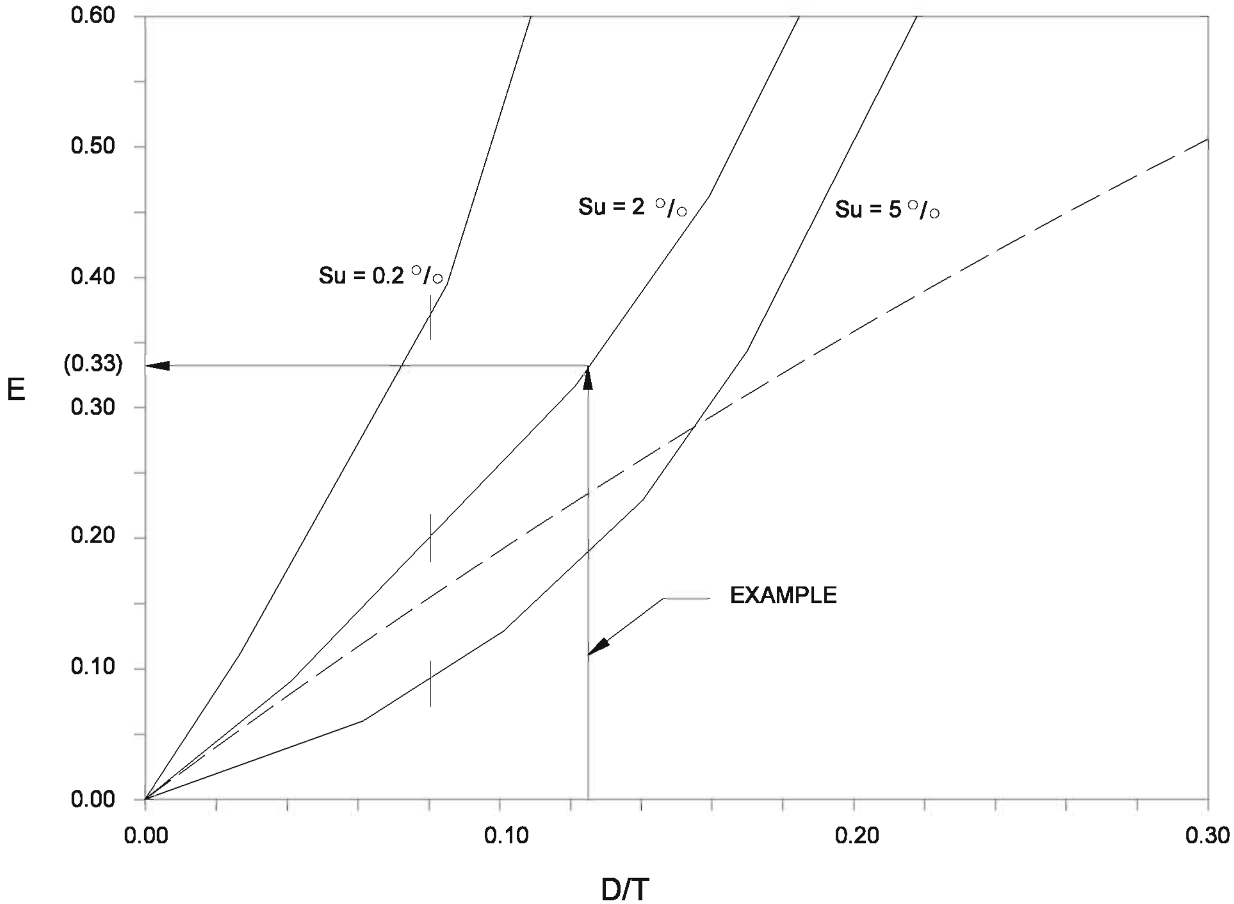
GUTTER CROSS SECTION
(Equation 33-3.3)

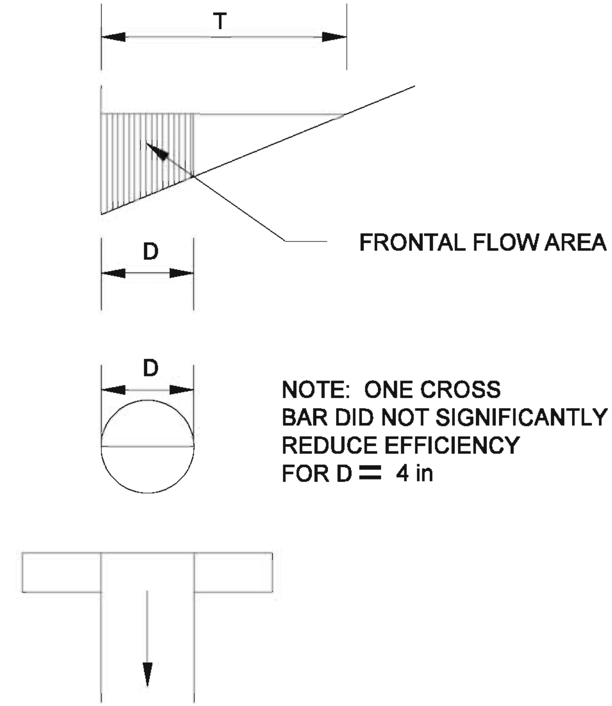
Figure 33-3A



GUTTER CROSS SECTION
(equation 33-4.1)

Figure 33-4A





EXAMPLE

GIVEN:

D =6 in; T = 4.0 ft
Su = 2 °/o

THEN:

D/T = 0.125
E = 0.33

Key:

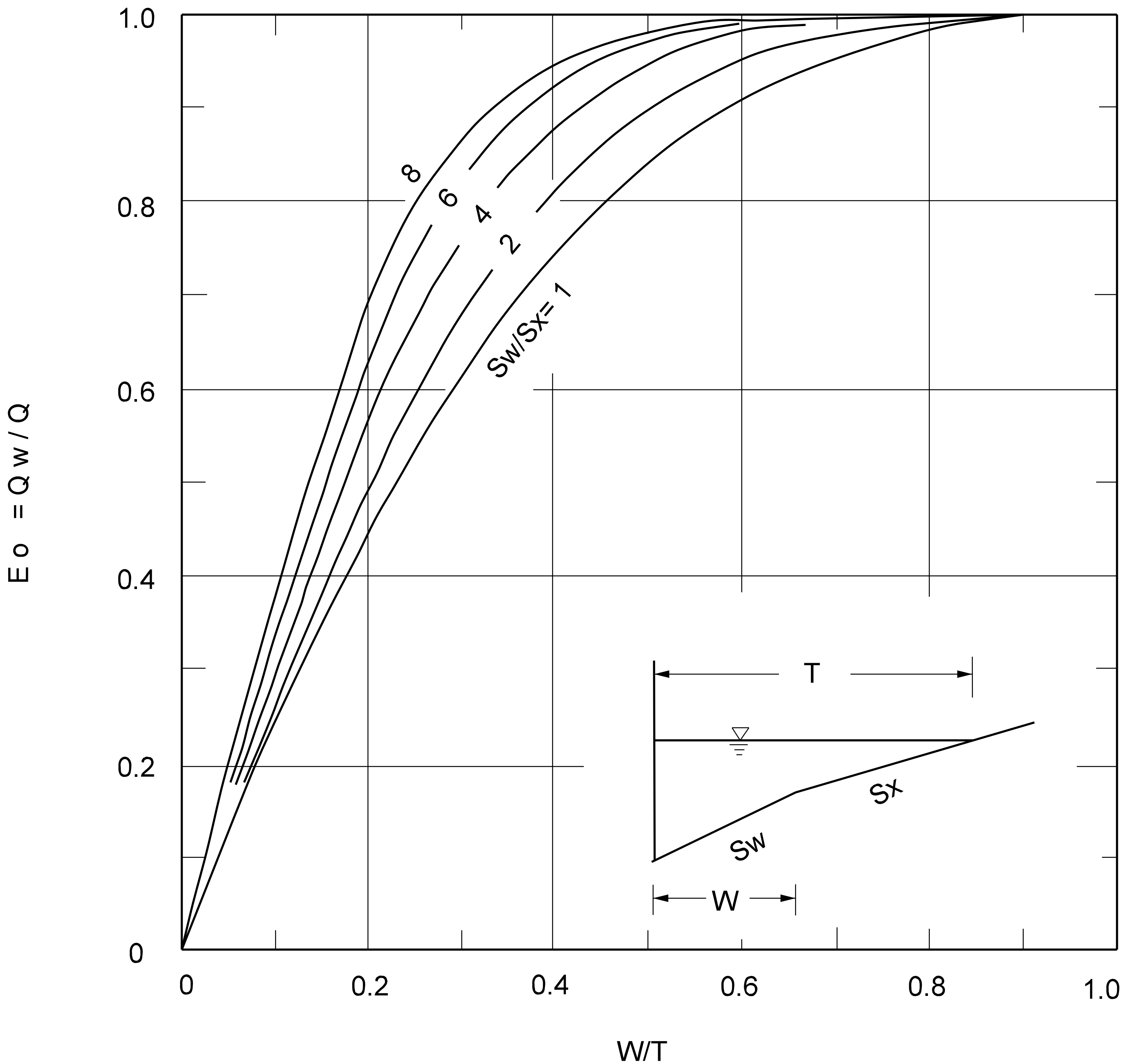
------ *Empiric curves based on laboratory data by Anderson, 1973*

- - - *100% of frontal flow of width, D*

su *Slope at next to last upstream inlet*

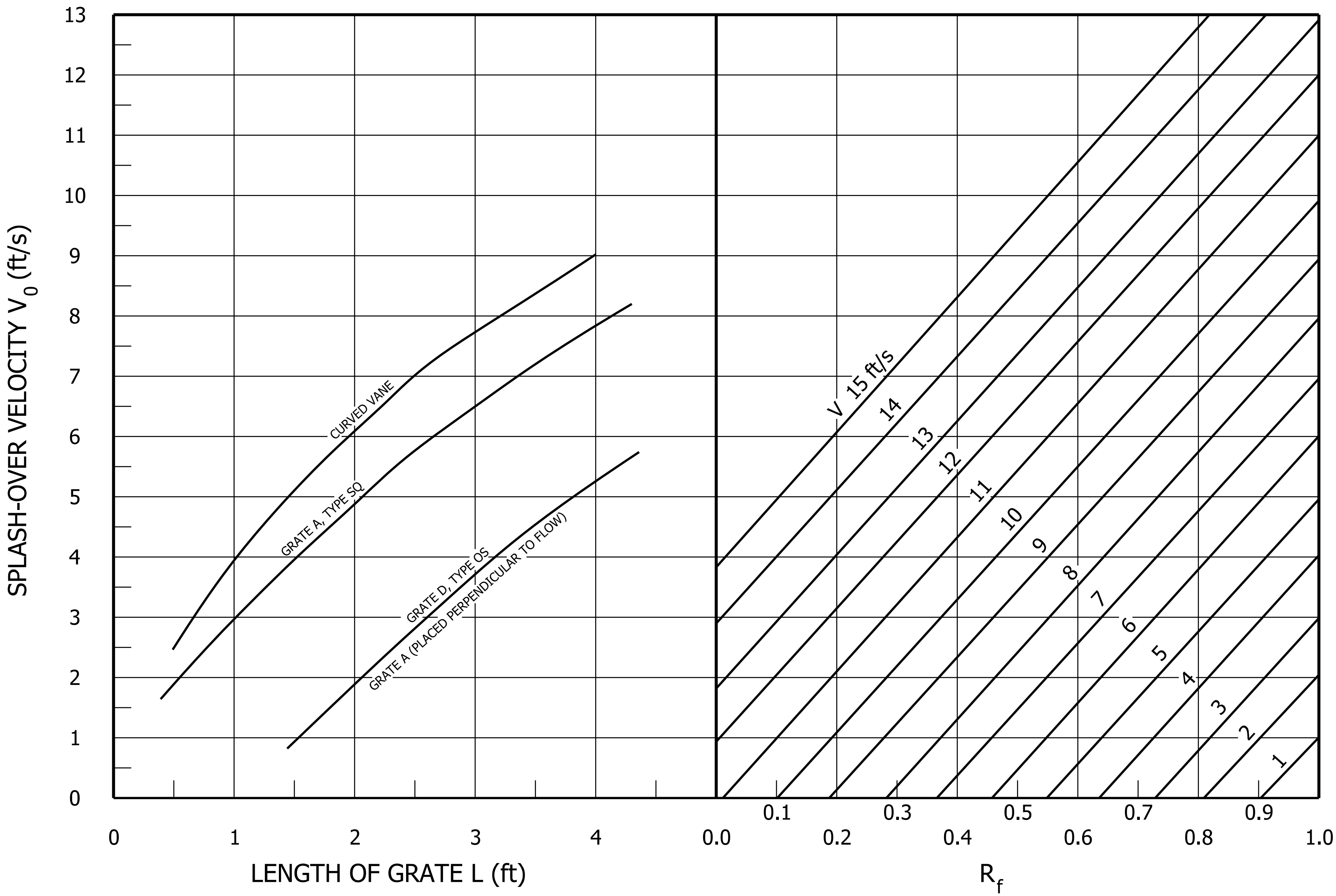
EFFICIENCY CURVES FOR CIRCULAR DRAINS

Figure 33-5A



RATIO OF FRONTAL FLOW TO TOTAL GUTTER FLOW
(Rectangular Inlets)

Figure 33-5B



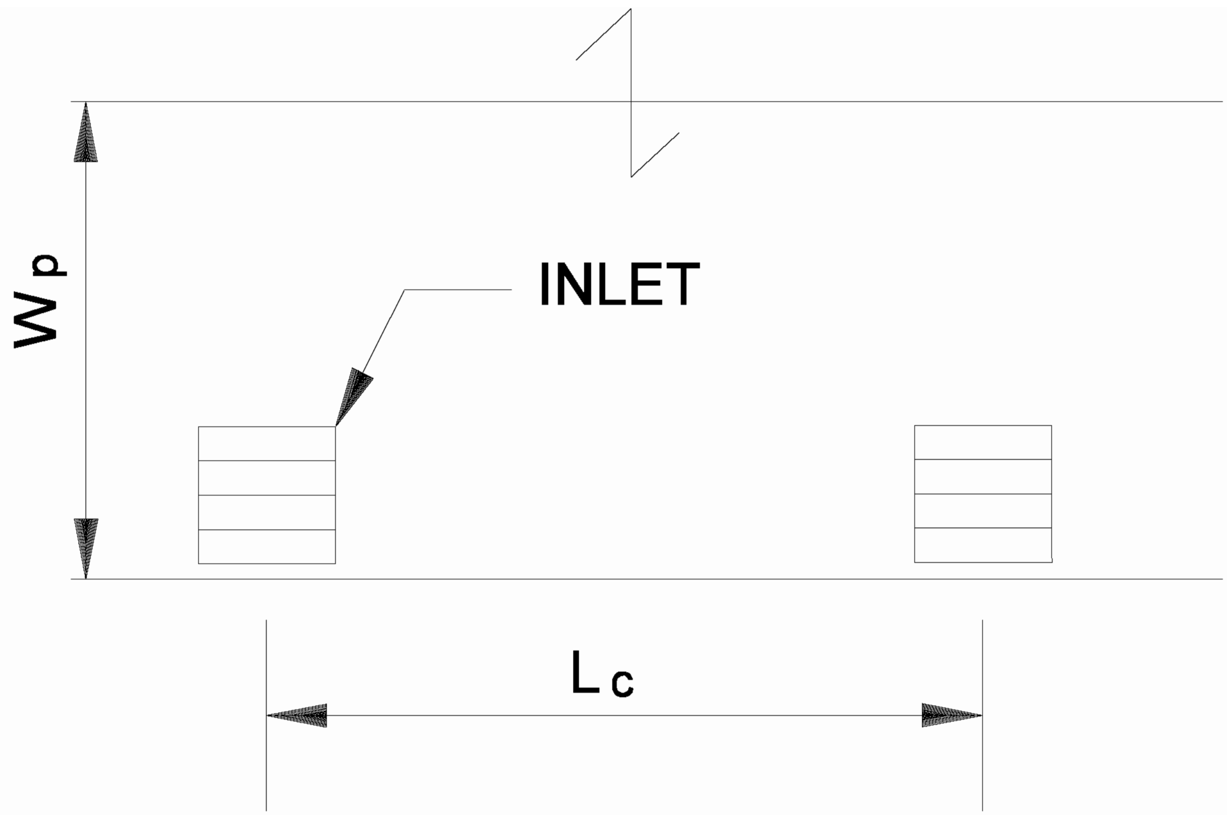
GRATE INLET FRONTAL FLOW INTERCEPTION EFFICIENCY

Figure 33-5C

Duration (min)	i_{10} (in./h)
5	6.4
10	5.44
15	4.56
30	3.28
60	2.12
120	1.36

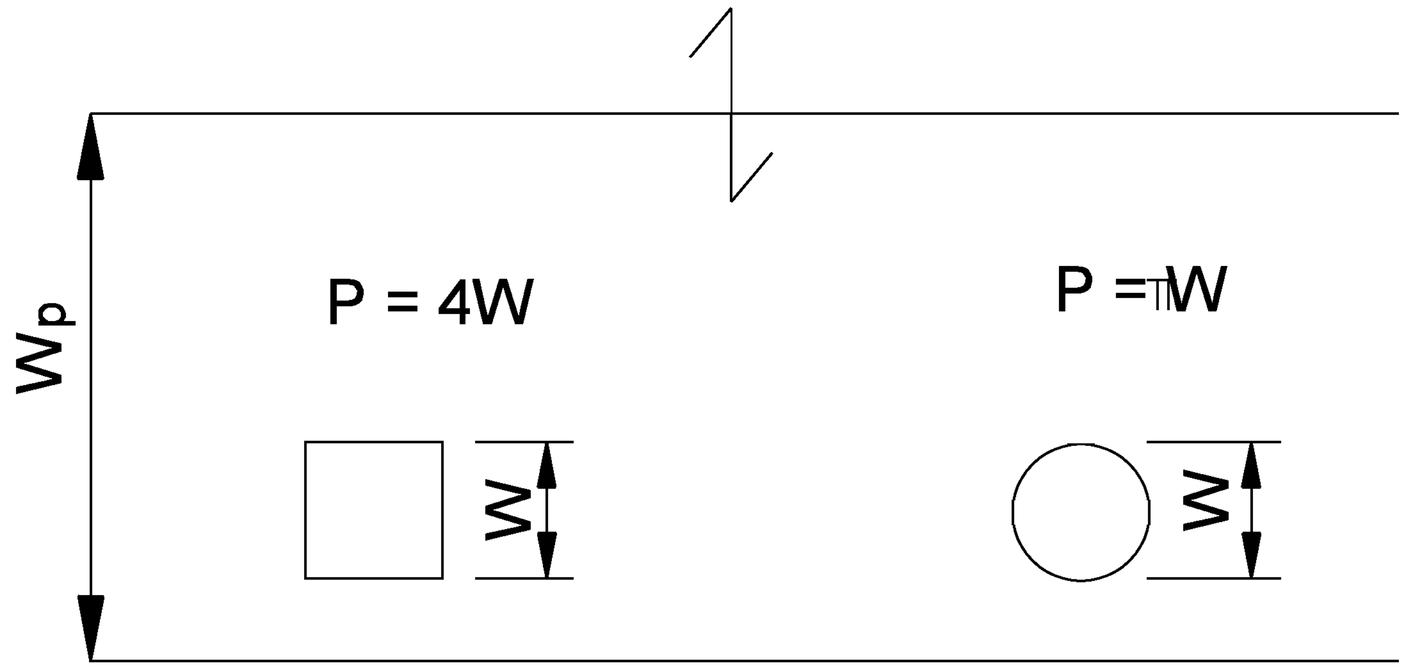
IDF DATA FROM HYDRO
(40 deg Latitude and 86 deg Longitude)

Figure 33-5D



PARAMETERS FOR EQUATION 33-5.6

Figure 33-5E



PARAMETERS FOR EQUATION 33-5.7

Figure 33-5F

TABLE OF CONTENTS

LIST OF FIGURES

CHAPTER THIRTY-FOUR

ENERGY DISSIPATORS

34-1.0 INTRODUCTION

34-1.01 Overview

The failure or damage of a culvert or detention-basin outlet structure can be traced to unchecked erosion. Erosive forces which are at work in the natural drainage network are often exacerbated by the construction of a highway or by other urban development. Interception and concentration of overland flow or constriction of a natural waterway inevitably results in an increased erosion potential. To protect the culvert and adjacent areas, it can be necessary to use an energy dissipator.

34-1.02 Definition

An energy dissipater is a device designed to protect downstream areas from erosion by reducing the velocity of flow to acceptable limits.

34-1.03 Purpose

This Chapter provides the information as follows:

1. design procedures which 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; and

2. results of analysis using the HYDRAIN system and the HY8 software.

34-1.04 Symbols

See Figure 34-1A, Symbols, Definitions, and Units.

34-2.0 DESIGN CRITERIA

34-2.01 Overview

34-2.01(01) Policy

Policy is a set of goals that establish a definite course of action or method of action and that are selected to guide and determine present and future decisions (see Section 28-4.0). Policy is implemented through design criteria for making decisions.

34-2.01(02) Design Criteria

Design criteria are the standards by which a policy is carried out or placed into action. They form the basis for the selection of the final design configuration. Listed below by categories are the design criteria which should be considered for an energy-dissipator design.

34-2.02 Dissipator-Type Selection

The dissipator type selected for a site must be appropriate to the location. The terms internal and external are used to indicate the location of the dissipator in relation to the culvert. An external dissipator is located outside of the culvert. An internal dissipator is located within the culvert barrel. The following applies to type selection.

1. Internal Dissipator. An internal dissipator is used as follows:

    a. the scour hole at the culvert outlet is unacceptable;
    b. the right of way is limited;
    c. debris is not a problem; and
    d. moderate velocity reduction is needed.

2. Natural Scour Hole. A natural scour holes is used as follows:

    a. undermining of the culvert outlet will not occur or it is practicable to be checked by a cutoff wall;

    b. the expected scour hole will not cause costly property damage; and

    c. there is no nuisance effect.

    3. External Dissipator. An external dissipator is used where the outlet scour hole is not acceptable, and a moderate amount of debris is present.

4. Stilling Basin. A stilling basin is used where the outlet scour hole is not acceptable, and debris is present.

34-2.03 Design Limitations

The following applies.

1. Ice Buildup. If ice buildup is a factor, it should be mitigated by sizing the structure to not obstruct the winter low flow, and by using an external dissipator.

2. Flood Frequency. The flood frequency used in the design of the energy dissipator device should be the same flood frequency used for the culvert design.

3. Maximum Culvert-Exit Velocity. The culvert-exit velocity should be less than 8 ft/s, or should be mitigated by using channel stabilization and energy dissipation.

4. Tailwater Relationship. The downstream hydraulic conditions should be evaluated to determine a tailwater depth and the maximum velocity for an open channel. A lake, pond, or large water body should be evaluated using the high-water elevation that has the same frequency as the design flood for the culvert.

34-2.04 Design Options

34-2.04(01) Material Selection

The material selected for the dissipator should be based on a comparison of the total cost over the design life of alternate materials and should not be made using first cost as the only criterion. This comparison should consider replacement cost, the difficulty of construction, and traffic delay.

34-2.04(02) Culvert Outlet Type

In choosing a dissipator, the selected culvert end treatment has the following implications.

1. A culvert end which is projecting or mitered to the fill slope offers no outlet protection.

2. Headwalls provide embankment stability and erosion protection. They provide protection from buoyancy and reduce damage to the culvert.

3. Commercial end sections add little cost to the culvert and may require less maintenance, retard embankment erosion, and incur less damage from maintenance.

4. Aprons do not reduce outlet velocity but, if used, should extend at least one culvert height downstream. They shall not protrude above the normal streambed elevation.

5. Wingwalls are used where the sideslopes of the channel are unstable, where the culvert is skewed to the normal channel flow, to redirect outlet velocity, or to retain fill.

34-2.04(03) Safety Considerations

Traffic should be protected from an external energy dissipator by locating it outside the appropriate clear-zone distance as described in Chapter Forty-nine.

<u>34-2.05 Related Designs</u>

34-2.05(01) Culvert

The culvert should be designed independent of the dissipator design (see Chapter Thirty-one), with the exception of an internal dissipator, which may require an iterative solution. The culvert design should be completed before the outlet protection is designed and should include computation of outlet velocity.

34-2.05(02) Downstream Channel

The downstream channel protection should be designed concurrently with the dissipator design (see Chapter Thirty).

34-3.0 DESIGN PHILOSOPHY

<u>34-3.01 Alternative Analysis</u>

Choose alternatives which satisfy the topography, and design policies and criteria.

Analyze alternatives for environmental impact, hydraulic efficiency, and risk and cost.

The selected dissipator should satisfy the selected structural and hydraulic criteria, and should be based on the following:

1. construction and maintenance costs;
2. risk of failure or property damage;
3. traffic safety;
4. environmental or aesthetic considerations;
5. political or nuisance considerations; and
6. land-use requirements.

34-3.02 Design Methods

The designer must choose as follows:

1. to design for local scour or channel degradation;

2. to mitigate or monitor erosion problems;

3. to use a drop structure, internal dissipator, scour hole, external dissipater, or stilling basin; and

4. to use charts or computer software.

34-3.02(01) Types of Scour

The following apply.

1. Local Scour. Local scour is the result of high-velocity flow at the culvert outlet and extends only a limited distance downstream.

2. 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 each runoff event.

34-3.02(02) Scour-Hazard Protection

The following apply.

1. Mitigated. The scour-hazard protection device should be designed by providing protection at the culvert outlet as follows:

a. Initial protection should be sufficient to provide assurance that extensive damage cannot result from one design runoff event.

b. The protection device should be inspected after a major storm to determine if protection must be increased or extended.

2. Monitored. The site should be inspected after a major-storm event to determine if protection is needed.

34-3.02(03) Dissipator Types

The following types are available.

1. Scour Hole. The design of a scour hole is described in Section 34-4.0.

2. Internal Dissipator. This includes the tumbling-flow type and the increased-resistance type.

   This Chapter does not address the design of an internal dissipator. The designer should refer to FHWA HEC 14 *Hydraulic Design of Energy Dissipators for Culverts and Channels*, September 1985, revised in 1995, and FHWA/OH-84/007 *Internal Energy Dissipators*, if design procedure is needed.

3. External Dissipator. This includes the following:

   a. USBR Type VI Impact (see Section 34-9.0);
   b. riprap (see Section 34-8.0);
   c. CSU rigid boundary (see HEC 14);
   d. Contra Costa (see HEC 14);
   e. hook (see HEC 14); and
   f. hydraulic jump (see HEC 14).

4. Stilling Basin. This includes the following:

   a. Saint Anthony Falls (SAF) (see Section 34-7.0);
   b. USBR Type II (see HEC 14);
   c. USBR Type III (see HEC 14); and
   d. USBR Type IV (see HEC 14).

5. Drop Structure. See HEC 14.

34-3.02(04) Computational Methods

The following are available.

1. Charts. Charts are required for a manual solution. Charts required for the design of a scour hole, riprap basin, USBR Type VI impact basin, or SAF basin are included herein. Charts required for the design of another type of energy dissipator are shown in HEC 14.

2. Computer Software. HY-8 (FHWA Culvert Analysis Software) Version 4.1 or later, includes an energy-dissipator module which can be used to analyze most types of energy dissipators described in HEC 14.

34-4.0 DESIGN EQUATIONS

34-4.01 General

An exact theoretical analysis of flow at a culvert outlet is complex because the required information is as follows:

1. analyzing non-uniform and rapidly-varying flow;
2. applying energy and momentum balance;
3. determining where a hydraulic jump will occur;
4. applying the results of hydraulic model studies; and
5. consideration of temporary upstream storage effects.

34-4.02 Approach

The design procedure provided herein is based on the following.

1. Model studies were used to calibrate the equations and charts for scour-hole estimating and energy-dissipator design.

2. HEC 14 (revised version, 1995) is the base reference and includes a full explanation of all equations and procedures used herein, with the exception of those discussed in Section 34-4.03.

34-4.03 Culvert-Outlet Conditions

The culvert design establishes the outlet-flow conditions. However, these parameters may require closer analysis for energy-dissipator design.

1. Depth, d_o (ft) The normal depth assumption should be reviewed, and a water-surface profile calculated if $L < 50d_o$. The brink depth (see HEC 14 for curves) should be used for a mild slope and low tailwater, not critical depth.

2. Area, A_o (ft^2) The cross-sectional area of flow at the culvert outlet should be calculated using d_o.

3. Velocity, V_o (ft/s) The culvert outlet velocity should be calculated as follows:

$$V_o = \frac{Q}{A_o} \qquad \text{(Equation 34-4.1)}$$

Where Q = discharge, ft^3/s

4. Froude Number, Fr. The Froude number is a flow parameter that has been used to design an energy dissipater, and is calculated using the formula as follows:

$$Fr = \frac{V_o}{(gd_o)^{0.5}} \qquad \text{(Equation 34-4.2)}$$

Where g = acceleration due to gravity, 32.2 ft/s^2

5. Equivalent Depth, $d_E = (A_o/2)^{0.5}$ (ft) Equivalent depth is an artificial depth which is calculated for a culvert which is not rectangular, so that a reasonable Fr can be determined.

6. Discharge Intensity, DI_c. Discharge intensity is a flow parameter similar to Fr which is used for a circular culvert of diameter, D, which is flowing full, as follows:

$$DI_c = \frac{Q}{g^{0.5} D^{2.5}} \qquad \text{(Equation 34-4.3)}$$

7. Discharge Intensity Modified, DI. Modified discharge intensity, as described in HEC 14, Chapter V, (revised version, 1995), is as follows:

$$DI = \frac{Q}{g^{0.5} R_c^{2.5}} \qquad \text{(Equation 34-4.4)}$$

Where: Q = discharge, ft^3/s
A_c = culvert area, ft^2
P_c = culvert perimeter, ft
R_c = A_c/P_c

34-4.04 Scour-Hole Estimation

HEC 14, Chapter V, (revised version, 1995) includes an estimating procedure for scour-hole geometry based on soil, flow data, and culvert geometry. This scour-prediction procedure is intended to serve together with the maintenance history and site reconnaissance information for determining energy-dissipator needs.

Only a scour hole on cohesionless material will be discussed herein. For a scour hole on cohesive soil, see HEC 14, Chapter V for details.

The results of tests by the U.S. Army Waterways Experiment Station, Vicksburg, Mississippi, indicate that scour-hole geometry varies with the tailwater conditions. The maximum scour-hole geometry occurs at a tailwater depth of 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.

The following empirical equations defining the relationship between the culvert discharge intensity and time, and the length, width, depth, and volume of the scour hole, are provided for the maximum or extreme scour situation.

$$\left[\frac{d_s}{R_c},\frac{W_s}{R_c},\frac{L_s}{R_c}\right]=\left(\frac{C_sC_h\alpha}{\sigma^{0.33}}\right)\left(\frac{Q}{g^{0.5}R_c^{\ 2.5}}\right)^{\beta}\left(\frac{t}{316}\right)^{\theta} \qquad \text{(Equation 34-4.5)}$$

Where: d_S = maximum depth of scour hole, ft
L_S = length scour hole, ft
W_S = width of scour hole, ft

$$d_S,\ W_S,\ \text{or}\ L_S = R_cF_1F_2F_3 \qquad \text{(Equation 34-4.6)}$$

Where: $F_1=\dfrac{C_sC_h\alpha}{\sigma^{0.33}}$ and $F_2=\left(\dfrac{Q}{g^{0.5}R_c^{\ 2.5}}\right)^{\beta}$

t = 30 min or the time of concentration, if longer
R_c = hydraulic radius of drainage structure flowing full
σ = material standard deviation (σ = 2.10 for gravel, or 1.87 for sand)

ά, ß, θ, C_s, and C_h are coefficients as shown in Figure 34-4A, Coefficients for Scour-Hole Estimation

F_1, F_2 and F_3 are factors to aid the computation, as shown in Step 7B, Figure 34-5A, Energy Dissipator Checklist. An editable version of this form may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/.

34-5.0 DESIGN PROCEDURE

The following design procedure is intended to provide a convenient and organized method for manually designing an energy dissipator. The designer should be familiar with all of the equations in Section 34-4.0 before using the procedure. Application of the following design method without an understanding of hydraulics can result in an inadequate, unsafe, or costly structure.

1. Step 1. Assemble Site Data And Project File.

    a. See culvert design file for site survey.
    b. Review Section 34-2.0 for applicable criteria.

2. Step 2. Determine Hydrology. See culvert design file.

3. Step 3. Select Design *Q*.

    a. See Section 34-2.03.
    b. See culvert design file.
    c. Select flood frequency.
    d. Determine *Q* from frequency plot (Step 2).

4. Step 4. Design Downstream Channel.

    a. See culvert design file.
    b. Determine channel slope, cross section, normal depth, and velocity.
    c. Check bed- and bank-materials stability.

5. Step 5. Design Culvert. See culvert design file and obtain design discharge, outlet flow conditions (velocity and depth), culvert type (size, shape, and roughness), culvert slope, and performance curve.

6. Step 6. Summarize Data On Design Form.

    a. Use Figure 34-5A, Energy Dissipator Checklist.

b. Enter data from Steps 1-5 into Figure 34-5A.

7. Step 7. Estimate Scour-Hole Size.

a. Enter input for the scour equation shown in Figure 34-5A.
b. Calculate d_S, W_S, L_S using Equation 34-4.5 or 34-4.6.

8. Step 8. Determine Need for Dissipator. An energy dissipator is needed if one or more conditions apply as follows:

a. the estimated scour-hole dimensions, which 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 significantly greater than V_d.

9. Step 9. Select Design Alternative. See Section 34-2.04.

10. Step 10. Design Dissipator. Use the following design procedure and charts.

a. Section 34-7.0 for SAF.
b. Section 34-8.0 for riprap.
c. Section 34-9.0 for USBR Type VI.

11. Step 11. Design Riprap Transition. A dissipator will likely require some protection adjacent to the basin exit. The length of protection can be judged based on the difference between V_o and V_d. The riprap should be designed using HEC 11.

12. Step 12. Review Results.

a. If downstream channel conditions (velocity, depth, and stability) are exceeded, either design riprap for the channel (Step 4), or select another dissipator (Step 9).

b. If the preferred energy dissipator type affects the culvert hydraulics, return to Step 5 and calculate culvert performance.

c. If a debris-control structure is required upstream, see HEC 9.

d. If a check Q was used for the culvert design, assess the dissipator performance with this discharge.

13. Step 13. Documentation.

    a. See Chapter Twenty-eight.
    b. Include computations in the culvert report or file.

34-6.0 DESIGN EXAMPLE

34-6.01 Design Example Steps

The following example problem uses the culvert data provided in Chapter Thirty-one.

1. Step 1. Assemble Site Data and Project File.

    a. Site survey. The culvert project file includes USGS, site, and location maps, roadway profile, and embankment cross sections. Site-visit notes indicate no sediment or debris problems and no nearby structures. See Figure 34-6.1 for site data.

    b. Studies by Other Agencies. None.

    c. Design Criteria.

        (1) 50-year frequency for design.
        (2) 100-year frequency for check.

2. Step 2. Determine Hydrology. USGS regression equations yield the following:

    a. $Q_{50} = 400$ ft^3/s
    b. $Q_{100} = 500$ ft^3/s

3. Step 3. Select Design Q. Use $Q_{50} = 400$ ft^3/s, as requested by the design criteria.

4. Step 4. Design Downstream Channel.

    a. See Figure 34-6.2 for cross section of channel with slope = 0.05.

Point	Station, ft	Elevation, ft
1	12.3	182.9
2	22.3	177.8
3	32.7	177.3
4	34.7	175.3

5	39.7	175.3
6	41.7	177.3
7	51.7	177.8
8	62.0	182.9

b. Rating curve for channel. Calculating normal depth yields the following:

Q (ft^3/s)	TW (ft)	V (ft/s)
100.0	1.43	11.33
200.0	2.10	14.00
300.0	2.57	16.20
400.0	2.87	17.83
500.0	3.13	19.13

c. At V_{50} = 17.83 ft/s, the 3-in. gravel material which makes up the channel boundary is not stable and riprap is needed for a transition.

5. Step 5. Design Culvert. A reinforced-concrete box culvert, 7 ft by 6 ft, with a beveled entrance on a slope of 0.05 was the selected design. The FHWA HY8 program showed that this culvert is operating at inlet control and has the following properties.

Q (ft^3/s)	HW_i (ft)	V_o (ft/s)
Q_{50} = 400.0	7.73	28.70
Q_{ot} = 460.0	8.63	29.60
Q_{100}= 500.0	8.73	29.67

6. Step 6. Summarize Data On Design Form. See Figure 34-5A.

7. Step 7. Size Scour Hole. The size of the scour hole is determined using Equations 34-4.5 and 34-4.6. For a channel with gravel bed, the standard deviation of the material, σ, is 2.10. Figure 34-4A shows that the value of C_s =1.00 and C_h = 1.08. See Figure 34-6A, Energy Dissipator Checklist (Example), for a summary of the computation.

8. Step 8. Determine Need For Dissipator. The scour-hole dimensions are excessive, and, since V_o = 28.70 ft/s is much greater than V_d = 17.83 ft/s, an energy dissipator is needed.

9. Step 9. Select Design Alternative. See Section 34-2.04.

10. Step 10. Design Dissipator. The design of an SAF stilling basin is as shown in Section 34-7.0.

11. Step 11. Design Riprap Transition. Protection is required (see HEC 11).

12. Step 12. Review Results. The downstream-channel conditions are matched by the dissipator.

13. Step 13. Documentation.

    a. See Chapter Twenty-eight.
    b. Include computations in the culvert report or file.

34-6.02 Computer Output

The scour-hole geometry can also be computed by using the FHWA microcomputer program HY-8, Culvert Analysis, Version 4.0 or later, Energy Dissipators module. A hardcopy of the module output is as shown as Figure 34-6B. The dimensions of the scour hole computed by the HY-8 program are reasonably close to the values calculated above.

34-7.0 STILLING BASIN

34-7.01 Overview

The St. Anthony Falls (SAF) stilling basin uses a forced hydraulic jump to dissipate energy and has the properties as follows:

1. is based on model studies conducted by the Natural Resources Conservation Service (NRCS) at the St. Anthony Falls (SAF) Hydraulic Laboratory of the University of Minnesota;

2. uses chute blocks, baffle blocks, and an end sill to force the hydraulic jump and reduce jump length by about 80%; and

3. is recommended where Fr ranges from 1.7 to 17.0.

34-7.02 Equations

1. Basin Width, W_B.

    a. for a box culvert, $W_B = B$ = culvert width (ft)
    b. for a pipe, W_B = culvert diameter, D, (ft) or

$$W_B = 1.769D \frac{Q}{g^{0.5} D^{2.5}} \qquad \text{(Equation 34-7.1)}$$

whichever is larger.

Where Q = discharge, ft^3/s

2. Flare (z:1). Flare is optional. If used, it should be flatter than 2:1.

3. Basin Length, L_B.

$$d_j = 0.5 d_1 \left[\left(1 + 8 Fr_1^2\right)^{0.5} - 1 \right] \qquad \text{(Equation 34-7.2)}$$

Where: d_1 = initial depth of water, ft
d_j = sequent depth of jump, ft
Fr_1 = Froude number entering basin, $\neq Fr$

$$L_B = \frac{4.5 d_j}{Fr_1^{0.76}} \qquad \text{(Equation 34-7.3)}$$

4. Basin Floor. The basin floor should be depressed below the streambed enough to obtain the depth, d_2, below the tailwater as follows:

a. For $1.7 \leq Fr_1 < 5.5$,

$$d_2 = d_j \left[1.1 - \left(\frac{Fr_1^2}{120} \right) \right] \qquad \text{(Equation 34-7.4)}$$

b. For $5.5 \leq Fr_1 < 11.0$,

$$d_2 = 0.85 d_j \qquad \text{(Equation 34-7.5)}$$

c. For $11.0 \leq Fr_1 < 17.0$,

$$d_2 = d_j \left[1.1 - \left(\frac{Fr_1^2}{800} \right) \right] \qquad \text{(Equation 34-7.6)}$$

5. 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 half-block at each wall

6. 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 ensure that 40% to 55% of W_{B2} is occupied by blocks
Staggered with chute blocks
Space at wall $\geq 0.38d_1$
Distance from chute blocks, $L_{1\text{-}3} = L_B/3$

7. End Sill Height. $h_4 = 0.07d_j$

8. Sidewall Height. $d_2 + 0.33d_j$

9. Wingwall Flare. 45 deg

34-7.03 Design Procedure

The design of a St. Anthony Falls (SAF) basin consists of the steps as follows.

1. Step 1. Select Basin Type.

   a. Rectangular or flared.
   b. Choose flare (if needed), z:1.
   c. Determine basin width, W_B.

2. Step 2. Select Depression.

   a. Choose depth d_2 to depress below the streambed, B_d.
   b. Assume $B_d = 0$ for first trial.

3. Step 3. Determine Input Flow.

a. d_1 and V_1, using energy equation.
b. Froude Number, Fr_1.

4. Step 4. Calculate Basin Dimensions.

a. d_j (Equation 34-7.2).
b. L_B (Equation 34-7.3).
c. d_2 (Equation 34-7.4, 34-7.5. or 34-7.6).
d. $L_S = (d_2 - TW)/S_S$
e. $L_T = B_d/S_T$ (see Figure 34-7A, St. Anthony Falls Basin Checklist). An editable version of this form may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/.
f. $L = L_T + L_B + L_S$ (see Figure 34-7A).

5. Step 5. Review Results.

a. If $d_2 \neq (B_d - LS_o + TW)$, return to Step 2.
b. If approximately equal, continue.

6. 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, $h_5 = d_2 + 0.33d_j$.

* * * * * * * *

34-7.04 Example 34-7.1

See Section 34-6.0 for input values. See Figure 34-7C, Energy Dissipator HY-8 Program Output, for completed computation form.

1. Step 1. Select Basin Type.

a. Use a rectangular basin.
b. No flare.
c. Basin width, W_B = 7.00 ft

2. Step 2. Select Depression. Trial 1: B_d = 1.83 m, $S_S = S_T = 1$.

3. Step 3. Determine Input Flow, Trial 1.

a. Energy equation, culvert to basin:
Culvert outlet = $B_d + d_o + V_o^2/2g$ = 6.00 + 1.87 + $(28.9)^2/2(32.2)$ = 20.83 ft
Basin floor = $0 + d_1 + V_1^2/2g$
Solve: $20.83 = d_1 + V_1^2/2g$

d_1	V_1	$d_1+V_1^2/2$
1.63	36.20	21.98 > 20.83
1.67	35.47	21.20 $\neq$ 20.83, Use.

b. $Fr_1 = \left(\dfrac{35.47}{(1.67 \times 32.2)^{0.50}} \right) = 4.80$

4. Step 4. Calculate Basin Dimensions, Trial 1.

a. d_j = 10.50 ft (Equation 34-7.2)
b. L_B = 14.33 ft (Equation 34-7.3)
c. d_2 = 9.53 ft (Equation 34-7.5)
d. $L_S = (d_2 - TW)/S_S$ = (9.53 – 2.86)/1 = 6.67 ft
e. $L_T = B_d/S_T$ = 6.00/1 = 6.00 ft
f. $L = L_T + L_B + L_S$ = 6.00 + 14.33 + 6.67 = 27 ft

5. Step 5. Review Results, Trial 1.

a. If d_2 does not equal $(B_d - LS_o + TW)$, adjust drop.
9.53 $\neq$ [6.00 – 27.00(0.05) + 2.87] = 7.52 ft

b. Add 9.53 – 7.52 = 2.01 more drop and return to Step 2.

6. Repeat Step 2. Select Depression. Trial 2: B_d = 2.41 m, $S_S = S_T = 1$.

7. Repeat Step 3. Determine Input Flow. Trial 2:

a. Energy equation, culvert to basin:
Culvert outlet = $B_d + d_o + V_o^2/2g$ = 8.03 + 2.07 + $(28.7)^2/2g$ = 22.89 ft
Basin floor = $0 + d_1 + V_1^2/2g$
Solve: $6.60 = d_1 + V_1^2/2g$

d_1	V_1	$d_1 = V_1^2/2g$
1.60	36.93	22.78 $\neq$ 22.89, Use.

b. $$Fr_1 = \frac{36.93}{[1.60 \times 32.2]^{0.5}} = 5.11$$

8. Repeat Step 4. Calculate Basin Dimensions. Trial 2:

   a. d_j = 10.53 ft (Equation 34-7.2)
   b. L_B = 13.93 ft (Equation 34-7.3)
   c. d_2 = 9.40 ft (Equation 34-7.5)
   d. $L_S = (d_2 - TW)/S_S$ = 6.53 ft
   e. $L_T = B_d/S_T$ = 8.03/1 = 8.03 ft
   f. $L = L_T + L_B + L_S$ = 8.03 + 13.93 + 6.53 = 28.5 ft

9. Repeat Step 5. Review Results. Trial 2:

   $d_2 = 9.40 \neq [8.03 - 28.5(0.05) + 2.87] = 9.47$ ft.
   Is approximately equal; continue.

10. Step 6. Size Elements. Trial 2:

   a. Chute blocks, h_1, W_1, W_2, N_c
   $h_1 = d_1$ = 1.60 ft
   $W_1 = 0.75d_1$ = 1.20 ft
   $N_c = W_B/2W_1$ = 7.00/2(1.2) = 2.92; use 3
   Adjusted W_1 = 7.00/2(3) = 1.17 ft = W_2
   Use 2 full blocks, 3 spaces and a half-block at each wall.

   b. Baffle blocks, h_3, W_3, W_4, N_B, $L_{1\text{-}3}$
   $h_3 = d_1$ = 1.60 ft
   $W_3 = 0.75d_1$ = 1.20 ft
   Use 3 blocks, as above, $W_3 = W_4$ = 1.17 ft
   $L_{1\text{-}3} = L_B/3$ = 13.93/3 = 4.64 ft

   c. End sill, $h_4 = 0.07d_j$ = 0.07(10.53) = 0.74 ft

   d. Sidewall height, $h_5 = d_2 + 0.33d_j$ = 9.40 + 0.33(10.53) = 12.88 ft
   See Figure 34-7A(0) for block design.

34-7.05 Computer Output

The dissipator geometry can be computed using the HY-8 Culvert Analysis microcomputer program, Energy Dissipator module. The output of the culvert and channel input data, and computed geometry using this module are shown as Figure 34-7B, St. Anthony Falls Basin, Example.

34-8.0 RIPRAP BASIN

34-8.01 Overview

The riprap-basin design is based on laboratory data obtained from full-scale prototypical installations. The following are the principal features of the basin.

1. Preshaping and lining with riprap of median size, d_{50}.
2. Constructing the floor at a depth of h_S below the invert, where h_S is the depth of scour that will occur in a pad of riprap of size d_{50}.
3. Sizing d_{50} so that $2 < h_S/d_{50} < 4$.
4. Sizing the length of the dissipating pool to be $10h_S$ or $3W_o$, whichever is larger for a single barrel. The overall length of the basin is $15h_S$ or $4W_o$, whichever is larger.
5. Angular-rock results were approximately the same as the results for rounded material.
6. Layout details are shown on Figure 34-8A, Details of Riprap-Basin Energy Dissipator.

For high tailwater, $TW/d_o > 0.75$, the following applies.

1. The high-velocity core of water emerging from the culvert retains its jet-like character as it passes through the basin.
2. The scour hole is not as deep as with low tailwater and is longer.
3. Riprap may be required for the channel downstream of the rock-lined basin.

34-8.02 Design Procedure

An editable version of Figure 34-8C, Riprap Basin Design Checklist, may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/.

1. Step 1. Determine Input Flow. d_o or d_E, V_o, Fr at the culvert outlet, and d_E, the equivalent depth at the brink = $(A/2)^{0.5}$.

2. Step 2. Check TW. Determine if $TW/d_o \leq 0.75$.

3. Step 3. Determine d_{50}.

   a. Use Figure 34-8B, Riprap Basin Depth of Scour.
   b. Select d_{50}/d_E. Satisfactory results will be obtained if $0.25 < d_{50}/d_E < 0.45$.
   c. Obtain h_S/d_E using Fr and Figure 34-8B.
   d. Check if $2 < h_S/d_{50} < 4$ and repeat until d_{50} is found to be within the range.

4. Step 4. Size basin as shown in Figure 34-8A, Details of Riprap Basin Energy Dissipator.

   a. Determine length of the dissipating pool, $L_S = 10h_S$ or $3W_o$ minimum.

   b. Determine length of basin, $L_B = 15h_S$ or $4W_o$ minimum.

   c. Thickness of riprap:
      (1) Approach, $3d_{50}$ or $1.5d_{max}$
      (2) Remainder, $2d_{50}$ or $1.5d_{max}$

5. Step 5. Determine V_B.

   a. Basin exit depth, d_B = critical depth at basin exit.
   b. Basin exit velocity, $V_B = Q/W_B d_B$.
   c. Compare V_B with the average normal flow velocity in the natural channel, V_d.

6. Step 6. High Tailwater Design.

   a. Design a basin for low tailwater conditions, Steps 1-5.

   b. Compute equivalent circular diameter, D_E, for brink area as follows:

$$A = \frac{\pi D_E^{\ 2}}{4} = d_o W_o$$

   c. Estimate centerline velocity at a series of downstream cross sections using Figure 34-8D, Distribution of Centerline Velocity for Flow from Submerged Outlets.

   d. Size riprap using HEC 11, Use of Riprap For Bank Protection.

7. Step 7. Design Filter. This is necessary unless the streambed material is sufficiently well-graded. Follow the instructions shown in HEC 11, Section 4.4.

* * * * * * * *

34-8.03 Low Tailwater, Example 34-8.1

Given: Box culvert, 8 ft by 6 ft
Design discharge Q = 800 ft^3/s
Supercritical flow in culvert
Normal flow depth, d_o = brink depth; d_E = 4.00 ft
Tailwater depth, TW = 2.83 ft

1. Step 1. Determine Input Flow.

$d_o = d_E$ for rectangular section
$d_o = d_E = 4$ ft
$V_o = Q/A = 800/(4)8 = 25$ ft/s

$$Fr = \frac{V}{\left(gd_g\right)^{0.5}} = \frac{25}{\left[(32.2)(4)\right]^{0.5}} = 2.20 < 3.0 \text{, therefore OK}$$

2. Step 2. Check TW.

Determine if $TW/d_o \leq 0.75$
$TW/d_E = 2.83/4.0 = 0.71$
$TW/d_E < 0.75$, therefore OK

3. Step 3. Determine d_{50}.

a. Use Figure 34-8B.

b. Select $d_{50}/d_E = 0.45$
$d_{50} = 0.45(4) = 1.8$ ft

c. Obtain h_S/d_E using $Fr = 2.2$ and $0.41 \leq d_{50}/d_E \leq 0.5$
$h_S/d_E = 1.6$

d. Check if $2 < h_S/d_{50} < 4$:
$h_S = 4(1.6) = 6.4$ ft
$h_S/d_{50} = 6.4/1.8 = 3.55$ ft
$2 < 3.55 < 4$, therefore OK

4. Step 4. Size basin as shown in Figure 34-8A, Details of Riprap Basin Energy Dissipator.

   a. Determine length of dissipating pool, L_S.
   $L_S = 10h_S = 10(6.50) = 65.00$ ft
   minimum $= 3W_o = 3(8) = 24.00$ ft
   Therefore, use $L_S = 65.00$ ft

   b. Determine length of basin, L_B.
   $L_B = 15h_S = 15(6.50) = 97.50$ ft
   minimum $= 4W_o = 4(8.00) = 32.00$ ft
   Therefore, use $L_B = 97.50$ ft

   c. Determine the thickness of riprap.
   Approach, $3d_{50} = 3(1.83) = 5.49$ ft
   Remainder, $2d_{50} = 2(1.83) = 3.66$ ft

5. Step 5. Determine V_B.

   a. d_B = critical depth at basin exit = 3.30 ft, assuming a rectangular cross section with width $W_B = 24$ ft

   b. $V_B = Q/W_B d_B = 800/(24)(3.30) = 10.10$ ft/s

   c. $V_B = 10.10$ ft/s $< V_d = 18.3$ ft/s

* * * * * * * *

34-8.04 High Tailwater, Example 34-8.2

Given: Data for the channel and the culvert are the same as for Example 34-8.1, except that the tailwater depth, $TW = 4.26$ ft.
$TW/d_o = 4.20/4.00 = 1.06 > 0.75$
Downstream channel can tolerate only 7 ft/s.

1. Steps 1 through 5 are the same as in Example 34-8.1.

2. Step 6. High-Tailwater Design.

   a. Design a basin for the low-tailwater condition, Steps 1-5 as above:

d_{50} = 1.83 ft, h_S = 6.50 ft
L_S = 65.00 ft, L_B = 97.50 ft

b. Compute equivalent circular diameter, D_E, for the brink area as follows:

$$A = \frac{\pi D_E^{\ 2}}{4} = d_o W_o = 4(8) = 32.00 \ ft^2$$

$$D_E = \left[\frac{(32.0)4}{\pi}\right]^{0.5} = 6.5 \ ft$$

V_o = 25.4 ft/s

c. Estimate centerline velocity at a series of downstream cross sections using Figure 34-8D, Distribution of Centerline Velocity for Flow from Submerged Outlets.

L/D_E [1]	L	V_L/V_o	V_L	d_{50} [2]
10	65.00	0.59	15.0	1.43
15 [3]	97.50	0.37	9.40	0.60
20	123.0	0.30	7.73	0.40
21	137.2	0.28	7.73	0.40

[1] Use $W_o = D_E$ in Figure 34-8D.
[2] from Figure 34-8E, Riprap Size Versus Exit Velocity (After HEC 14).
[3] is on a logarithmic scale, therefore, interpolations must be made logarithmically.

d. Size riprap using HEC 11. The channel can be lined with the same size rock used for the basin. Protection must extend at least 135 ft downstream.

34-8.05 Computer Output

The dissipator geometry can be computed using the HY-8, Culvert Analysis microcomputer program, Energy Dissipator module. The output of the culvert and channel input data, and computed geometry using this module are shown as Figure 34-8G, Riprap Stilling Basin HY-8 Program Output.

34-9.0 IMPACT BASIN USBR TYPE VI

34-9.01 Overview

Figure 34-9A, USBR Type VI Impact Dissipator, was developed by the U.S. Bureau of Reclamation (USBR). The basin requirements are as follows:

1. it is referred to as the USBR Type VI basin, or hanging baffle;
2. it is contained in a relatively small boxlike structure;
3. it requires no tailwater for successful performance;
4. it also may be used for an open channel; and
5. it is not recommended where debris or ice buildup may cause substantial clogging.

The following applies to the USBR Type VI basin.

1. 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.

2. 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.

3. 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.

4. Limitations. A discharge of up to 400 ft^3/s per barrel and a velocity as high as 50 ft/s can be used without subjecting the structure to cavitation damage.

5. Tailwater. A moderate depth of tailwater will improve performance. For best performance, set the basin so that the maximum tailwater does not exceed $h_3 + (h_2/2)$.

6. Slope. If the culvert slope is greater than 15 deg, a horizontal section of at least four culvert widths should be provided upstream.

7. End Treatment. An end sill with a low-flow drainage slot, 45-deg wingwalls, and a cutoff wall should be provided at the end of the basin.

8. Riprap. Riprap should be placed downstream of the basin for a length of at least four conduit widths.

34-9.02 Design Procedure

1. Step 1. Calculate Equivalent Depth, d_E.

   a. Rectangular section, $d_E = d_o = y_o$.
   b. Other type of section, $d_E = (A/2)^{0.5}$.

2. Step 2. Determine Input Flow.

   a. Froude number, $Fr = V_o/(gd_E)^{0.5}$.
   b. Specific energy, $H_o = d_E + V_o^2/2g$.

3. Step 3. Determine Basin Width, W.

   a. Use Figure 34-9B, Design Curve for USBR Type VI Dissipator.
   b. Enter with Fr and read H_o/W.

4. Step 4. Size Basin.

   a. Use Figure 34-9C, Dimensions of USBR Type VI Basin, and W.
   b. Obtain the remaining dimensions.

5. Step 5. Energy Loss.

   a. Use Figure 34-9D, Energy Loss for USBR Type VI Dissipator.
   b. Enter with Fr and read H_L/H_o.

6. Step 6. Exit Velocity, V_B.

   a. Exit energy, $H_E = H_o - H_L$.
   b. $H_E = d_B + V_B^2/2g$.
   c. $V_B = Q/Wd_B$.

* * * * * * * *

Example 34-9.1

An editable version of Figure 34-9E, Impact Basin Type VI Checklist, may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/.

Given: D = 48-in. dia. pipe, S_o = 0.15, n = 0.015

Q = 300 ft^3/s, d_o = 2.3 ft, V_o = 40.00 ft/s

1. Step 1. Calculate Equivalent Depth, d_E.
   Other type of section, $d_E = (A/2)^{0.5}$
   $A = Q/V_o$ = 300/40.00 = 7.50 ft^2
   $d_E = (7.50/2)^{0.5}$ = 1.94 ft

2. Step 2. Determine Input Flow.

   a. Froude number, $Fr_o = V_o/(gd_E)^{0.5}$
   $Fr = 40.63/[32.2(1.94)]^{0.5} = 5.06$

   b. Specific energy, $H_o = d_E + V_o^2/2g$
   $H_o = 1.94 + (40.00)^2/(2)(32.2) = 26.78$ ft

3. Step 3. Determine Basin Width, W.

   a. Use Figure 34-9B, Design Curve for USBR Type VI Dissipator.
   b. Enter with Fr = 5.06 and read H_o/W = 1.68
   c. W = 26.78/1.68 = 15.94 ft ≈ 16 ft

4. Step 4. Size Basin.

   a. Use Figure 34-9C, Dimensions of USBR Type VI Basin, and W.
   b. Obtain the remaining dimensions.

5. Step 5. Energy Loss.

   a. Use Figure 34-9D, Energy Loss for USBR Type VI Dissipator.
   b. Enter with Fr = 5.05 and read H_L/H_o = 0.67
   c. H_L = 0.66(26.78) = 17.67 ft

6. Step 6. Exit Velocity, V_B.

   a. Exit energy, $H_E = H_o - H_L$ = 26.78 – 17.67 = 9.10 ft
   b. $H_E = d_B + V_B^2/2g$ = 9.10 ft
   c. $V_B = Q/Wd_B = (300)/16d_B = 18.75/d_B$

d_B	V_B	$d_B+V_B^2/2g = 9.10$
2.33 = d_c	8.33	3.41
1.00	19.4	6.84

0.67	29.2	13.9
0.87	22.4	8.66
0.90	21.6	8.14
0.83	23.3	9.25, approx. 9.10, use.

34-9.04 Computer Output

The dissipator geometry can be computed using the HY-8 Culvert Analysis, microcomputer program, Energy Dissipator module. The output of the culvert and channel input data, and computed geometry using this module are shown as Figure 34-9G, USBR Type 6 Dissipator HY-8 Program Output.

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Symbol	Definition	Unit
A	Cross-sectional area	ft^2
A_o	Area of flow at culvert outlet	ft^2
D	Height of culvert	in.
d_{50}	Mean diameter of riprap	in.
d_E	Equivalent depth at brink	ft
d_o	Normal flow depth at brink	ft
DI	Discharge Intensity Modified	(none)
Fr	Froude number	(none)
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/s
S_o	Slope of streambed	ft/ft
TW	Tailwater depth	ft
V_d	Velocity downstream	ft/s
V_L	Velocity at L from brink	ft/s
V_o	Normal velocity at brink	ft/s
W_o	Diameter or width of culvert	in.
W_S	Width of scour hole	ft

SYMBOLS, DEFINITIONS, AND UNITS

Figure 34-1A

A. Coefficient for Culvert-Outlet Scour, Cohesionless Materials

Property	α	β	θ
Depth, d_S	2.27	0.39	0.06
Width, W_S	6.94	0.53	0.08
Length, L_S	17.10	0.47	0.10
Volume, V_S	127.08	1.24	0.18

B. Coefficient C_S for Outlet Above the Bed

H_S	Depth	Width	Length	Volume
0	1.00	1.00	1.00	1.00
1	1.22	1.51	0.73	1.28
2	1.26	1.54	0.73	1.47
4	1.34	1.66	0.73	1.55
H_S is the height above bed in pipe diameters, ft.				

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

COEFFICIENTS FOR SCOUR-HOLE ESTIMATION

Figure 34-4A

ENERGY DISSIPATOR CHECKLIST

Route Des No. Project No.
Designer: Date:
Reviewer: Date:

SCOUR EQUATIONS

$$\frac{d_s}{R_c}, \frac{W_s}{R_c}, \frac{L_s}{R_c} = C_{sCk}\left(\frac{a}{\sigma^{1/3}}\right)\left(\frac{Q}{g^{0.5}R_c^{\ 2.5}}\right)^{\beta}\left(\frac{t}{316}\right)^{\theta}$$

$$d_s, W_s, L_s = \left(\frac{C_s C_h \alpha}{\sigma^{1/3}}\right)(DI)^{\beta}\left(\frac{t}{316}\right)^{\theta}(R_c)$$

$$d_s, W_s, L_s = (F_1)(F_2)(F_3)(R_c)$$

STEP 6 – DATA SUMMARY

Parameters	Culvert	Channel
Station		
Control		
Type		
Height, *D*		
Width, *B*		
Length, *L*		
Material		
Manning's *n*		
Side Slope		
Discharge, *Q*		
Depth, *d*		
Velocity, *V*		
$Fr = V/(gd)^{0.5}$		
Flow Area, *A*		
Slope		

STEP 7A – EQUATION INPUT DATA

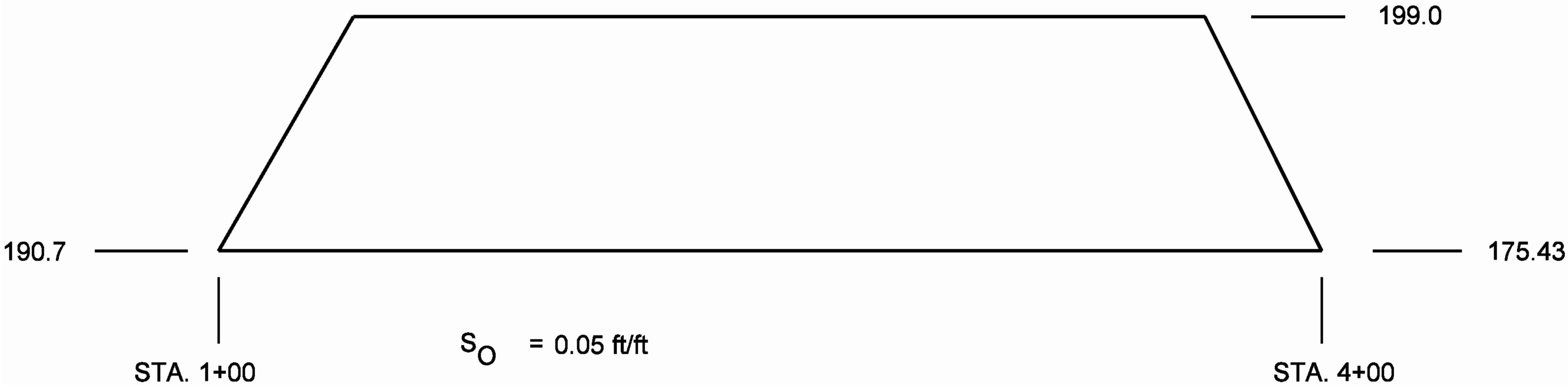
FACTOR	Value
Q = Discharge, ft^3/s	
A_c = Culvert area, ft^2	
P_c = Perimeter, ft	
$R_c = A_c / P_c$	
DI = Discharge Intensity	
t = time of concentration	

STEP 7B – SCOUR COMPUTATION

Factor	Depth, ft	Width, ft	Length, ft
α	2.27	6.94	17.10
β	0.39	0.53	0.47
θ	0.06	0.08	0.10
F_1			
F_2			
F_3			
$(F_1)(F_2)(F_3)(R_c)$			
Allowable			

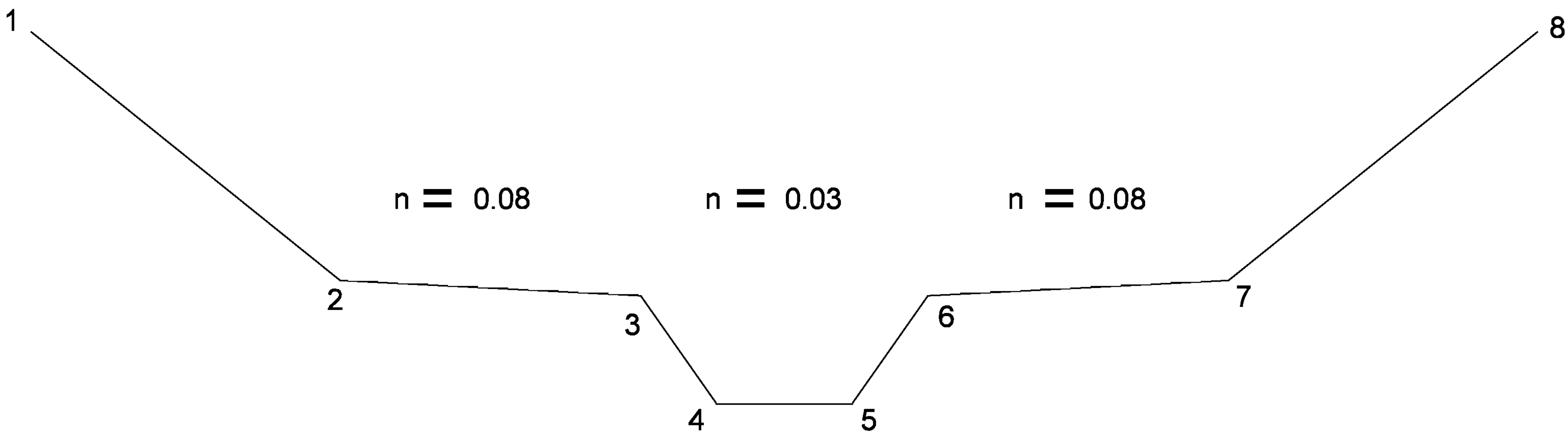
If calculated scour > allowable scour, and:

1. $Fr > 3$, design a SAF basin
2. $Fr \leq 3$, design a riprap basin
3. $Fr \leq 3$, design a USBR Type VI



Energy Dissipator Design Example - Site Data

Figure 34-6.1



ENERGY DISSIPATOR DESIGN EXAMPLE - DOWNSTREAM CHANNEL SECTION

Figure 34-6.2

ENERGY DISSIPATOR CHECKLIST

Project No.	I-31(88) over Example Creek		
Designer	PLT	Date	8/26/88
Reviewer	Te Anh Ngo, OK. DOT	Date	6/2/97

SCOUR EQUATIONS

$$\frac{d_S}{R_C},\frac{W_S}{R_C},\frac{L_S}{R_C}=C_S C_h\left[\frac{\alpha}{\sigma^{1/3}}\right]\left[\frac{Q}{g^{.5}R_C^{2.5}}\right]^{\beta}\left[\frac{t}{316}\right]^{\theta}$$

$$d_S,W_S,L_S = [C_S C_h \alpha/\sigma^{1/3}][DI]^{\beta}[t/316]^{\theta}\ R_c$$

$$d_S,W_S,L_S = [F_1]\ [F_2]\ [F_3]\ R_c$$

STEP 6 - DATA SUMMARY

Parameters	Culvert	Channel
Station	1 + 01	4 + 06
Control	Inlet	Super.
Type	RCB	Natural
Height, D	6 ft	7.6 ft
Width, B	7 ft	29.5 ft
Length, L	305 ft	-----
Material	Concrete	Gravel
Manning's n	0.012	0.03 & 0.08
Side Slope	---	1V:1H
Discharge, Q	400 ft^3/s	400 ft^3/s
Depth, d	1.9 ft	2.9 ft
Velocity, V	28.7 ft/s	17.8 ft/s
$Fr = V/(gd)^{0.5}$	3.54	2.01
Flow Area, A	14.4 ft^2	23.5 ft^2
Slope	0.05 ft/ft	0.05 ft/ft

STEP 7A - EQUATION INPUT DATA

FACTOR	VALUE
Q = Discharge, ft^3/s	400 ft^3/s
A_c = Culvert area, ft^2	43 ft^2
P_c = Perimeter, ft	26 ft
$R_c = A_c / P_c$	1.63
DI= Discharge Intensity	21.1
t = time of concentration	30 minutes

STEP 7B - SCOUR COMPUTATION

Factor	Depth	Width	Length
α	2.27	6.94	17.10
β	0.39	0.53	0.47
θ	0.06	0.08	0.10
F_1	1.92	6.94	15.62
F_2	3.28	5.03	4.19
F_3	0.87	0.83	0.79
$[F_1][F_2][F_3]R_c$	8.93	47.2	84.3
Allowable	7.0*	29.5*	65.0*

If calculated scour > Allowable and:
1. Fr > 3, design a SAF basin
2. Fr < 3, design a riprap basin
3. Fr < 3, design a USBR Type VI

* These values are not standards. They may vary, depending on design criteria. In this case, calculated scour > Allowable and Q < 425 ft^3/s:: Recommend a SAF Basin.

Figure 34-6A

FHWA CULVERT ANALYSIS, HY-8, VERSION 6.0

CURRENT DATE	CURRENT TIME	FILE NAME	FILE DATE
06-04-2003	10:56:51	CHPTR11A	06-04-2003

CULVERT AND CHANNEL DATA

CULVERT NO. 1
CULVERT TYPE: 7 ft x 6 ft BOX
CULVERT LENGTH = 305.2 ft
NO. OF BARRELS = 1.0
FLOW PER BARREL = 400 ft^3/s
INVERT ELEVATION = 175.27 ft
OUTLET VELOCITY = 29.15 ft/s
OUTLET DEPTH = 2.05 ft

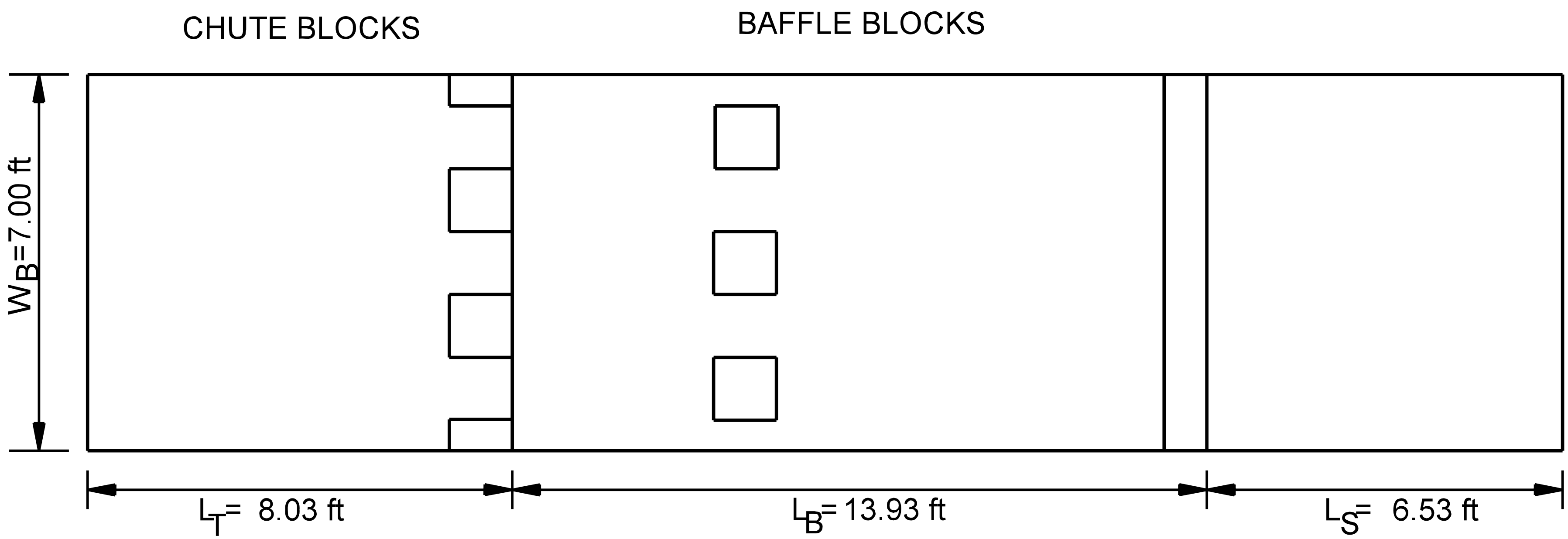
DOWNSTREAM CHANNEL
CHANNEL TYPE : IRREGULAR
BOTTOM WIDTH = 7.00 ft
TAILWATER DEPTH = 2.87 ft
TOTAL DESIGN FLOW = 400 ft^3/s
BOTTOM ELEVATION = 172.27 ft
NORMAL VELOCITY = 17.84 ft/s

SCOUR HOLE GEOMETRY AND SOIL DATA

LENGTH = 95.62 ft
WIDTH = 53.59 ft
DEPTH = 10.05 ft
VOLUME = 17,656.8 ft^3
MAXIMUM SCOUR OCCURS 38.24 ft DOWNSTREAM OF CULVERT
SOIL TYPE: NONCOHESIVE
SAND SIZES:
- D16 = 1/3 in.
- D50 = 1/2 in.
- D84 = 3/4 in.

SCOUR HOLE GEOMETRY HY-8 PROGRAM OUTPUT

Figure 34-6B



ST. ANTHONY FALLS BASIN EXAMPLE - BLOCK DESIGN

Figure 34-7.1

ST. ANTHONY FALLS (SAF) BASIN CHECKLIST

Route Project No.
Designer Date
Reviewer Date

SAF BASIN DESIGN VALUES	TRIAL 1	FINAL TRIAL
Type		
Flare (z:1)		
Width, W_B		
Depression, B_d		
$S_S = S_T$		
Depth, d_o		
Velocity, V_o		
$B_o = d_o + V_o^2/2g$		
Depth, d_1		
Velocity, V_1		
Fr_1		
d_j		
L_B		
d_2		
L_S		
$L_T = B_d / S_T$		
$L = L_B + L_S + L_T$		
$B_d = LS_o + TW$		

DIMENSIONS OF ELEMENTS	TRIAL 1	FINAL TRIAL	DIMENSIONS OF ELEMENTS	TRIAL 1	FINAL TRIAL
CHUTE BLOCK			BAFFLE BLOCK		
Height, h_1			Height, h_3		
Width, W_1			Width, W_3		
Spacing, W_2			Spacing, W_4		
Block No., N_C			Block No., N_B		
END SILL			SIDE WALL		
Height, h_4			Height, h_5		

ST. ANTHONY FALLS (SAF) BASIN

Project No. Example Problem
Designer ______ Date ______
Reviewer Te Anh Ngo Date 9/19/96

SAF BASIN DESIGN VALUES	TRIAL 1	FINAL TRIAL
Type	Rect.	Rect.
Flare (Z:1)	1:1	1:1
Width, W_B	7.00	7.00
Depression, B_d	6.00	6.00
$S_S = S_T$	1:1	1:1
Depth, d_o	2.07	2.07
Velocity, V_o	28.7	28.7
$B_o = d_o+V_o^2/2g$	20.83	20.83
Depth, d_1	1.67	1.60
Velocity, V_1	35.47	36.93
Fr_1	4.80	5.11
d_j	10.50	10.53
L_B	14.33	13.93
d_2	9.53	9.40
L_S	6.67	6.53
$L_T = B_d / S_T$	6.00	8.03
$L = L_B+L_S+L_T$	27.00	28.50
B_d - LS_o + TW	7.52	7.52

DIMENSIONS OF ELEMENTS	TRIAL 1	FINAL TRIAL	DIMENSIONS OF ELEMENTS	TRIAL 1	FINAL TRIAL
CHUTE BLOCK			BAFFLE BLOCK		
Height, h_1	1.60	1.60	Height, h_3	1.60	1.60
Width, W_1	1.17	1.17	Width, W_3	1.17	1.17
Spacing, W_2	1.17	1.17	Spacing, W_4	1.17	1.17
Block No., N_C	3	3	Block No., N_B	3	3
END SILL			SIDE WALL		
Height, h_4	0.73	0.73	Height, h_5	12.87	12.87

ST. ANTHONY FALLS BASIN
(Example Problem)
Figure 34-7B

FHWA CULVERT ANALYSIS, HY-8, VERSION 6.0

CURRENT DATE	CURRENT TIME	FILE NAME	FILE DATE
09-19-2003	15:26:05	CHPTR11A	09-19-2003

CULVERT AND CHANNEL DATA

CULVERT NO. 1	DOWNSTREAM CHANNEL
CULVERT TYPE: 7.0 ft x 6.0 ft BOX	CHANNEL TYPE: IRREGULAR
CULVERT LENGTH = 300.37 ft	BOTTOM WIDTH = 7.12 ft
NO. OF BARRELS = 1.0	TAILWATER DEPTH = 2.87 ft
FLOW PER BARREL = 400 ft^3/s	TOTAL DESIGN FLOW = 400 ft^3/s
INVERT ELEVATION = 175.27 ft	BOTTOM ELEVATION = 175.27 ft
OUTLET VELOCITY = 28.7 ft/s	NORMAL VELOCITY = 17.85 ft/s
OUTLET DEPTH = 2.05 ft	

ST. ANTHONY FALLS BASIN -- FINAL DESIGN

LB = 14.07 ft	LS = 13.88 ft	LT = 18.52 ft
L = 46.47 ft	Y1 = 1.536 ft	Y2 = 9.653 ft
Z1 = 166.00 ft	Z2 = 166.00 ft	Z3 = 172.94 ft
WB = 7.000 ft	WB3 = 7.000 ft	

------------------------------ CHUTE BLOCKS ------------------------------

H1 = 1.532 ft	W1 = 1.167 ft	W2 = 1.167 ft	NC = 3.000

------------------------------ BAFFLE BLOCKS -----------------------------

W3 = 1.167 ft	W4 = 1.167 ft	NB = 3.000
H3 = 1.60 ft		LCB = 4.69 ft

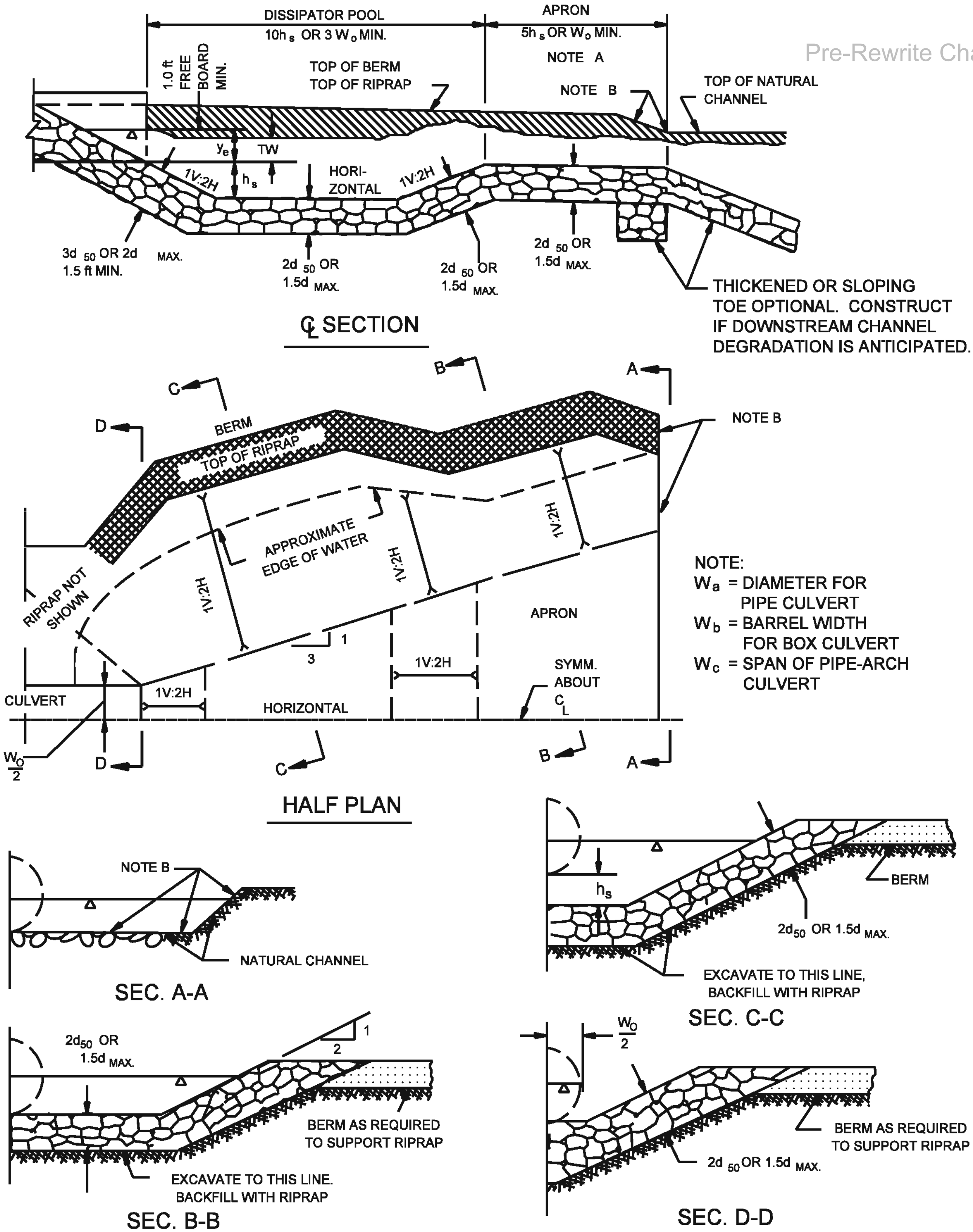
------------------------------ END SILL ----------------------------

H4 = 0.73 ft

BASIN OUTLET VELOCITY = 17.85 ft/s

ENERGY DISSIPATOR HY-8 PROGRAM OUTPUT

Figure 34-7C

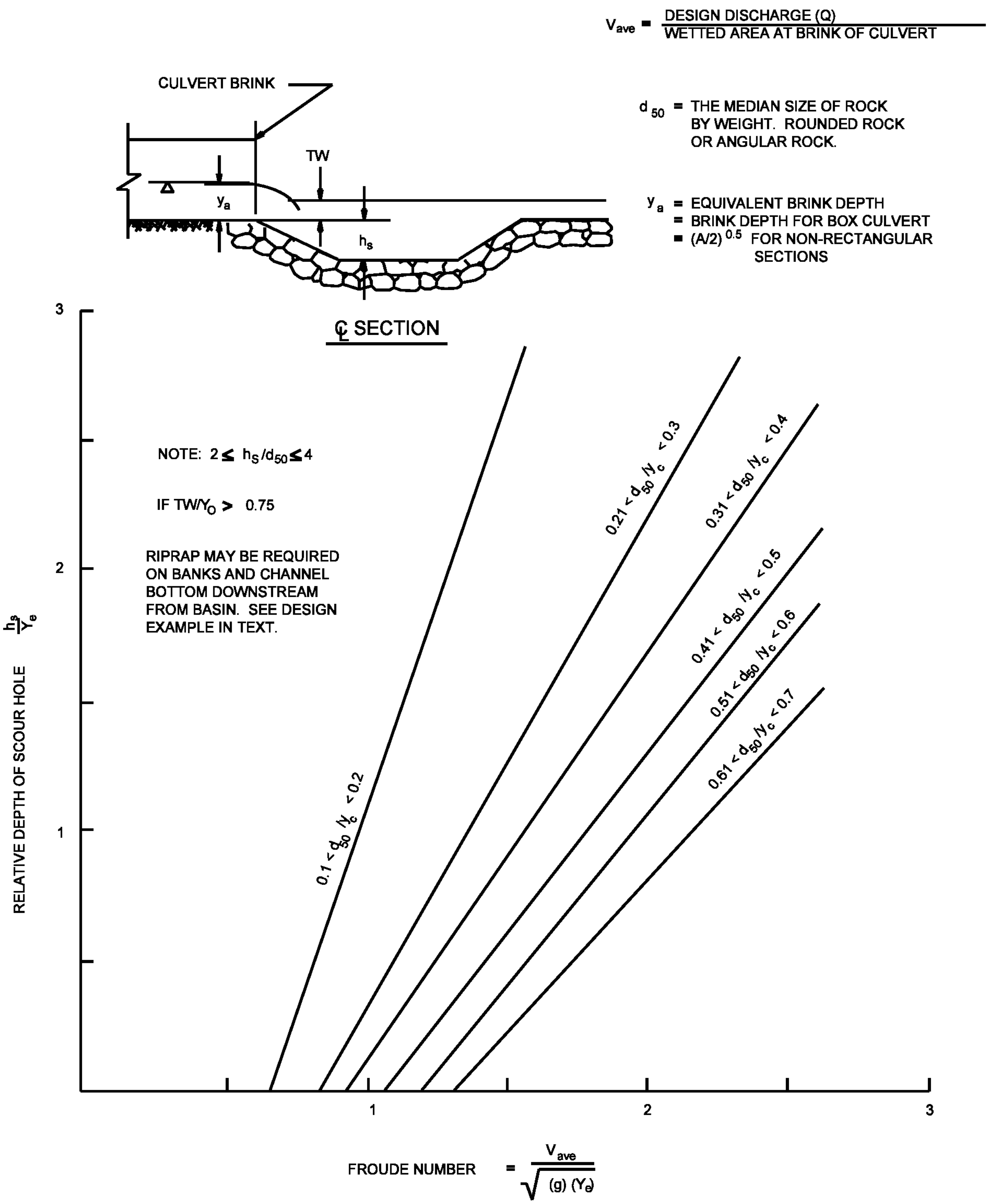


NOTE A - IF EXIT VELOCITY OF BASIN IS SPECIFIED, EXTEND BASIN AS REQURIED TO OBTAIN SUFFICIENT CROSS-SECTIONAL AREA AT SECTION A-A SUCH THAT Q_{des}/(CROSS SECTION AREA AT SEC. A-A) = SPECIFIED EXIT VELOCITY.

NOTE B - WARP BASIN TO CONFORM TO NATURAL STREAM CHANNEL. TOP OF RIPRAP IN FLOOR OF BASIN SHOULD BE AT THE SAME ELEVATION OR LOWER THAN NATURAL CHANNEL BOTTOM AT SEC. A-A.

DETAILS OF RIPRAP BASIN ENERGY DISSIPATOR

Figure 34-8A



RIPRAP BASIN DEPTH OF SCOUR

Figure 34-8B

RIPRAP BASIN CHECKLIST

Route	Project No.
Designer	Date
Reviewer	Date

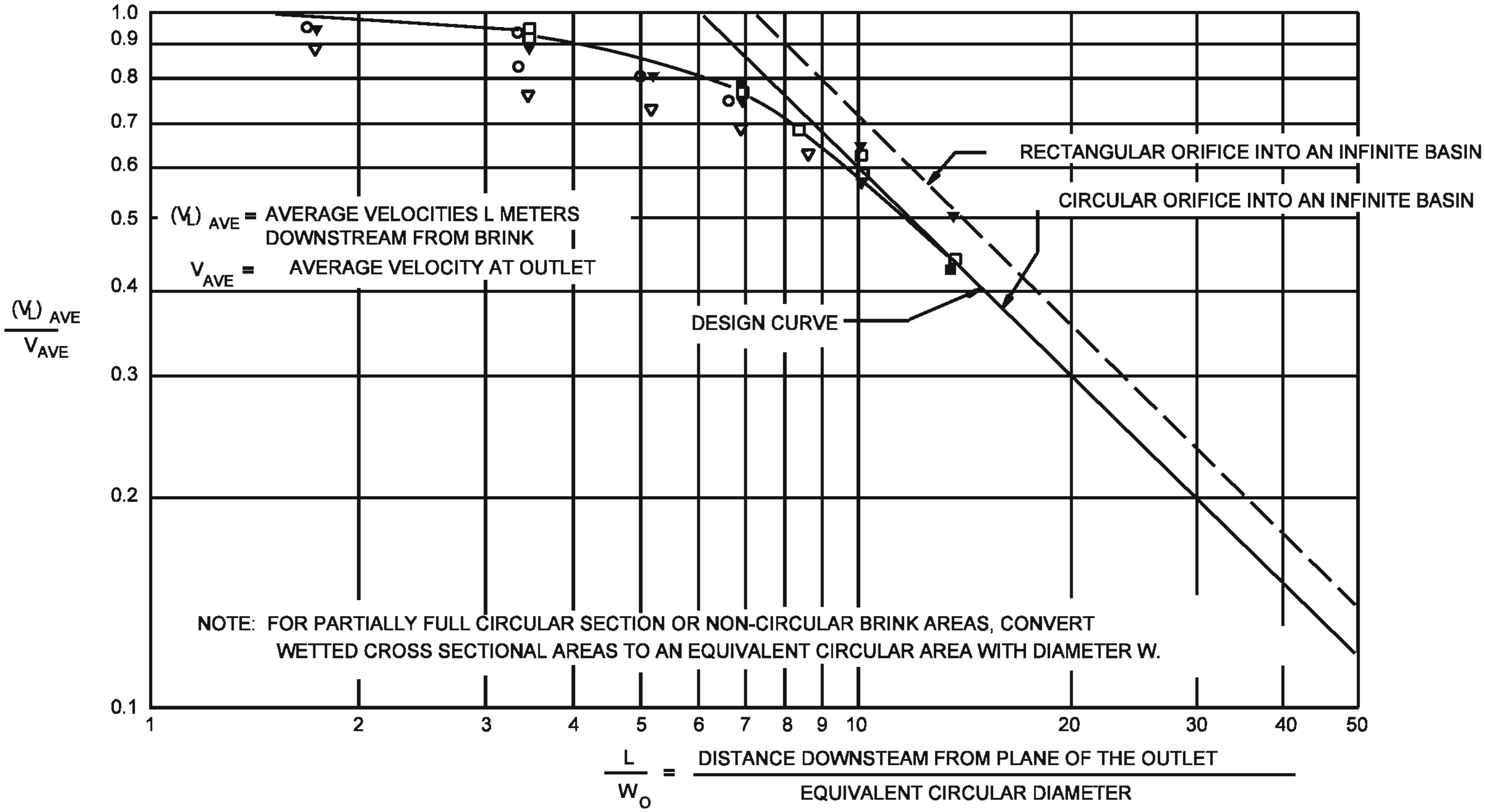
DESIGN VALUES (*IDM* Figure 34-8B)	TRIAL 1	FINAL TRIAL
Equivalent Depth, d_E		
D_{50}/d_E		
D_{50}		
Froude No., Fr		
h_S/d_E		
h_S		
h_S/D_{50}		
$2 < h_S/D_{50} < 4$		

BASIN DIMENSIONS		FEET	
Pool length is the larger of:	$10h_S$		
	$3W_o$		
Basin length is the larger of:	$15h_S$		
	$4W_o$		
Approach Thickness	$3D_{50}$		
Basin Thickness	$2D_{50}$		

TAILWATER CHECK	
Tailwater, TW	
Equivalent depth, d_E	
TW/d_E	
IF $TW/d_E > 0.75$, calculate riprap downstream using *IDM* Figure 34-8D	
$D_E = (4A_c/\pi)^{0.5}$	

DOWNSTREAM RIPRAP (*IDM* Figure 34-8D)

L/D_E	L	V_L/V_o	V_L	D_{50}

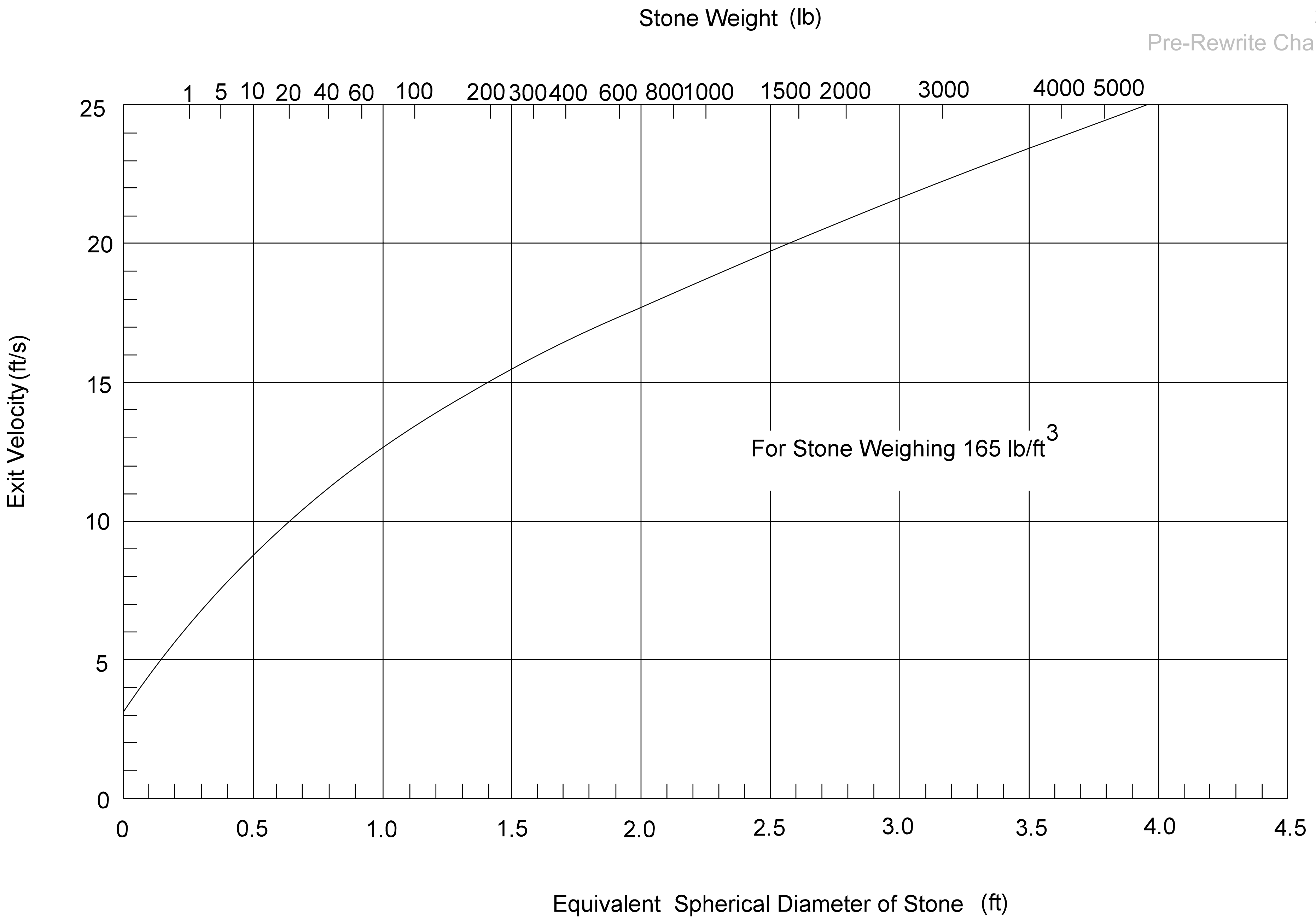


SYM	W_O (ft)	Q (ft³/s)	V_{AVE} (ft/s)	TW (ft)
□	1.44	23	15.09	1.61
□	1.44	14	10.30	1.61
○	3.08	65	9.29	3.08
●	3.08	84	11.91	3.08
▽	1.44	21	14.01	1.25
▽	1.44	14	9.29	1.25

Note: To be used for predicting channel velocities downstream from culvert outlets where hihg tailwater prevails. Velocities obtained from te uf this figure can be used with Figure 2 of HEC 11 for sizing rip rap. (Do not us Fig of HEC 11: use Mean Velocity Values.)

DISTRIBUTION OF CENTERLINE VELOCITY FLOW
FROM SUBMERGED OUTLETS

Figure 34-8D



RIPRAP SIZES VERSUS EXIT VELOCITY (AFTER HEC 14)

Figure 34-8E

RIPRAP BASIN

Project No. ______________________________
Designer ______________________ Date __________
Reviewer ______________________ Date __________

DESIGN VALUES (Figure 34-8B)	TRIAL 1	FINAL TRIAL
Equi. Depth, d_E	4.00 ft	4.00ft
D_{50}/d_E	0.45	0.45
D_{50}	1.83 ft	1.83 ft
Froude No., Fr	2.20	2.20
h_S/d_E	1.60	1.60
h_S	6.50 ft	6.50 ft
h_S/D_{50}	3.55	3.55
$2 < h_S/D_{50} < 4$	OK	

BASIN DIMENSIONS		METERS	
Pool length is the larger of:	$10h_S$	65.0	65.0
	$3W_o$	24.0	
Basin length is the larger of:	$15h_S$	97.5	97.5
	$4W_o$	32.0	
Approach Thickness	$3D_{50}$	5.40	
Basin Thickness	$2D_{50}$	3.60	

TAILWATER CHECK	
Tailwater, TW	4.20 ft
Equivalent depth, d_E	4.00 ft
TW/d_E	1.05
IF $TW/d_E > 0.75$, calculate riprap downstream using Figure 34-8D	
$D_E=(4A_c/\ \ )^{0.5}$	

DOWNSTREAM RIPRAP (Figure 34-8D)				
L/D_E	L	V_L/V_o	V_L	D_{50}
10	65	0.59	15.0	0.43
15	97.5	0.37	9.40	0.18
20	130	0.30	7.73	0.12
21	137	0.28	7.73	0.12

RIPRAP BASIN DESIGN EXAMPLE

Figure 34-8F

FHWA CULVERT ANALYSIS, HY-8, VERSION 6.0

CURRENT DATE	CURRENT TIME	FILE NAME	FILE DATE
06-02-2003	15:23:59	ENERGY3	06-02-2003

CULVERT AND CHANNEL DATA

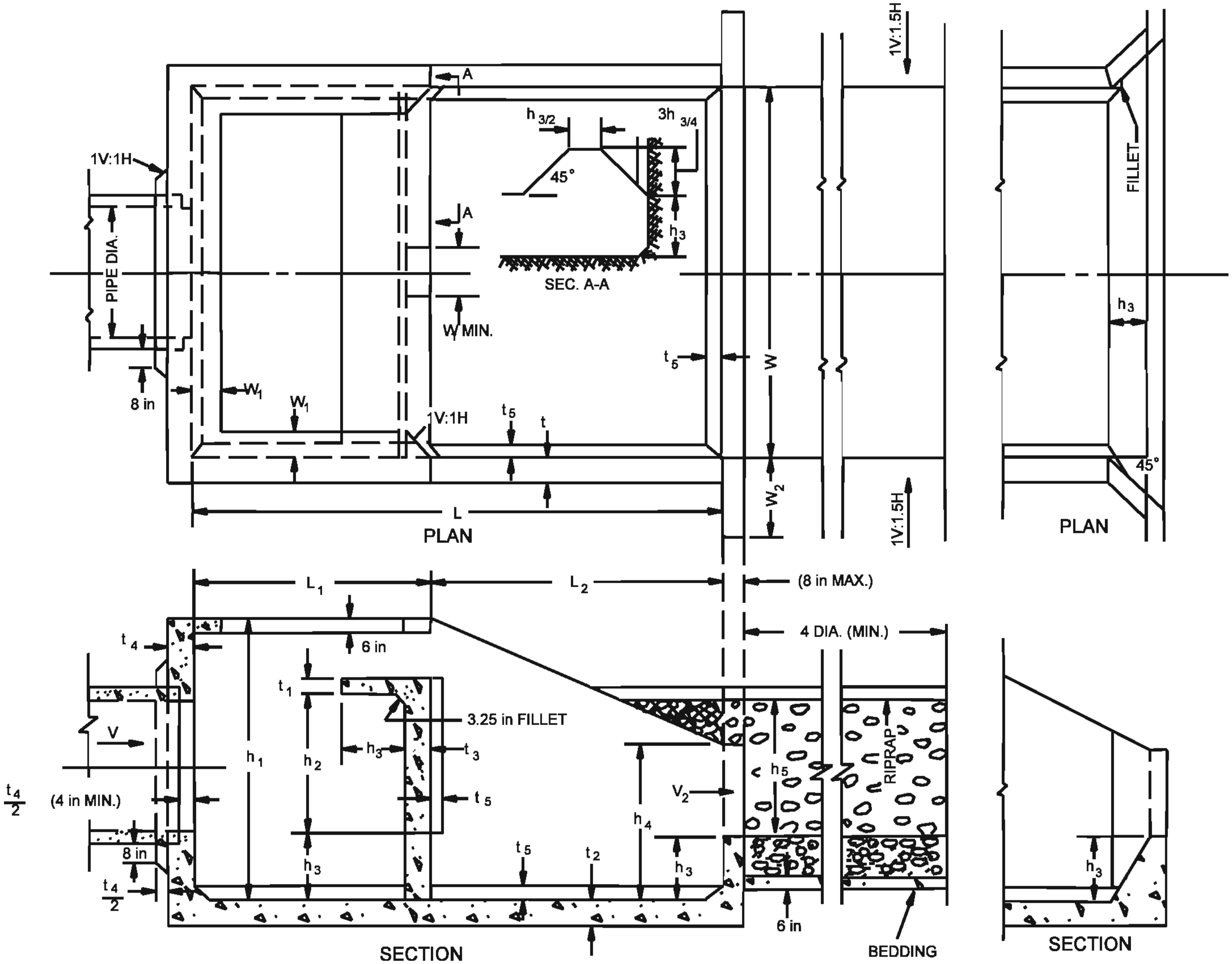
CULVERT NO. 1	DOWNSTREAM CHANNEL
CULVERT TYPE: 8.00 ft x 6.00 ft BOX	CHANNEL TYPE: IRREGULAR
CULVERT LENGTH = 305.0 ft	BOTTOM WIDTH = 8.00 ft
NO. OF BARRELS = 1.0	TAILWATER DEPTH = 2.840 ft
FLOW PER BARREL = 800 ft^3/s	TOTAL DESIGN FLOW = 800 ft^3/s
INVERT ELEVATION = 175.26 ft	BOTTOM ELEVATION = 175.27 ft
OUTLET VELOCITY = 25.82 ft/s	NORMAL VELOCITY = 32.15 ft/s
OUTLET DEPTH = 4.06 ft	

RIPRAP STILLING BASIN -- FINAL DESIGN

THE LENGTH OF THE BASIN	=	97.840 ft
THE LENGTH OF THE POOL	=	65.227 ft
THE LENGTH OF THE APRON	=	32.613 ft
THE WIDTH OF THE BASIN AT THE OUTLET	=	8.000 ft
THE DEPTH OF POOL BELOW CULVERT INVERT	=	6.523 ft
THE THICKNESS OF THE RIPRAP ON THE APRON	=	6.667 ft
THE THICKNESS OF THE RIPRAP ON THE REST OF THE BASIN	=	5.000ft
THE BASIN OUTLET VELOCITY	=	17.487 ft/s
THE DEPTH OF FLOW AT BASIN OUTLET	=	6.000 ft

RIPRAP STILLING BASIN HY-8 PROGRAM OUTPUT

Figure 34-8G

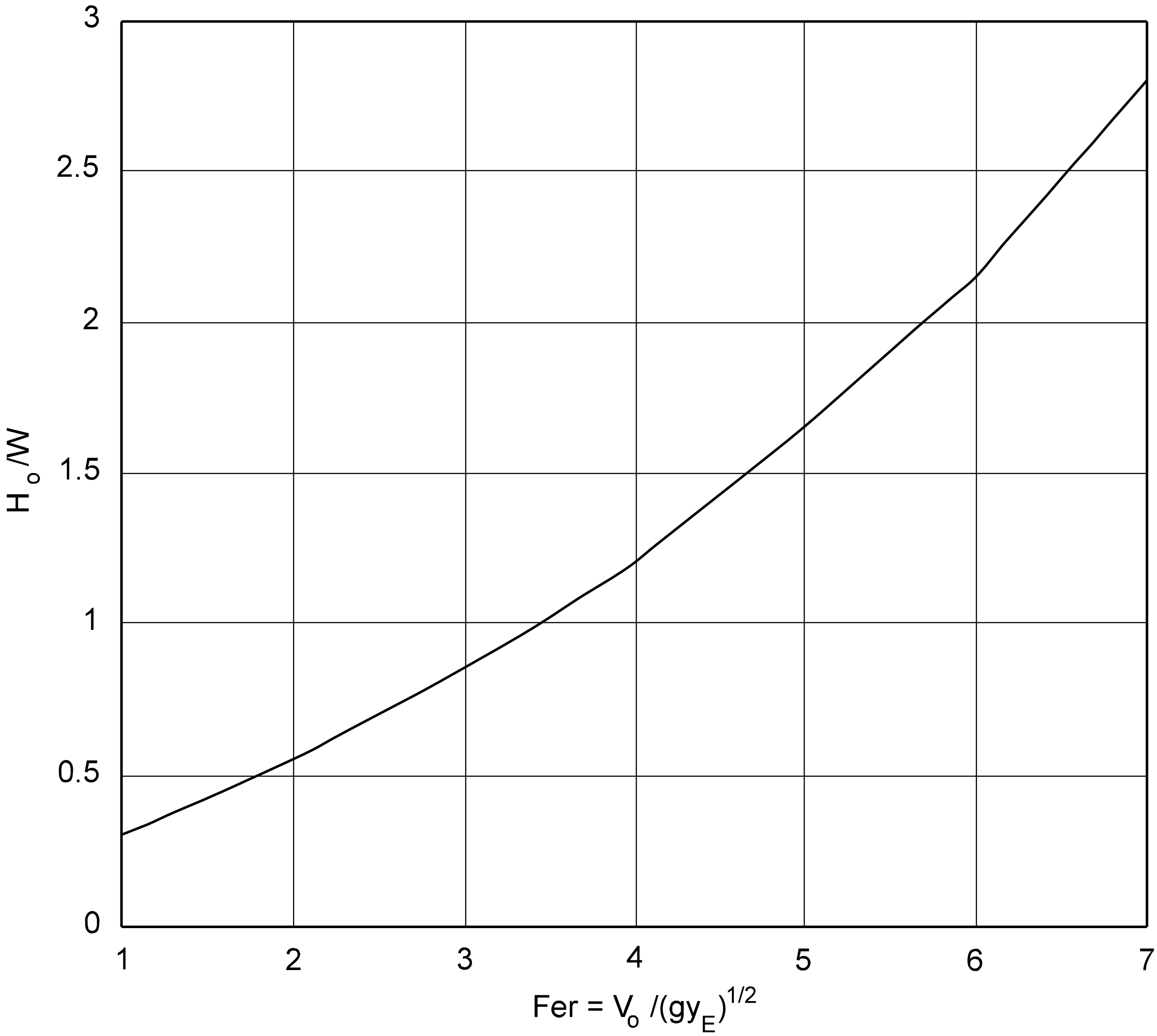


STILLING BASIN DESIGN

ALTERNATIVE END SILL

USBR TYPE VI (IMPACT) DISSIPATOR

Figure 34-9A



DESIGN CURVE FOR USBR TYPE VI DISSIPATOR

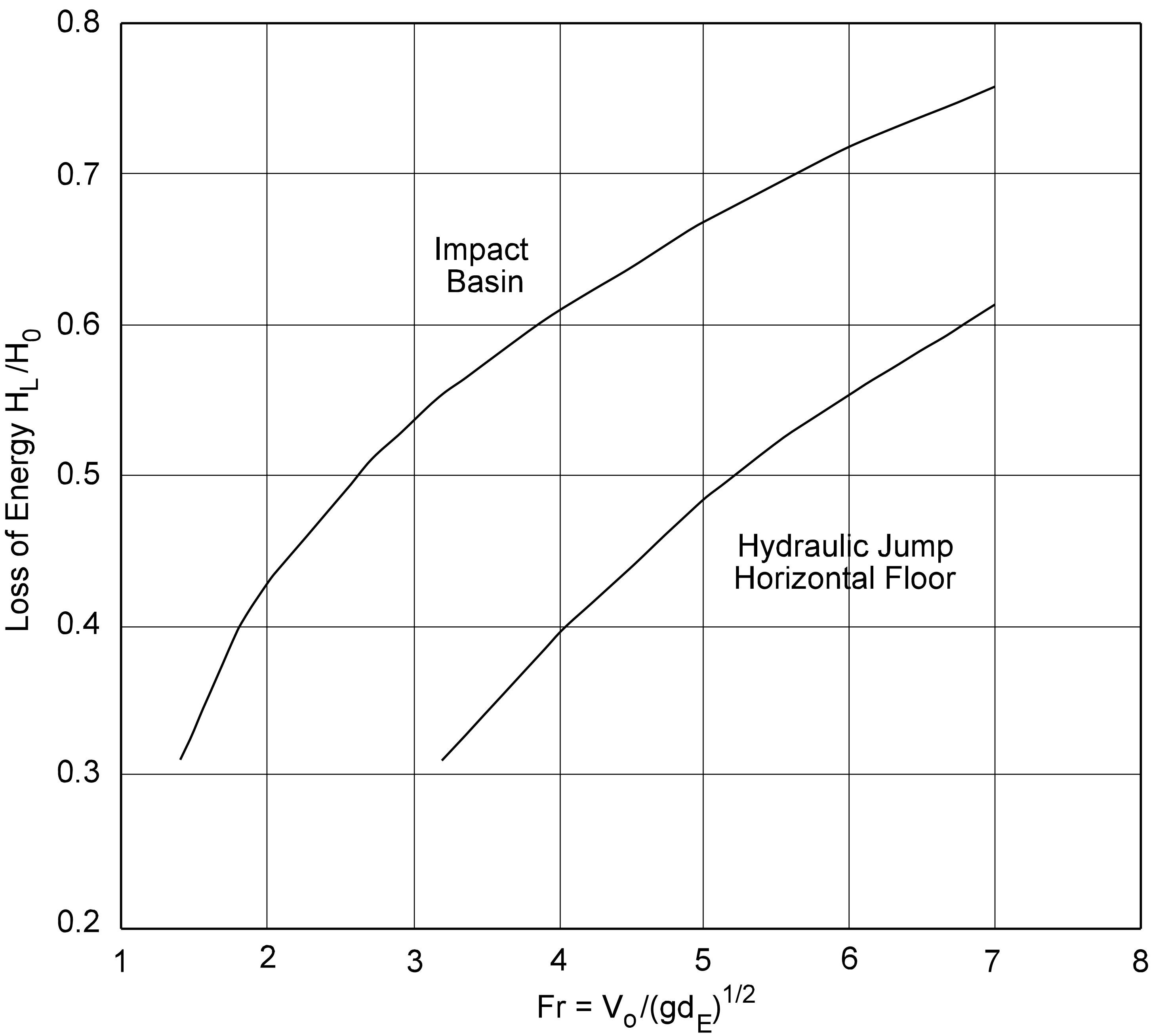
Figure 34-9B

W	h_1	h_2	h_3	h_4	*L*	L_1	L_2
3.33	2.63	1.27	0.57	1.43	4.67	1.97	2.63
5.00	3.87	1.90	0.83	2.07	6.67	2.93	3.87
6.67	5.13	2.50	1.10	2.77	8.93	3.80	5.13
8.33	6.43	3.13	1.40	3.47	11.10	4.77	6.43
10.00	7.67	3.73	1.67	4.17	13.40	5.73	7.67
11.67	8.93	4.40	1.93	4.87	15.50	6.67	8.93
13.33	10.40	5.03	2.23	5.57	17.77	7.60	10.27
15.00	11.53	5.60	2.50	6.27	20.00	8.53	11.53
16.67	12.73	6.23	2.77	6.93	21.73	9.47	12.73
18.33	13.97	6.77	3.03	7.63	24.30	10.40	13.97
20.00	15.33	7.50	3.33	8.33	26.60	11.40	15.33
W	W_1	W_2	t_1	t_2	t_3	t_4	t_5
3.33	0.27	0.87	0.50	0.50	0.50	0.50	0.27
5.00	0.43	1.40	0.50	0.50	0.50	0.50	0.27
6.67	0.50	1.83	0.50	0.50	0.50	0.50	0.27
8.33	0.60	2.27	0.53	0.60	0.60	0.53	0.27
10.00	0.73	2.77	0.67	0.67	0.73	0.67	0.27
11.67	0.87	3.03	0.67	0.77	0.77	0.70	0.33
13.33	1.00	3.03	0.67	0.93	0.83	0.83	0.33
15.00	1.20	3.03	0.67	1.00	1.00	1.00	0.43
16.67	1.30	3.03	0.73	1.03	1.00	1.00	0.50
18.33	1.37	3.03	0.73	1.10	1.10	1.10	0.60
20.00	1.50	3.03	0.83	1.20	1.17	1.17	0.63

Note: The h, L, *and* W *values in are in feet. The* t *values are in seconds.*

DIMENSIONS OF USBR TYPE VI BASIN

Figure 34-9C



ENERGY LOSS FOR USBR TYPE VI DISSIPATOR

Figure 34-9D

USBR TYPE VI BASIN CHECKLIST

Route	Project No.
Designer	Date
Reviewer	Date

DETERMINE BASIN WIDTH, W (*IDM* Figure 34-9B)	TRIAL 1	FINAL TRIAL
Equivalent Depth, d_E		
V_o (ft/s)		
$H_o = d_E + V_o^2/2g$		
Froude No, Fr		
H_o/W		
$W = H_o/(H_o/W)$		

CHECK OUTLET VELOCITY, V_o			
H_L/H_o (*IDM* Figure 34-9D)			
$H_L = (H_L/H_o)H_o$			
$H_e = H_o - H_L$			
d_B			
$V_B = (Q/W)/d_B$			
$(H_e)_T = d_B + V_B^2/2g$			

BASIN DIMENSIONS (FT) FROM *IDM* FIGURE 34-9C

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

USBR TYPE VI BASIN CHECKLIST

Project No. ______________________________

Designer _______________________ Date ___________

Reviewer _______________________ Date ___________

DETERMINE BASIN WIDTH, W (*IDM* Figure 34-9B)	TRIAL 1	FINAL TRIAL
Equivalent Depth, d_E		
V_o (ft/s)		
$H_o = d_E + V_o^2/2g$		
Froude No, Fr		
H_o/W		
$W = H_o/(H_o/W)$		

CHECK OUTLET VELOCITY, V_o			
H_L/H_o (*IDM* Figure 34-9D)			
$H_L = (H_L/H_o)H_o$			
$H_e = H_o - H_L$			
d_B			
$V_B = (Q/W)/d_B$			
$(H_e)_T = d_B + V_B^2/2g$			

BASIN DIMENSIONS (FEET) FROM *IDM* FIGURE 34-9C

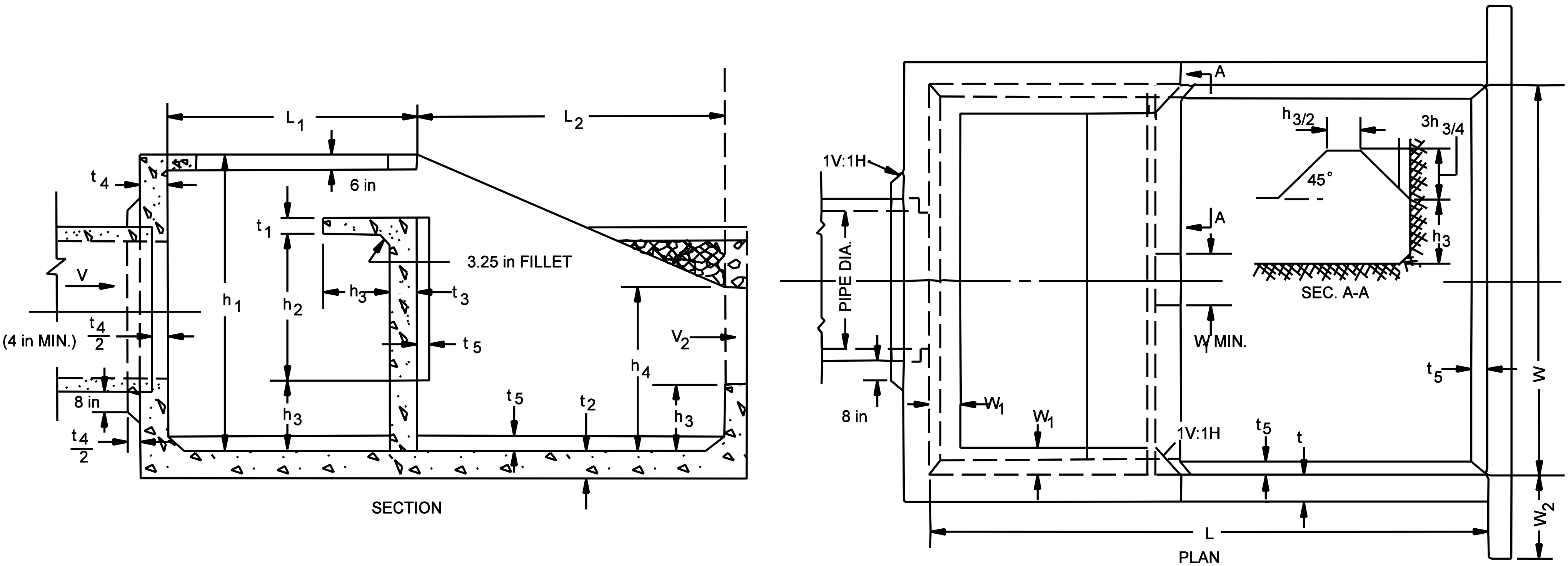
W	h_1	h_2	h_3	h_4	L	L_1	L_2
W	W_1	W_2	t_1	t_2	t_3	t_4	t_5

Click When Done

Clear Data

USBR Basin Type VI Details (Design Example)

Back To Design Example



USBR BASIN TYPE VI DETAILS
(Design Example)

FHWA CULVERT ANALYSIS, HY-8, VERSION 6.0

CURRENT DATE	CURRENT TIME	FILE NAME	FILE DATE
06-02-2003	16:13:53	ENERGY4	06-02-2003

CULVERT AND CHANNEL DATA

CULVERT NO. 1	DOWNSTREAM CHANNEL
CULVERT TYPE: 48 in. CIRCULAR	CHANNEL TYPE: IRREGULAR
CULVERT LENGTH = 308.21 ft	BOTTOM WIDTH = 5.00 ft
NO. OF BARRELS = 1.0	TAILWATER DEPTH = 2.56 ft
FLOW PER BARREL = 300 ft^3/s	TOTAL DESIGN FLOW = 300 ft^3/s
INVERT ELEVATION = 175.23 ft	BOTTOM ELEVATION = 175.27 ft
OUTLET VELOCITY = 25.1 ft/s	NORMAL VELOCITY = 16.25 ft/s
OUTLET DEPTH = 4.00 ft	

USBR TYPE 6 DISSIPATOR - FINAL DESIGN

BASIN OUTLET VELOCITY = 3.31 ft/s

W = 16.257 ft	W1 = 1.270 ft	W2 = 3.047 ft
L = 21.673 ft	L1 = 9.230 ft	L2 = 12.447 ft
H1 = 12.447 ft	H2 = 6.097 ft	H3 = 2.710 ft
H4 = 6.773 ft	T1 = 0.763 ft	T2 = 1.017 ft
T3 = 1.017 ft	T4 = 1.017 ft	T5 = 0.507 ft

USBR TYPE 6 DISSIPATOR HY-8 PROGRAM OUTPUT

Figure 34-9G

TABLE OF CONTENTS

LIST OF FIGURES

CHAPTER THIRTY-FIVE

STORAGE FACILITIES

35-1.0 INTRODUCTION

35-1.01 Overview

The traditional design of a storm-drainage system has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. As an area urbanizes, this type of design may result in major drainage and flooding problems downstream. The engineering community is now more conscious of the quality of the environment and the impact that uncontrolled increases in runoff can have on our customers. Under favorable conditions, the temporary storage of some of the storm-runoff can decrease downstream flows and often the cost of the downstream conveyance system. This Chapter provides design criteria for a detention or retention storage basin as well as procedures for performing preliminary and final sizing and reservoir-routing calculations.

35-1.02 Safety Considerations

Ponding of water for a significant period of time, at a relatively shallow depth, may introduce an additional risk factor for property damage, personal injury, or loss of life. A storage facility in a location that is easily accessible to the public should be provided with warning signs and fencing adequate to prevent entry onto the site by unauthorized persons. A storage facility located adjacent to a roadway should be provided with an adequate clear zone to minimize the accidental entry of an errant vehicle.

35-1.03 Detention and Retention

An urban stormwater storage facility is referred to as either a detention or retention facility. For this Chapter, these are defined as follows.

1. Detention. A detention facility is that designed to reduce the peak discharge and only detain runoff for a short period of time. Detention storage involves detaining or slowing runoff and then releasing it. A detention basin has a positive outlet that completely empties all runoff between storms. The excavation of a detention facility may sometimes extend below the water table or outlet level where the bottom is sealed by sedimentation.

This is referred to as a detention pond or wet-bottom detention basin. The detention pond also has a positive outlet and releases all temporary storage.

A detention facility may be designed to contain a permanent pool of water. The use of a dry-bottom detention pond is recommended for an INDOT project. Because most of the design procedures are the same for a wet- or a dry-bottom detention facility, the term storage facility will be used in this Chapter to mean either.

2. Retention. A retention facility retains runoff for an indefinite amount of time and has no positive outlet. Runoff is removed only by infiltration through a porous bottom or by evaporation. A retention pond or lake is an example of a retention facility that may be built in a development, and, may enhance the overall project. A retention basin is designed to drain into the groundwater table. This is not addressed herein.

A storage facility is most often small in terms of storage capacity and dam height, and will serve a single outfall from a watershed of a few acres. A very small facility may be contained in a parking lot or other on-site facility. Although the same principles apply to each storage facility, Section 35-10.0 more-specifically relates to a smaller installation.

If other procedures are needed for the design of a detention or retention facility, these will be specified.

35-1.04 Computer Programs

Routing calculations needed to design a storage facility, although not extremely complex, are time-consuming and repetitive. To assist with these calculations, there are many available reservoir-routing computer programs. If the watershed draining into a storage facility is greater than 2 acres, design should be based upon reservoir-routing methods which develop hydrographs for both inflow and outflow. A smaller basin may be analyzed using the storage-indication method or the Rational Method.

35-2.0 USES

35-2.01 Introduction [Rev. Jan. 2011]

The use of a storage facility for stormwater management has increased in recent years. Controlling the quantity of stormwater using a storage facility can provide the potential benefits as follows:

1. prevention or reduction of peak runoff rate increases caused by urban development;

2. mitigation of downstream drainage capacity problems;
3. reduction or elimination of the need for downstream outfall improvements; and
4. maintenance of historically low flow rates by controlled discharge from storage.
5. improvement of downstream water quality through stormwater-pollution-prevention BMP design features.

35-2.02 Objectives

The objectives for managing stormwater quantity by a storage facility are based on limiting peak runoff rates to match either or both of the values as follows:

1. historic rates for specific design conditions (i.e., post-development peak equals pre-development peak for a particular frequency of occurrence); or

2. non-hazardous discharge capacity of the downstream drainage system.

For a watershed without an adequate outfall, the total volume of runoff is critical. A storage facility is used to store the stormwater due to increases in volume and control-discharge rates.

35-3.0 SYMBOLS AND DEFINITIONS

To provide consistency within this Chapter and throughout this *Manual*, the symbols in Figure 35-3A will be used. These symbols were selected because of their wide use in technical publications. The same symbol may be 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.

35-4.0 DESIGN CRITERIA

35-4.01 General Criteria [Rev. Jan. 2011[

Storage may be developed in a depressed area in a parking lot, road embankment, freeway interchange, or a small lake, pond, or depression within an urban development. The utility of a storage facility depends on the amount of storage, its location within the system, and its operational characteristics. An analysis of such a storage facility should consist of comparing the design flow at a point or points downstream of the proposed storage site, with or without storage. Other flows in excess of the design flow that may be expected to pass through the storage facility may be required in the analysis (i.e., 100-year flood). The design criteria for a storage facility should include the following:

1. release rate;
2. storage volume;
3. grading and depth requirements;
4. outlet works;
5. location; and
6. water-quality design requirements.

35-4.02 Release Rate

At a minimum, a storage facility should be designed to detain the 50-year, post-development peak runoff and release it at the 10-year, pre-developed peak runoff rate. If applicable, it should also satisfy the more-restrictive requirements that may be imposed by a local jurisdiction. An emergency overflow capable of accommodating the 100-year discharge may be required in a facility using a dam.

35-4.03 Storage

Routing calculations must be used to demonstrate that the facility-storage volume is adequate to provide the required detention. If sedimentation during construction causes loss of detention volume, design dimensions should be restored before completion of the project. For a detention basin, all detention volume should be drained within the average period between storm events, or 72 h.

35-4.04 Grading and Depth

35-4.04(01) General

The construction of a storage facility requires excavation or placement of an earthen embankment to obtain sufficient storage volume. The embankment should be of less than 6.5 ft height. A vegetated embankment should have side slopes not steeper than 3H:1V. A riprap-protected embankment should not be steeper than 2H:1V. An excavated storage facility should not have an operating design pool depth of greater than 5 ft unless specifically approved by the Hydraulics Team.

A minimum freeboard of 1 ft above the 100-year-storm high-water elevation should be provided.

Other considerations in setting the depth 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 should also be considered in an urban area. Fencing of a basin is addressed in Section 35-14.0.

35-4.04(02) Dry-Bottom Detention

The area above the normal high-water elevation of a storage facility should be sloped toward the facility to allow drainage and to prevent standing water. Finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of a storage facility should be graded toward the outlet to prevent standing-water conditions. A low flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flow and to prevent standing-water conditions.

35-4.04(03) Wet-Bottom Detention

The maximum depth of a permanent storage facility will be determined based on site conditions and design constraints. If the facility provides a permanent pool of water, a depth sufficient to discourage growth of weeds should be considered. A depth of 6.5 ft is reasonable.

35-4.05 Outlet Works

Outlet works selected for a storage facility include a principal spillway or an emergency overflow and must be able to accomplish the design functions of the facility. Outlet works can take the form of combinations of a drop inlet, pipe, weir, or orifice. A slotted-riser pipe is discouraged because of clogging problems. A curb opening may be used for parking-lot storage. The principal spillway is intended to convey the design storm without allowing flow to enter an emergency outlet.

An orifice outlet takes the form of a restriction of 12 in. or less placed in a larger pipe. The preferred design for such an outlet consists of placing a smaller pipe on the flowline of a larger pipe. The smaller pipe will be the required size to achieve the desired detention results and is approximately 12 in. length. Grout is placed around the smaller pipe to fill the area of the larger. This type of construction provides for adequate maintenance and is more durable than a single constrictor plate.

35-5.0 GENERAL PROCEDURE

35-5.01 Data Needs

The data required to complete the storage design and routing calculations are as follows:

1. inflow hydrograph for each selected design storm;

2. stage-storage curve for the proposed storage facility (see Figure 35-5A for an example). For a large storage volume, use acre/feet, otherwise use cubic feet; and

3. stage-discharge curve for each outlet-control structure (see Figure 35-5B for an example).

Using these data, a design procedure is used to route the inflow hydrograph through the storage facility with different basin and outlet configurations until the desired outflow hydrograph is achieved. See the example problem in Section 35-8.0.

35-5.02 Stage-Storage Curve

A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir. The data for this type of curve are developed using a topographic map and one of the following formulas, the average-end area, frustum of a pyramid, or prismoidal. A storage basin may be irregular in shape to blend well with the surrounding terrain and to improve aesthetics. Therefore, the average-end-area formula is preferred as the method to be used for a non-geometric area. The double-end-area formula is expressed as follows:

$$V_{1,2} = \frac{d\left(A_1 + A_2\right)}{2} \qquad \text{(Equation 35-5.1)}$$

Where:

$V_{1,2}$ = storage volume, ft^3, between elevations 1 and 2
A_1 = surface area at elevation 1, ft^2
A_2 = surface area at elevation 2, ft^2
d = change in elevation between points 1 and 2, ft

The frustum of a pyramid is expressed as follows:

$$V = \frac{d\left[A_1 + \left(A_1 A_2\right)^{0.5} + A_2\right]}{3} \qquad \text{(Equation 35-5.2)}$$

Where:

V = volume of frustum of a pyramid, ft^3
d = change in elevation between points 1 and 2, ft
A_1 = surface area at elevation 1, ft^2
A_2 = surface area at elevation 2, ft^2

The prismoidal formula for a trapezoidal basin is expressed as follows:

$$V = LWD + \frac{D^2(L+W)}{Z} + \frac{4D^3}{3Z^2} \qquad \text{(Equation 35-5.3)}$$

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 horizontal to vertical

35-5.03 Stage-Discharge Curve

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A storage facility has two spillways: principal and emergency. The principal spillway is designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal spillway or outlet. Tailwater influences and structure losses must be considered in developing discharge curves.

The emergency spillway, when needed, is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal spillway. This spillway should be designed taking into account the potential threat to downstream life and property if the storage facility were to fail. The stage-discharge curve should take into account the discharge characteristics of both the principal and emergency spillways.

35-5.04 Procedure

The procedure for using the above data in the design of a storage facility is described below.

1. Compute inflow hydrographs for runoff from the design storm using the procedure outlined in Chapter Twenty-nine.

2. Perform preliminary calculations to evaluate detention-storage requirements for the hydrographs from Step 1 (see Section 35-7.0 for recommended methods).

3. Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used.

4. Size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage. Ascertain that tailwater effects have been considered.

5. Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using the storage-routing equations. If the routed post-development peak discharge from the 50-year design storm exceeds the pre-development 10-year peak discharge, or if the peak stage varies significantly from the estimated peak stage from Step 4, revise the estimated volume and return to Step 3.

6. Where required, consider emergency overflow from runoff due to the 100-year design storm and established freeboard requirements.

7. Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

This procedure can involve a significant number of reservoir-routing calculations to obtain the desired results.

35-5.05 Computer Procedures

A number of commercial computer software packages exist which automate a number of the steps described above. Although these programs can greatly accelerate the design process, they should be used with caution. The output from these programs should be reviewed considering sound engineering judgment. Except in modeling a drainage area of less than 2 acres, the programs must be capable of developing hydrographs for both inflow and outflow. For an area of less than 2 acres, the Rational Method is acceptable for generating the inflow hydrographs.

35-6.0 OUTLET HYDRAULICS

35-6.01 Outlets

Sharp-crested-weir flow equations for a no-end contraction, a two-end contraction, and submerged discharge conditions are provided below, followed by equations for a broad-crested weir, V-notch weir, proportional weir, orifice, or a combination of these facilities. If a culvert is used as an outlet works, the procedure described in Chapter Thirty-one should be used to develop stage-discharge data. In analyzing release rates, tailwater influences must be considered to determine the effective head on each outlet. A slotted riser-pipe outlet facility should be avoided.

35-6.02 Sharp-Crested Weir

A sharp-crested weir with no-end contractions is illustrated in Figure 35-6A. The discharge equation for this configuration is as follows (Chow, 1959):

$$Q = LH^{1.5}\left[3.27 + 0.4\left(\frac{H}{H_c}\right)\right] \qquad \text{(Equation 35-6.1)}$$

Where:
- Q = discharge, ft^3/s
- H = head above weir crest excluding velocity head, ft
- H_c = height of weir crest above channel bottom, ft
- L = horizontal weir length, ft

A sharp-crested weir with two-end contractions is illustrated in Figure 35-6B. The discharge equation for this configuration is as follows (Chow, 1959):

$$Q = (L - 0.2H)H^{1.5}\left[3.27 + 0.4\left(\frac{H}{H_c}\right)\right] \qquad \text{(Equation 35-6.2)}$$

Where the variables are the same as for Equation 35-6.4.

Figure 35-6C illustrates a sharp-crested weir and head.

A sharp-crested weir will be affected by submergence if 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 as follows (Brater and King, 1976):

$$Q_S = Q_f\left[1 - \left(\frac{H_2}{H_1}\right)^{1.5}\right]^{0.385} \qquad \text{(Equation 35-6.3)}$$

Where:
- Q_S = submergence flow, ft^3/s
- Q_f = free flow, ft^3/s
- H_1 = upstream head above crest, ft
- H_2 = downstream head above crest, ft

35-6.03 Broad-Crested Weir

The equation used for the broad-crested weir is as follows (Brater and King, 1976):

$$Q = CLH^{1.5} \qquad \text{(Equation 35-6.4)}$$

Where:
- Q = discharge, ft^3/s

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 yields the maximum C value of 1.704. For sharp corners on the broad-crested weir, a minimum C value of 1.435 should be used. Additional information on C value as a function of weir-crest breadth and head is shown in Figure 35-6D.

35-6.04 V-Notch Weir

The discharge through a V-notch weir can be calculated from the equation as follows (Brater and King, 1976):

$$Q = 2.5 \tan(\varphi/2) H^{2.5} \qquad \text{(Equation 35-6.5)}$$

Where:
Q = discharge, ft^3/s
φ = angle of V-notch, deg
H = head on apex of notch, ft

35-6.05 Proportional Weir

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 are as follows (Sandvik, 1985):

$$Q = 4.97 a^{0.5} b \left(H - \frac{a}{3} \right) \qquad \text{(Equation 35-6.6)}$$

$$\frac{x}{b} = 1 - \left(\frac{1}{3.17} \right) \left[\arctan \left(\frac{y}{a} \right)^{0.5} \right] \qquad \text{(Equation 35-6.7)}$$

Where Q = discharge, ft^3/s

Dimensions a, b, H, x, and y are shown in Figure 35-6E.

35-6.06 Orifice

A pipe smaller than 12 in. diameter may be analyzed as a submerged orifice if H/D is greater than 1.5. For square-edged entrance conditions, the formula that applies is as follows:

$$Q = 0.6A(2gH)^{0.5} \quad \text{(Equation 35-6.8)}$$

Where:
- Q = discharge, ft^3/s
- A = cross-section area of pipe, ft^2
- g = acceleration due to gravity, 32.2 ft/s^2
- D = diameter of pipe, ft
- H = head on pipe, from the center of pipe to the water surface, ft

Where the tailwater is higher than the center of the opening, the head is calculated as the difference in water-surface elevations.

35-7.0 PRELIMINARY DETENTION CALCULATIONS

35-7.01 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 35-7A.

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as follows:

$$V_S = 0.5T_i(Q_i - Q_o) \quad \text{(Equation 35-7.1)}$$

Where:
- V_S = storage-volume estimate, ft^3
- Q_i = peak inflow rate, ft^3/s
- Q_o = peak outflow rate, ft^3/s
- T_i = duration of basin inflow, s

Consistent units may be used for Equation 35-7.1.

35-7.02 Alternative Method

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained through the following regression-equation procedure (Wycoff & Singh, 1986).

1. Determine input data, including the allowable peak outflow rate, Q_o, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.

2. Calculate a preliminary estimate of the ratio V_S/V_r using the input data from Step 1 and the equation as follows:

$$\frac{V_S}{V_r} = \frac{\left[1.291\left(1-\frac{Q_o}{Q_i}\right)^{0.753}\right]}{\left[\left(\frac{tb}{tp}\right)^{0.411}\right]} \qquad \text{(Equation 35-7.2)}$$

Where:

V_S	=	volume of storage, ft^3
V_r	=	volume of runoff, ft^3
Q_o	=	outflow peak flow, ft^3/s
Q_i	=	inflow peak flow, ft^3/s
t_b	=	time base of the inflow hydrograph, h (Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak.)
t_p	=	time to peak of the inflow hydrograph, h

35-7.03 Peak-Flow Reduction

A preliminary estimate of the potential peak-flow reduction for a selected storage volume can be obtained by the following procedure.

1. Determine the following.

   a. volume of runoff, V_r
   b. peak flow rate of the inflow hydrograph, Q_i
   c. time base of the inflow hydrograph, t_b
   d. time to peak of the inflow hydrograph, t_p
   e. storage volume, V_S

2. Calculate a preliminary estimate of the potential peak-flow reduction for the selected storage volume using the equation as follows (Singh, 1976).

$$\frac{Q_o}{Q_i} = 1 - 0.712\left(\frac{V_S}{V_r}\right)^{1.328}\left(\frac{t_b}{t_p}\right)^{0.546} \qquad \text{(Equation 35-7.3)}$$

Where:
- Q_o = outflow peak flow, ft^3/s
- Q_i = inflow peak flow, ft^3/s
- V_S = volume of storage, ft^3
- V_r = volume of runoff, ft^3
- t_b = time base of the inflow hydrograph, h (Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak.)
- t_p = time to peak of the inflow hydrograph, h

3. Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak-flow reduction calculated from Step 2 to obtain the estimated peak outflow rate, Q_o, for the selected storage volume.

35-7.04 Preliminary Basin Dimensions

The following applies.

1. Plot the control-structure location on a contour map.

2. Select a desired depth of ponding for the design storm.

3. Divide the estimated storage volume needed by the desired depth to obtain the surface area required of the reservoir.

4. Based on site conditions and contours, estimate the geometric shapes required to provide the estimated reservoir surface area.

35-8.0 ROUTING CALCULATIONS

The following procedure is used to perform routing through a reservoir or storage facility (Puls Method of storage routing).

1. Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility. Example stage-storage and stage-discharge curves are shown in Figures 35-8A and 35-8B, respectively.

2. Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph ($\Delta t < T_c/5$).

3. Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of $S \pm (O_1/2)\Delta t$ versus stage. An example tabulation of storage-characteristics curve data is shown in Figure 35-8C.

4. For a given time interval, 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 (example shown in Figure 35-8D).

5. Determine the value of $S_2 + (O_2/2)\Delta t$ from the equation as follows:

$$S_2 + \frac{O_2 \Delta t}{2} = \left(S_1 - \frac{O_1 \Delta t}{2} \right) + \frac{(I_1 + I_2)\Delta t}{2} \qquad \text{(Equation 35-8.1)}$$

Where:

	S_2	=	storage volume at time 2, ft^3
	O_2	=	outflow rate at time 2, ft^3/s
	Δt	=	routing time period, s
	S_1	=	storage volume at time 1, ft^3
	O_1	=	outflow rate at time 1, ft^3/s
	I_1	=	inflow rate at time 1, ft^3/s
	I_2	=	inflow rate at time 2, ft^3/s

Other consistent units are equally appropriate.

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.

7. Determine the value of O_2 which corresponds to a stage of H_2 determined in Step 6, using the stage-discharge curve.

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.

35-9.0 EXAMPLE PROBLEM

35-9.01 Example

This example demonstrates the application of the methodology provided herein for the design of a detention-storage facility. Example inflow hydrographs and associated peak discharges for both pre- and post-development conditions are assumed to have been developed using hydrologic methods from Chapter Twenty-nine.

35-9.02 Design Discharge and Hydrographs

A storage facility is to be designed for runoff from both the 10-year and 50-year design storms. INDOT requires that the 50-year post-development peak discharge attains or does not exceed the 10-year pre-development peak discharge. Example peak discharges from the 10-year and 50-year design storm events are as follows:

1. pre-development 10-year peak discharge = 5.66 ft^3/s; and
2. post-development 50-year peak discharge = 7.08 ft^3/s.

Because the post-development 50-year peak discharge must not exceed the pre-development 10-year peak discharge, the allowable outflow discharge cannot exceed 5.66 ft^3/s.

Example runoff hydrographs are shown in Figure 35-9A. Inflow durations from the post-development hydrographs are approximately 1.2 and 1.25 h, respectively, for runoff from the 10-year and 50-year storms.

35-9.03 Preliminary Volume Calculations

Preliminary estimates of required storage volume are obtained using the simplified method outlined in Section 35-7.0. The required storage volume, V_S, to contain the difference between $Q_{50\ (POST)}$ and $Q_{10\ (PRE)}$ is computed using Equation 35-7.1.

$$V_S = 0.5T_i(Q_i - Q_o) = 0.5T_i\ (Q_{50\ (POST)} - Q_{10\ (PRE)})$$
$$\mathrm{V_S} = 0.5\ (1.25)\ (3600)\ (250 - 200) = 112500\ ft^3$$

35-9.04 Design and Routing Calculations

Stage-discharge and stage-storage characteristics of a storage facility that should provide adequate peak flow attenuation for runoff from the 50-year design storm is provided in Figure

35-9B. The storage-discharge relationship is developed by requiring the preliminary storage volume estimates of runoff for the 50-year design storm to be provided once the corresponding allowable peak discharge occurred. Discharge values are computed by solving the broad-crested-weir equation for head, H, assuming a constant discharge coefficient of 1.71, a weir length of 4.00 ft and no tailwater submergence. The capacity of the storage-relief structure is assumed to be negligible.

Storage routing is conducted for runoff from the 50-year design storm to confirm the preliminary storage-volume estimates and to establish design-water surface elevations. Routing results using the Stage-Discharge-Storage data shown in Figure 35-9B and the Storage Characteristics Curves shown in Figures 35-8A and 35-8B, and 0.1-h time steps are shown in Figure 35-9C for runoff from the 50-year design storm. The preliminary design provides adequate peak discharge attenuation.

For the routing calculations, Equation 35-8.1 should be used.

The value in Column 6 should equal that in Column 3 plus that in Column 5.

Because the routed peak discharge is lower than the maximum allowable peak discharges for the 50-year design storm event, the weir length can 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. The preliminary design provides hydraulic details only. The final design should consider site constraints such as depth of water, side slope stability and maintenance, grading to prevent standing water, and provisions for public safety.

35-10.0 DRY-BOTTOM DETENTION BASIN

35-10.01 Introduction

A dry-bottom detention basin is a depressed area that stores runoff during wet weather and is dry the rest of the time. It is popular because of its comparatively low cost; few design limitations; and ability to serve a large or small watershed.

35-10.02 Design

The following applies.

1. Quantity. The pond should be designed to provide the required detention. It should be able to safely pass a 100-year storm. It should be designed using the procedures described in Sections 35-5.4 and 35-5.5. A 100-year storm should be routed through the facility to ensure that the embankment will not be damaged or fail during the passage of the storm. To improve the efficiency of the outlet, it may be necessary to include an antivortex device.

2. Outlet. The outlet for a dry basin can be designed in a variety of configurations. However, INDOT discourages the use of a riser pipe. A larger flow is accommodated by an emergency spillway.

35-10.03 Other Considerations

The sideslopes of the pond should not be steeper than 3H:1V to facilitate maintenance activities. The floor of the pond should be sloped at 2% toward the outlet to prevent ponding. The maximum operating pool depth should not exceed 6.5 ft without approval of the INDOT Hydraulics Team.

Routine maintenance activities include an annual inspection, preferably during wet weather, and mowing, as needed.

35-11.0 WET-BOTTOM DETENTION BASIN

35-11.01 Introduction

A wet-bottom detention basin is similar to a dry-bottom detention basin in that it detains stormwater. However, it is different in that it retains a permanent pool during dry weather. A wet-bottom detention basin is more expensive than a dry-bottom detention basin.

35-11.02 Design

The following applies.

1. Quantity. A wet-bottom detention basin should provide the required detention and be able to safely pass a 100-year storm.

2. Outlet. The outlet can be designed in a variety of configurations. However, INDOT discourages the use of a riser pipe.

35-11.03 Other Considerations

The sideslopes of the pond should not be steeper than 3H:1V both above and below water for both safety and maintenance. Normal pool depth should not exceed 5 ft. The maximum operating-pool depth should not exceed 8.25 ft.

Routine maintenance includes annual inspections, preferably during wet weather, and mowing as needed.

35-11.04 Illustration

See Figure 35-11A for an illustration of a wet-bottom detention basin.

35-12.0 LANDLOCKED RETENTION [REV. JAN. 2011]

A watershed area that drains to a central depression without a positive outlet is typical of an area including karst topography. It can be evaluated using a mass-flow-routing procedure to estimate flood elevations. Although the procedure is straightforward, the evaluation of basin outflow is a complex hydrogeologic phenomenon that requires quality field measurements and a thorough understanding of local conditions. Because the outflow rate for a flooded condition is difficult to calculate, field measurements are desirable. See the AASHTO *Model Drainage Manual* for more-detailed information.

Directing a significant amount of treated or untreated stormwater toward a karst feature is an allowable practice if the following can be satisfied.

1. The project karst evaluator has previously indicated to the project engineer that no adverse reactions are expected.
2. The practice is consistent with the karst MOU of 1993.

35-13.0 CONSTRUCTION AND MAINTENANCE CONSIDERATIONS

To ensure acceptable performance and function, a storage facility that requires extensive maintenance is discouraged. The maintenance problems that are typical of an urban detention facility are as follows:

1. weed growth;
2. grass and vegetation maintenance;
3. bank deterioration;

4. standing water or soggy surfaces;
5. mosquito control;
6. blockage of outlet structures;
7. litter accumulation; and
8. maintenance of fences and perimeter plantings.

A proper design should focus on the elimination or reduction of maintenance requirements by addressing the potential for problems as follows.

1. Both weed growth and grass maintenance may be addressed by constructing sideslopes that can be maintained using available power-driven equipment, such as a tractor mower.

2. Bank deterioration can be controlled with protective lining or by limiting bank slopes.

3. Standing water or soggy surfaces may be eliminated by sloping the basin bottom toward the outlet, or constructing a low-flow pilot channel across the basin bottom from the inlet to the outlet.

4. Once the problems listed above are addressed, mosquito control will not be a major problem.

5. An outlet structure should be selected to minimize the possibility of blockage. Small pipes tend to block easily and should be avoided).

6. The facility should be located for easy access where the maintenance associated with litter and damage to fences or perimeter plantings can be conducted regularly.

35-14.0 PROTECTIVE TREATMENT

Safety considerations include reducing the chance of drowning by fencing the basin, reducing the maximum depth, or including ledges or mild slopes to prevent a person from falling in and facilitate his or her escape from the basin. Protective treatment may be required to prevent entry to a facility that poses a hazard to children, and, to a lesser extent, all persons. Fences and signs will be required for a detention area where the following conditions exist.

1. Rapid stage increases make escape difficult.
2. Water depth either exceeds 3.3 ft for more than 24 h or the area is permanently wet.
3. Sideslopes equal or exceed 1.5H:1V.

Where a storage facility is located near a roadway, the road should be provided with an adequate clear zone. The maximum operating-pool depth will be limited to 6.5 ft unless approved by the Hydraulics Team.

35-15.0 REFERENCES

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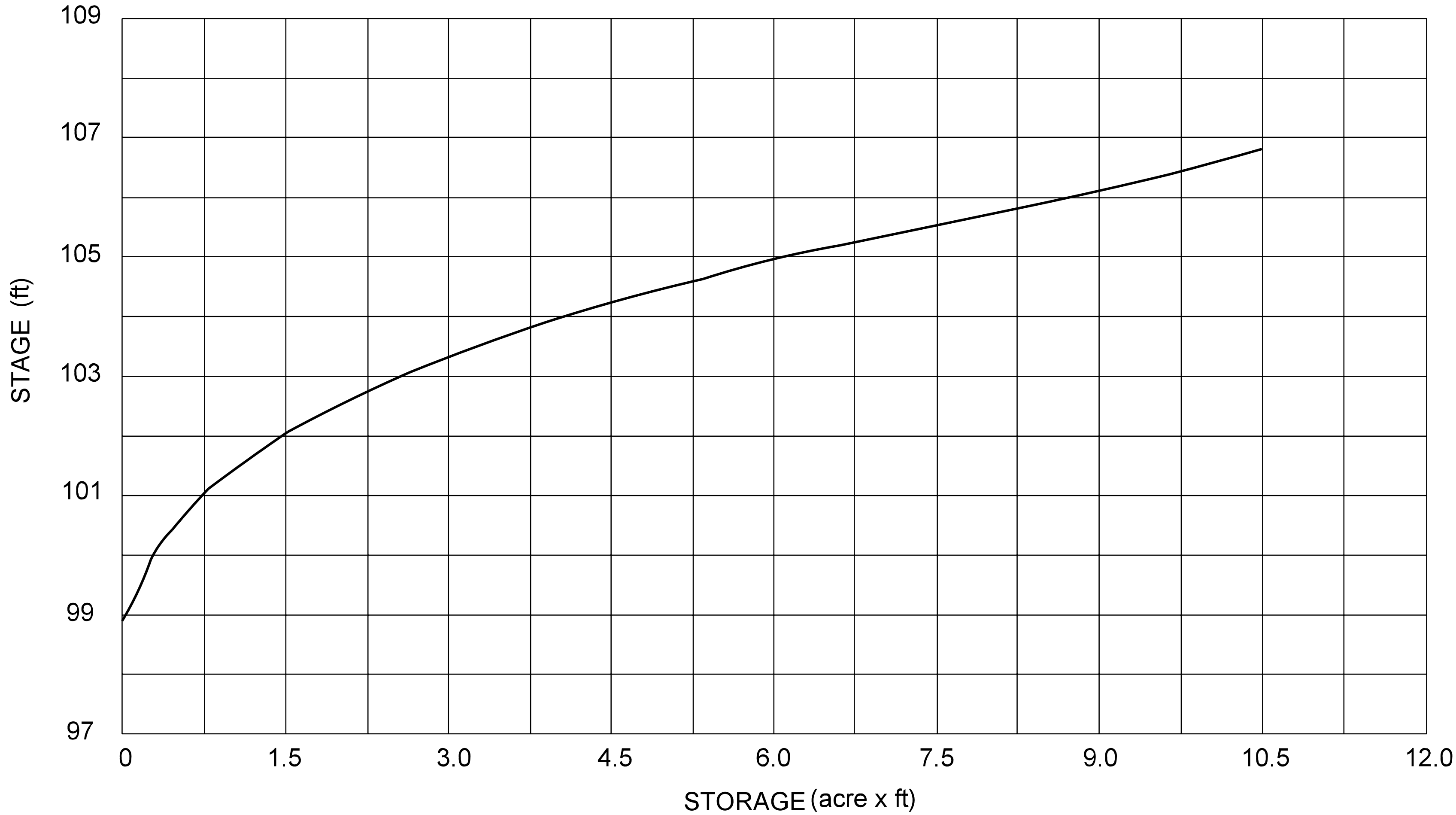
Symbol	Definition	Unit
A	Cross-sectional or surface area	ft^2
C	Weir coefficient	(none)
d	Change in elevation	ft
D	Depth of basin; Diameter of pipe	ft
f	Infiltration rate	in./h
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_c	Height of weir crest above channel bottom	ft
I	Inflow rate for storage computations	ft^3/s
L	Length	ft
O	Outflow rate	in./h
Q	Flow rate	in./h
S	Storage volume	ac-ft
t	Routing time period	S
t_b	Time based on hydrograph	H
T_i	Duration of basin inflow	H
t_p	Time to peak	H
V_S	Storage volume	ft^3
W	Width of basin	ft
z	Sideslope factor	(none)

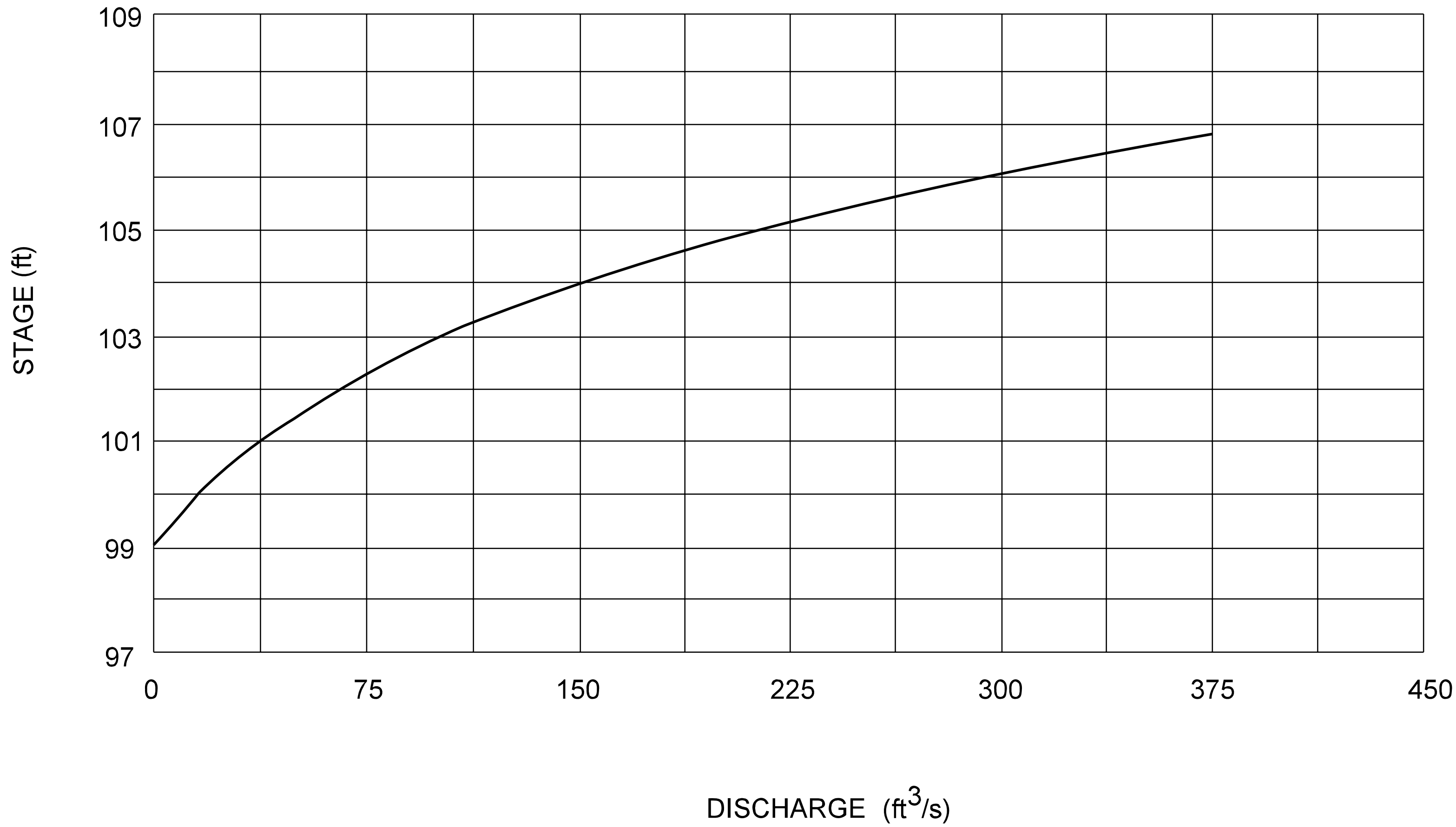
SYMBOLS, DEFINITIONS, AND UNITS

Figure 35-3A



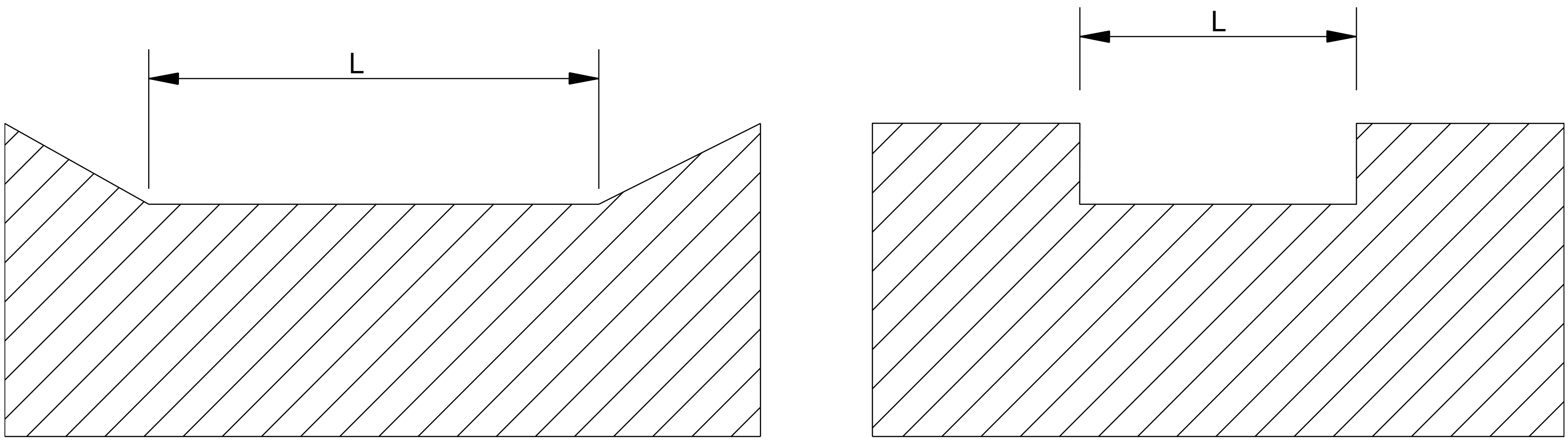
EXAMPLE STAGE-STORAGE CURVE

Figure 35-5A



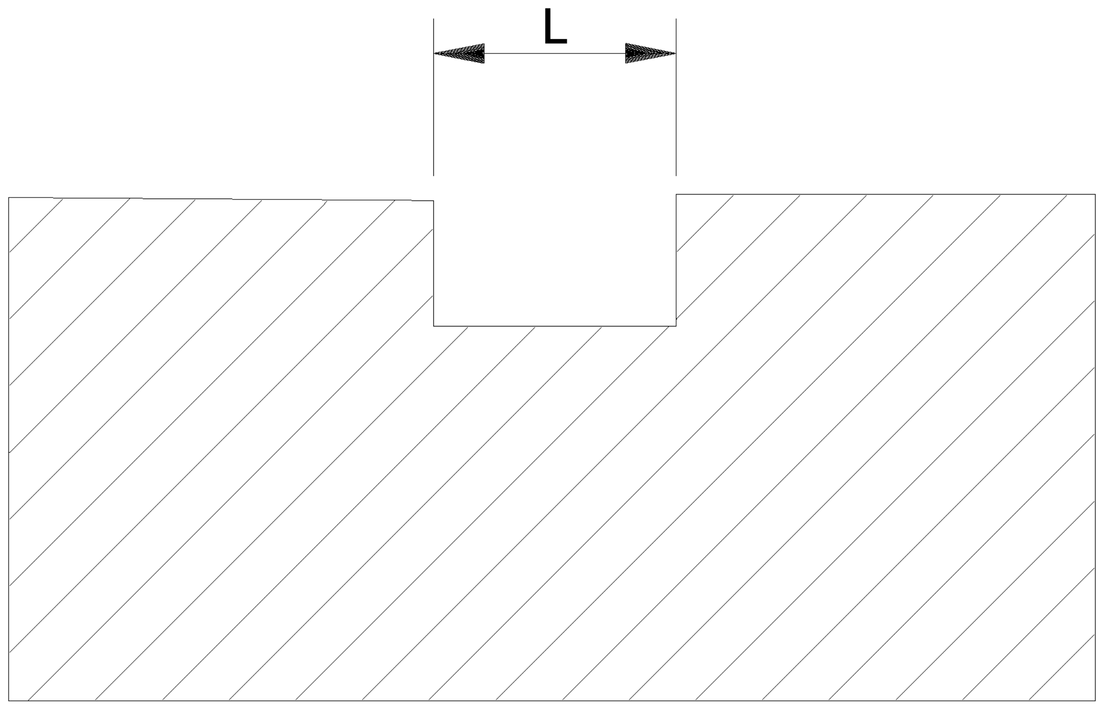
EXAMPLE STAGE-DISCHARGE CURVE

Figure 35-5B



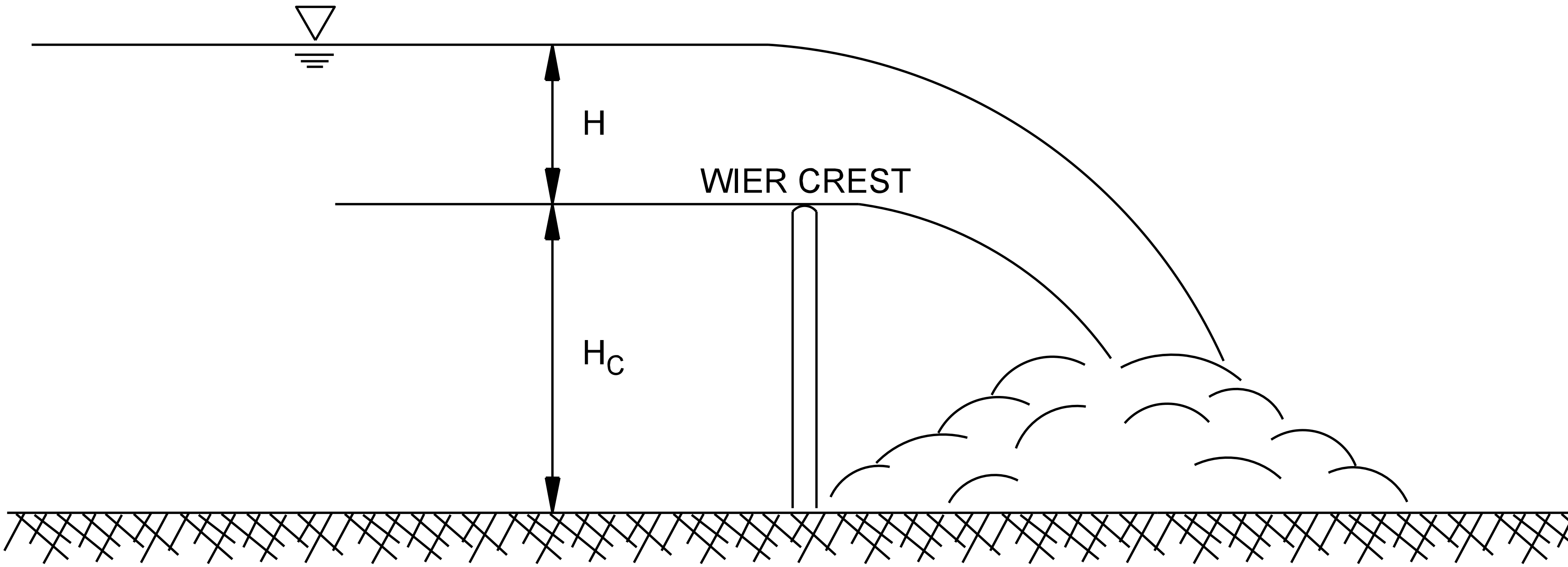
SHARP-CRESTED WEIR
(No-End Contractions)

Figure 35-6A



SHARP-CRESTED WEIR
(Two-End Contractions)

Figure 35-6B



SHARP-CRESTED WEIR AND HEAD

Figure 35-6C

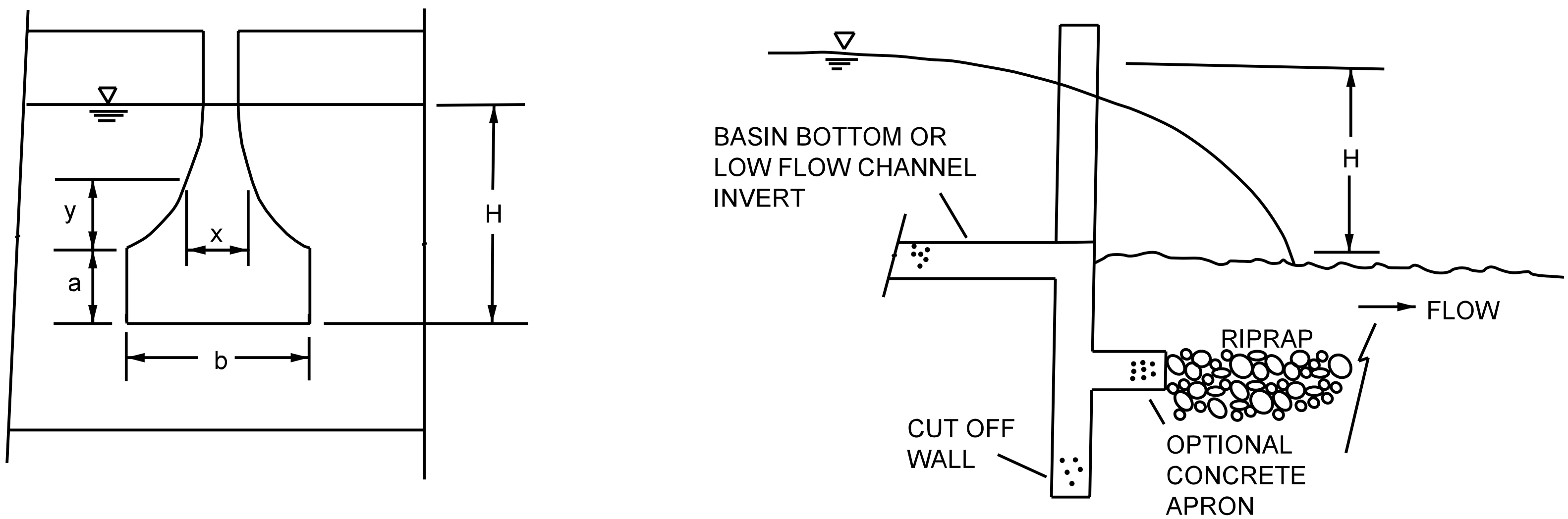
Measured Head, H^1	Breadth of the Crest of Weir (ft)										
(ft)	0.50	0.77	1.00	1.53	2.03	2.53	3.03	4.07	5.07	10.17	15.23
0.20	5.17	5.07	4.97	4.83	4.67	4.57	4.50	4.37	4.30	4.57	4.93
0.40	5.37	5.17	5.00	4.87	4.80	4.80	4.73	4.67	4.60	4.70	4.97
0.60	5.67	5.33	5.07	4.87	4.80	4.80	4.93	4.97	4.97	4.97	4.97
0.80	6.07	5.60	5.23	4.93	4.80	4.80	4.90	4.93	4.93	4.97	4.87
1.00	6.10	5.77	5.50	5.07	4.90	4.87	4.87	4.90	4.93	4.93	4.83
1.23	6.10	5.90	5.67	5.27	4.97	4.87	4.87	4.90	4.90	4.97	4.87
1.43	6.10	6.00	5.90	5.37	5.10	4.93	4.87	4.87	4.87	4.90	4.87
1.63	6.10	6.07	6.03	5.63	5.33	5.07	4.93	4.90	4.87	4.87	4.83
1.83	6.10	6.10	6.10	5.63	5.30	5.03	4.93	4.90	4.87	4.87	4.83
2.03	6.10	6.10	6.07	5.57	5.23	5.07	4.17	4.93	4.87	4.87	4.83
2.53	6.10	6.10	6.10	6.03	5.63	5.33	5.17	5.00	4.90	4.87	4.83
3.03	6.10	6.10	6.10	6.10	5.90	5.60	5.37	5.03	4.90	4.87	4.83
3.57	6.10	6.10	6.10	6.10	6.10	5.87	5.47	5.07	4.93	4.87	4.83
4.07	6.10	6.10	6.10	6.10	6.10	6.10	5.63	5.13	4.97	4.87	4.83
4.57	6.10	6.10	6.10	6.10	6.10	6.10	6.10	5.30	5.03	4.87	4.83
5.07	6.10	6.10	6.10	6.10	6.10	6.10	6.10	5.63	5.13	4.87	4.83
5.60	6.10	6.10	6.10	6.10	6.10	6.10	6.10	6.10	5.30	4.87	4.83

[1]*Measured at least 2.5H upstream of the weir.*

Reference: Brater and King (1976).

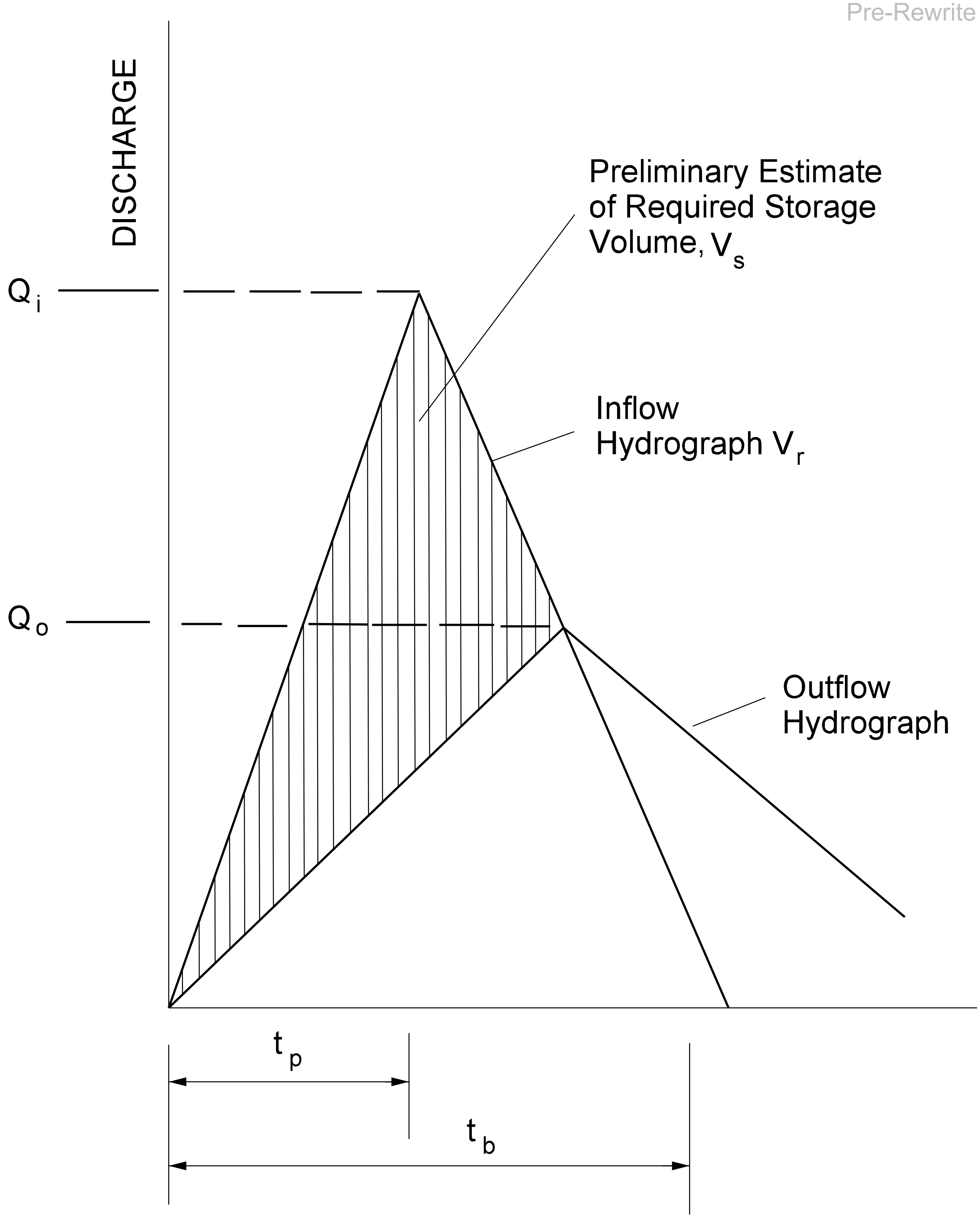
BROAD-CRESTED WEIR COEFFICIENT *C* VALUE AS A FUNCTION OF WEIR CREST BREADTH AND HEAD WEIR CREST BREADTH (ft)

Figure 35-6D



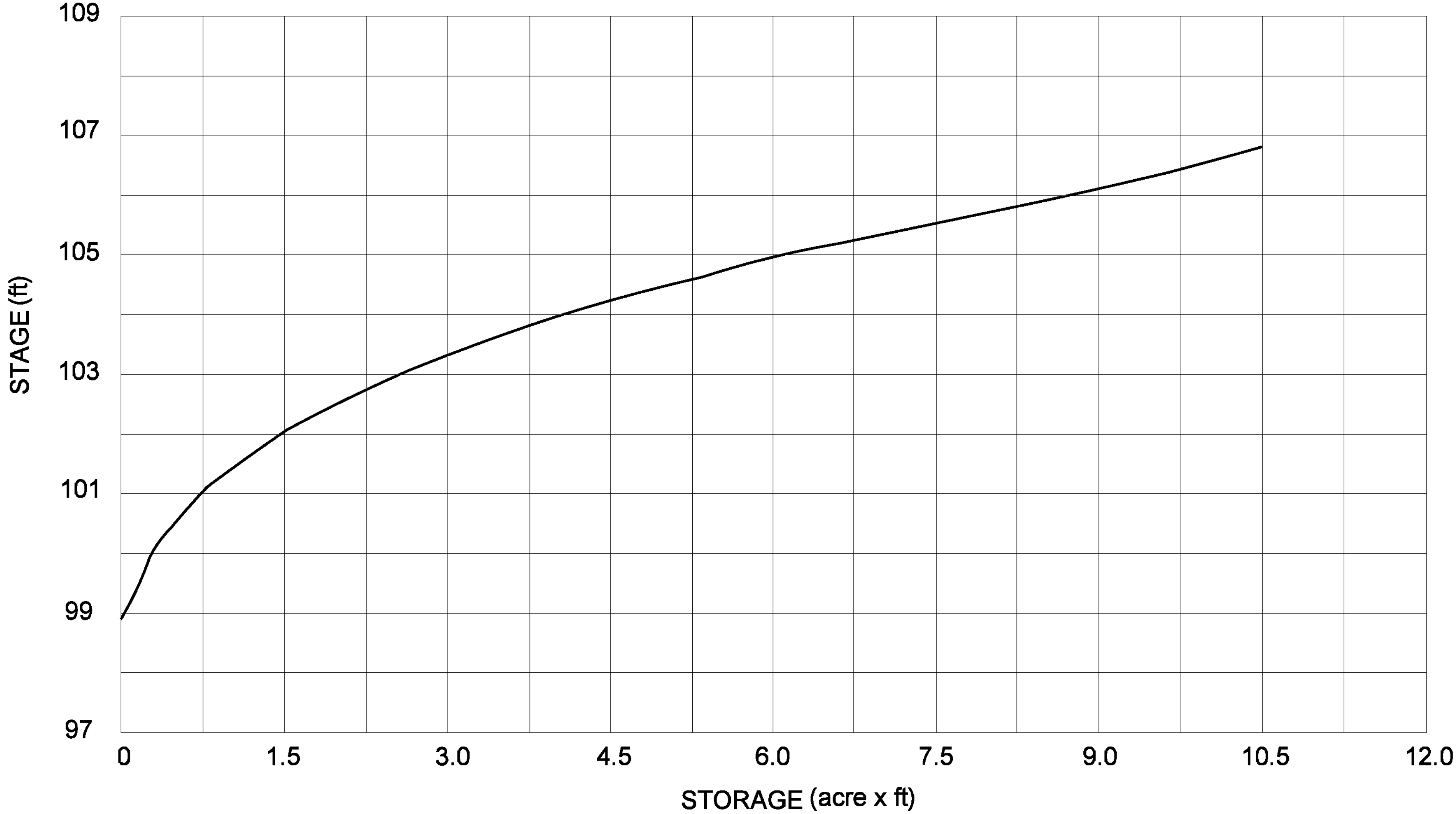
PROPORTIONAL WEIR DIMENSIONS

Figure 35-6E



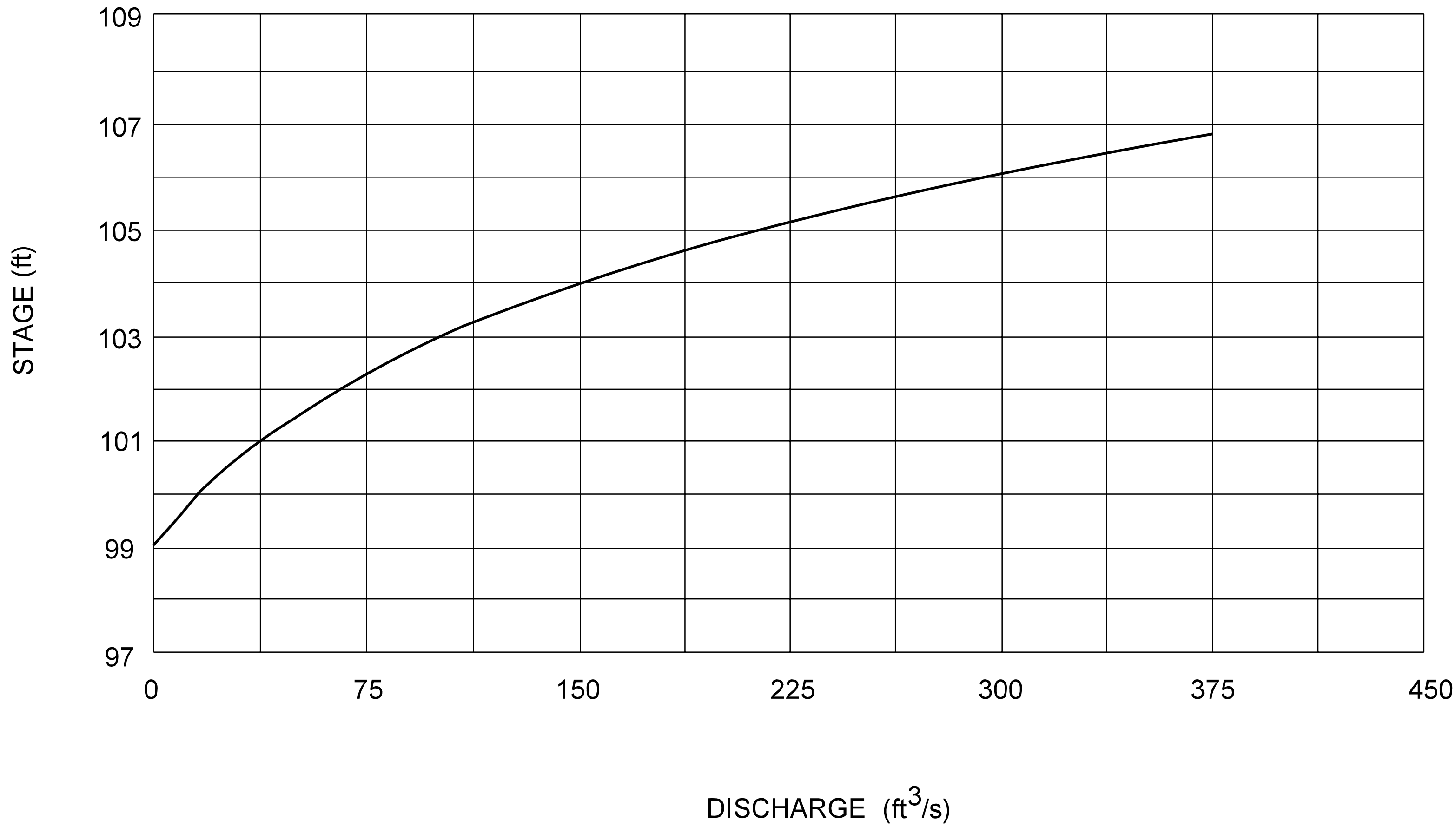
TRIANGULAR SHAPED HYDROGRAPHS
(For Preliminary Estimate Of Required Storage Volume)

Figure 35-7A



EXAMPLE STAGE-STORAGE CURVE

Figure 35-8A



EXAMPLE STAGE-DISCHARGE CURVE

Figure 35-8B

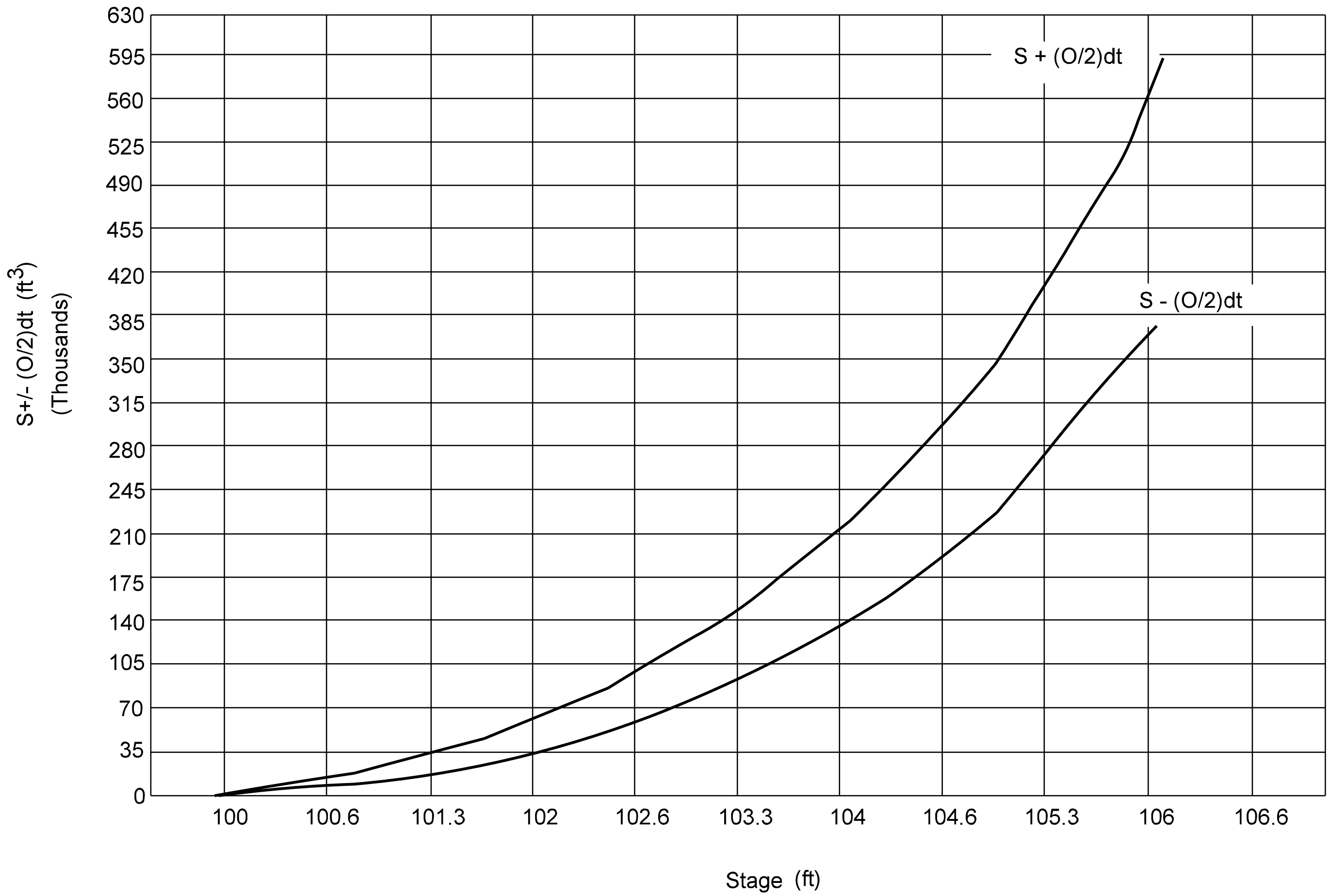
(1) Stage (ft)	(2) Storage [1] (ac-ft)	(3) Discharge [2] (ft^3/s)	(4) $S - (O/2)\Delta t$ (ac-ft)	(5) $S + (O/2)\Delta t$ (ac-ft)
98.4	0.05	0	0.05	0.05
99.3	0.3	15	0.20	0.40
100.0	0.8	35	0.56	1.04
100.9	1.6	63	1.17	2.03
101.7	2.8	95	2.15	3.45
102.5	4.4	143	3.41	5.39
103.4	6.6	200	5.22	7.98
104.2	10.0	275	8.11	11.89

[1] *Obtained from the Stage-Storage Curve.*
[2] *Obtained from the Stage-Discharge Curve.*

Note: t = 10 min = 0.167 h

STORAGE CHARACTERISTICS

Figure 35-8C



STORAGE CHARACTERISTIC CURVE

Figure 35-8D

Pre-Development Runoff		Post-Development Runoff
Time (h)	10-yr Storm (ft^3/s)	50-yr Storm (ft^3/s)
0	0	0
0.1	24	50
0.2	81	178
0.3	170	250 > 200
0.4	200	165
0.5	150	90
0.6	95	50
0.7	61	30
0.8	40	16
0.9	28	9
1	18	5
1.1	15	3
1.2	13	1

EXAMPLE RUNOFF HYDROGRAPHS

Figure 35-9A

(1)	(2)	(3)	(4)	(5)
Stage (ft)	Q (ft^3/s)	(ac-ft)	(ac-ft)	(ac-ft)
0.0	0	0.00	0.00	0.00
0.9	10	0.26	0.22	0.89
1.4	20	0.42	0.33	1.91
1.8	30	0.56	0.43	2.65
2.2	40	0.69	0.53	3.24
2.5	50	0.81	0.61	4.12
2.9	60	0.93	0.68	5.00
3.2	70	1.05	0.76	5.88
3.5	80	1.17	0.84	6.75
3.7	90	1.28	0.89	7.78
4.0	100	1.40	0.98	8.51
4.5	120	1.63	1.13	9.98
4.8	130	1.75	1.21	10.86
5.0	140	1.87	1.29	11.74
5.3	150	1.98	1.36	12.47
5.5	160	2.10	1.43	13.50
5.7	170	2.22	1.51	14.38
6.0	180	2.34	1.59	14.96
6.4	200	2.58	1.76	16.61
6.8	220	2.83	1.92	18.19
7.0	230	2.95	2.00	19.07
7.4	250	3.21	2.19	20.54

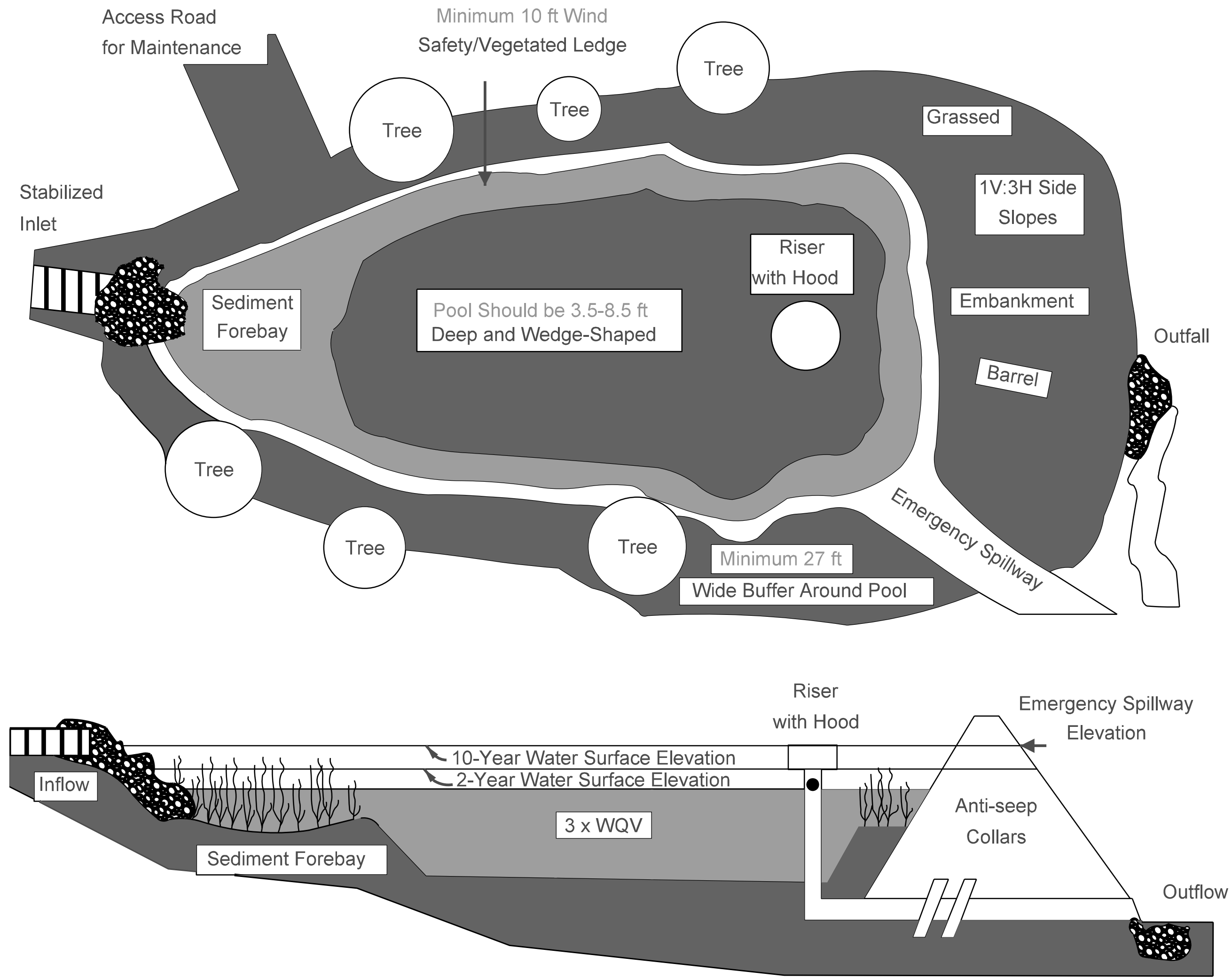
STAGE-DISCHARGE-STORAGE DATA

Figure 35-9B

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time (h)	Inflow (ft^3/s)	$[(I_1 + I_2)]/2$ (ac-ft)	H_1 (ft)	$S_1 - (O_1/2)\Delta t$ (ac-ft) (6) – (3)	$S_2 + (O_2/2)\Delta t$ (ac-ft) (3) + (5)	H_2 (ft)	Outflow (ft^3/s)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	50	0.21	0.21	0.00	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 < 200 OK
0.5	90	1.05	5.80	1.30	2.35	4.00	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	0.00	0.90	0.22	0.22	0.60	6

STORAGE ROUTING FOR THE 50-YEAR STORM

Figure 35-9C



WET POND
(After Schueler, 1987

Figure 35-11A

TABLE OF CONTENTS

LIST OF FIGURES

Figure Title

CHAPTER THIRTY-SIX

PAVEMENT/STORM DRAINAGE SYSTEMS

36-1.0 OVERVIEW

36-1.01 Introduction

This Chapter provides guidance on storm-drain design and analysis. The quality of the final in-place system reflects the attention accorded to every aspect of the design as well as that accorded to the construction and maintenance of the facility. The aspects of storm-drain design such as system planning, pavement drainage, gutter-flow calculations, inlet spacing, pipe sizing, and hydraulic grade line calculations are discussed herein.

The design of a drainage system must address the needs of the traveling public as well as those of the local community through which it passes. The drainage system for a roadway traversing an urbanized region is more complex than for a roadway traversing a sparsely-settled rural area. This is due to the following:

1. wide roadway sections and flat grades, both in the longitudinal and transverse directions, shallow water courses, absence of side channels;

2. more costly property damage which may occur from ponding of water or from flow of water through a built-up area; and

3. the roadway section must carry traffic but also act as a channel to convey the water to a disposal point. Unless proper precautions are taken, this flow of water along the roadway will interfere with or possibly halt the passage of highway traffic.

36-1.02 Inadequate Drainage

The most serious effects of an inadequate roadway drainage system are as follows:

1. damage to surrounding or adjacent property resulting from water overflowing the roadway curbs and entering such property;

2. risk and delay to traffic caused by excessive ponding in sags or excessive spread along the roadway; and

3. weakening of base and subgrade due to saturation from frequent ponding of long duration

36-2.0 POLICY AND GUIDELINES

36-2.01 Introduction [Rev. Jan. 2011]

A highway storm-drainage facility collects stormwater runoff and conveys it through the roadway right of way to adequately drain the roadway and minimize the potential for flooding and erosion to properties adjacent to the right of way. A storm-drainage facility consist of curbs, gutters, storm drains, side ditches or open channels (as appropriate), or culverts. The placement and hydraulic capacity of a storm-drainage facility should be designed to consider damage to adjacent property and to secure as low a degree of risk of traffic interruption due to flooding as is consistent with the importance of the road, the design traffic service requirements, and available funds. Stormwater-pollution-prevention requirements should be considered and addressed in the design process of each storm-drainage system.

The following is a summary of the policies for pavement drainage system design and analysis.

36-2.02 Bridge Deck

A zero gradient, sag vertical curve, or superelevation transition with a flat pavement section should be avoided on a bridge. The desirable longitudinal grade for bridge-deck drainage is 0.5% or steeper, especially for new construction. A flatter grade will be tolerated where it is not physically or economically desirable to satisfy this criterion. A bridge may not require drainage facilities. The quantity and quality of runoff should be maintained as required by applicable stormwater regulations. See Chapter Thirty-three for additional information.

36-2.03 Curbs, Inlets, and Turnouts

Curbs, inlets, or turnouts are used where runoff from the pavement can erode fill slopes, or where reduction of the right of way needed for shoulders, side ditches, or open channels, etc., is desirable. Where storm drains are necessary, the pavement section should be curbed.

36-2.04 Design Frequency

The design flood frequency for roadway drainage is related to the allowable water spread on the pavement and design speed. This design criterion is discussed in Section 36-7.0.

36-2.05 Detention Storage

Reduction of peak flow can be achieved through the storage of runoff in a detention basin, storm drainage pipe, swale, side ditch, open channel, or other detention storage facility. Stormwater can then be released to the downstream conveyance facility at a reduced flow rate. The concept should be considered where existing downstream conveyance facilities are inadequate to handle peak flow rates from a highway storm-drainage facility. A developer may not be permitted to increase runoff over existing conditions, thus necessitating a detention storage facility. Additional benefits include the reduction of downstream pipe sizes and the improvement of water quality by removing sediment or pollutants. For additional information, see Chapter Thirty-five.

36-2.06 Gutter-Flow Calculations

Gutter-flow calculations are necessary to relate the quantity of flow to the spread of water on a shoulder, parking lane, or pavement section. A composite gutter section has a greater hydraulic capacity for a normal cross slope than a uniform gutter section, and is therefore preferred. See Section 36-8.0 for additional information and procedures.

36-2.07 Hydrology

The Rational Method is the most common method in use for the design of a storm drain if the momentary peak flow rate is desired. Its use should be limited to a system with a drainage area of 200 acres or less. A minimum time of concentration of 5 min is acceptable. The Rational method is described in Chapter Twenty-nine.

36-2.08 Inlets

The term refers to each type, such as a grate inlet, curb inlet, or slotted inlet. A drainage inlet is sized and located to limit the spread of water on traffic lanes to a tolerable width for the design storm in accordance with the design criteria specified in Section 36-7.0. The width of water spread on the pavement at a sag should not be substantially greater than the width of spread encountered on a continuous grades.

A grate inlet or depression-of-curb-opening inlet should be located outside the through traffic lanes to minimize the shifting of a vehicle attempting to avoid it. A grate inlet should be bicycle-safe if used on a roadway that allows bicycle travel. If a grate inlet is used at a sag location, a double curved vane grate should be utilized to compensate for plugging that can occur.

Where significant ponding can occur, such as at an underpass or sag vertical curve in a depressed section, flanking inlets should be placed on each side of the inlet at the low point in the sag. See Section 36-9.03 for a discussion on the location of inlets.

36-2.09 Manholes

The maximum spacing of access structures whether manholes, junction boxes, or inlets should be approximately 400 ft. Figure 36-11B is useful in determining the relationship between manhole diameter, maximum pipe size, and deflection angle as defined in Figure 36-13B.

36-2.10 Roadside or Median Ditch

A large amount of runoff should be intercepted before it reaches the highway to minimize the deposition of sediment or other debris on the roadway, and to reduce the amount of water which must be carried in the gutter section. A median area or inside shoulder must be sloped to prevent runoff from the median area from flowing across the pavement. A surface channel should have adequate capacity for the design runoff and should be located and shaped to not present a traffic hazard. Where permitted by the design velocity, a channel should have a vegetative lining. An appropriate lining may be necessary where vegetation will not control erosion. See Chapter Thirty for detailed hydraulic information on a channel.

36-2.11 Storm Drain

A storm drain is defined as a closed-conduit system. It consists of that portion of the storm-drainage system that receives runoff from inlets and conveys the runoff to a point where it is then discharged into a side ditch or water body. At least one end is connected to a manhole, inlet, catch basin, or similar structure. A pipe which is connected to an inlet located in a paved median, grassed median, or lawn area is considered a storm-drain structure. A storm drain should have adequate capacity so that it can accommodate runoff that enters the system. It should be designed considering future development if appropriate. The storm-drain system for a sag vertical curve should have a higher level of flood protection to decrease the depth of potential ponding on the roadway or bridge. Where feasible, the storm drain should be designed to avoid existing utilities. The storm-drain outfall should be designed to ensure that the potential for erosion is minimized. The drainage-system design should be coordinated with the proposed staging of a large construction project to maintain an outlet throughout the construction project.

A storm-sewer trunk line should be located behind the curb or, if not practical, under the roadway without being located in the wheel path.

Design the main and all laterals as a system. The system must not operate under pressure for the design storm. The hydraulic grade line must not exceed a manhole, catch basin, or inlet rim elevation for the check storm.

The placement and capacity should be consistent with local stormwater management plans. A minimum pipe size of 12 in. with a minimum velocity of 2.5 ft/s is desirable to prevent sedimentation from occurring in the pipe.

36-2.12 System Planning

System planning prior to commencing the design of a storm-drain system is essential. The basic requirements are discussed in Section 36-5.0, and include the general design approach, type of data required, information on initiating a cooperative agreement with a municipality, the importance of a preliminary sketch, and some special considerations.

36-2.13 Storm-Drainage Agreement Policy

A storm-drainage agreement is required if a new or reconstructed INDOT drainage facility is designed to accommodate stormwater from a sewer controlled by a local public agency (LPA). This is applicable regardless of whether the shared drainage facility is constructed within or outside of INDOT right of way.

Where INDOT constructs a drainage facility outside the limits of the right of way to provide adequate drainage for a highway, I.C. 8-23-6-2 allows INDOT to assess a proportionate share of the cost of constructing the drainage facility outside the right of way to beneficiaries of the drainage structure. Therefore, a municipality or other beneficiary that connects to an INDOT drainage structure outside the limits of the right of way can be assessed a share of the cost of the drainage structure in proportion to the amount of drainage discharged. The proportionate share is calculated as follows:

$$A_B = C_F\left(\frac{Q_{OR}}{Q_T}\right)$$

Where: A_B = Amount of assessment to beneficiary
C_F = Cost of drainage facility
Q_{OR} = Discharge from storm sewer draining from outside INDOT R/W
Q_T = Total discharge of drainage facility

The remainder of the cost will be paid by INDOT.

By common law, INDOT also has the authority to seek a contribution from the LPA if stormwater from outside the INDOT right of way discharges into a drainage facility within the INDOT right of way. For example, if a municipality wishes to make a direct discharge into an INDOT trunkline storm drain, INDOT's policy will be to request a storm-drainage agreement for the trunkline sewer construction. The proportionate share will also be determined from Equation 36-2.1. If the discharge is in the form of sheet flow onto INDOT right of way, INDOT will not seek a contribution from the municipality involved. INDOT is not legally required to accept sheet-flow runoff from outside the right of way, but will do so as a matter of public policy.

If a particular situation involving sheet flow onto right of way is sufficiently significant to warrant a storm-drainage agreement, the LPA should agree to the necessary local contribution as a condition for initiating the State highway improvement. Such an agreement cannot be forced upon an LPA, but must be pre-arranged through negotiations between the LPA and the Planning Division or the Office of Environmental Services' Environmental Policy Team. However, this may occur as late as the design phase.

A situation may arise if INDOT storm-sewer construction results in a request for stormwater detention or a county assessment for the reconstruction of a regulated drain. See Section 28-3.07. If the situation also involves INDOT conveying city or town stormwater, INDOT should seek a storm-sewer cost-sharing contribution from the city or town. The procedure for determining the appropriate contribution by the city or town will be as described above. INDOT cannot site I.C. 8-23-6-2 as authority to pass on a portion of a county drainage assessment to the city or town. Only a county drainage board has the authority to levy a drainage assessment on a municipality or private-property owner if a regulated drain is involved.

A county drainage assessment does not require a formal agreement to be legally binding on INDOT. However, if an assessment includes a monetary contribution which relieves INDOT from providing stormwater detention mandated by the county, the conditions of the assessment should be formalized in a storm-drainage agreement.

The need for a storm-drainage agreement should be identified during the preliminary-plans development. Detailed information necessary for the preparation of the formal agreement should be coordinated with the municipality prior to INDOT design approval. The preliminary cost estimate of the trunkline sewer and the exact ratio to be used in determining the municipality's share should be verbally agreed to with the municipality. The ratio may be based on the sewer's cross-sectional area if the discharge of the municipality's storm sewer cannot be reasonably determined. The municipality should be notified in writing of the approximate cost of its share so that it can arrange financing.

After design approval, the formal storm-drainage agreement will be written to bind the LPA and the State. The Legal Services Division will prepare this document. The agreement must be signed by all parties concerned before the project may be scheduled for a letting.

36-2.14 Compatibility of Drainage Structure and Casting

Figure 36-2A shows which casting may be used with a given type of catch basin, inlet, or manhole. The information shown in the figure is complementary to that shown on the related INDOT *Standard Drawings*. In developing a drainage plan, the designer should refer to the figure to ascertain structure and casting compatibility. If the designer desires to use a structure-casting combination other than that permitted in the figure, he or she should contact the Production Management Division's Hydraulics Team.

36-3.0 SYMBOLS AND DEFINITIONS

To provide consistency within this Chapter and throughout this *Manual*, the symbols shown in Figure 36-3A will be used. The symbols were selected because of their wide use in storm-drainage publications.

36-4.0 CONCEPT DEFINITIONS

The following are the concepts which should be considered in a storm-drainage analysis or design. The concepts will be used throughout this Chapter in addressing the different aspects of storm drainage analysis.

1. Check Storm or Check Event. The use of a less-frequent event, such as a 50-year storm, to assess a hazard at a critical location where water can pond to an appreciable depth.

2. Combination Inlet. A drainage inlet composed of a curb-opening inlet and a grate inlet.

3. Crown or Soffit. The inside top of a pipe.

4. Culvert. A drainage structure which extends through the embankment on both ends for the purpose of conveying surface water under a roadway. It may have one or two inlets connected to it to convey drainage from the median area.

5. Curb Opening. A drainage inlet consisting of an opening in the roadway curb.

6. Drop Inlet. A drainage inlet with a horizontal or nearly-horizontal opening.

7. Equivalent Cross Slope. An imaginary straight cross slope having conveyance capacity equal to that of the given compound cross slope.

8. Flanking Inlet. An inlet placed upstream and on each side of another inlet at the low point in a sag vertical curve. The purpose of a flanking inlet is to intercept debris as the slope decreases and to act in relief of the inlet at the low point.

9. Flow. The quantity of water which is flowing.

10. Frontal Flow. The portion of the flow which passes over the upstream side of a grate.

11. Grate Inlet. A drainage inlet composed of a grate in the roadway section or at the roadside in a low point, swale, or channel.

12. Grate Perimeter. The sum of the lengths of all sides of a grate, except that a side adjacent to a curb is not considered a part of the perimeter in weir-flow computations.

13. Gutter. That portion of the roadway section adjacent to the curb which is utilized to convey stormwater runoff. A composite gutter section consists of the section immediately adjacent to the curb, of 24 in. width at a cross-slope of 2.5%, and the parking lane, shoulder, or pavement at a cross slope of 2.0%. A uniform gutter section has one constant cross slope. See Section 36-8.0 for additional information.

14. Hydraulic Grade Line. The locus of elevations to which water can rise in successive piezometer tubes if the tubes were installed along a pipe run (pressure head plus elevation head).

15. Inlet Efficiency. The ratio of flow intercepted by an inlet to total flow in the gutter.

16. Invert. The inside bottom of a pipe.

17. Lateral Line or Lead. This has inlets connected to it but has no other storm drains connected to it. It is 15 in. diameter or less and is a tributary to the trunkline.

18. Pressure Head. The height of a column of water that can exert a unit pressure equal to the pressure of the water.

19. Runby, Bypass, or Carryover. Flow which bypasses an inlet on grade and is carried in the street or channel to the next inlet downgrade. An inlet can be designed to allow a certain amount of runby for one design storm, or a larger or smaller amount for another storm.

20. Sag Point or Major Sag Point. A low point in a vertical curve. A major sag point is a low point that can overflow only if water can pond to a depth of 1.5 ft or more.

21. Scupper. A vertical hole through a bridge deck for the purpose of deck drainage. This can also be a horizontal opening in a curb or barrier.

22. Side-Flow Interception. Flow which is intercepted along the side of a grate inlet, as opposed to frontal interception.

23. 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. The types in use are slotted drain pipe and slotted vane-drain pipe. A slotted-drain inlet is used in conjunction with a single-grate inlet for cleanout access.

24. Storm Drain. Each pipe that is installed in conjunction with at least one inlet, catch basin, or manhole. A grassed-median inlet, lawn inlet, lawn catch basin, or pipe catch basin is considered to be a storm drain.

25. Splash-Over. The portion of frontal flow at a grate which skips or splashes over the grate and is not intercepted.

26. Spread. The width of stormwater flow in the gutter measured laterally from the roadway curb.

27. Trunkline or Main. The main storm-drain line. A lateral line may be connected at an inlet structure or manhole.

28. Velocity Head. Velocity head is a quantity proportional to the kinetic energy of flowing water expressed as a height or head of water ($V^2/2g$).

36-5.0 SYSTEM PLANNING

36-5.01 Introduction

The design of a 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.

36-5.02 General Design Approach

The design of a storm-drain system is a process which evolves as a project develops. The primary ingredients to this process are listed below in the sequence by which they may be carried out. All of the individuals who contribute to this process cannot be listed, because the list varies for each project. However, the Hydraulics Engineer’s role is as follows:

1. data collection (see Section 36-5.03);
2. coordination with other agencies (Section 36-5.04);
3. preliminary sketch (Section 36-5.05);
4. inlet locations and spacing (Sections 36-9.0 and 36-10.0);
5. plan layout of storm-drain system as follows:
   a. locate main outfall;
   b. determine direction of flow;
   c. locate existing utilities;
   d. locate connecting mains; and
   e. locate manholes;
6. size the pipes (Section 36-12.0);
7. review hydraulic grade line (Section 36-13.0);
8. prepare the plan; and
9. provide documentation (Chapter Twenty-eight).

36-5.03 Required Data [Rev. Jan. 2011]

The designer should be familiar with land-use patterns, the nature of the physical development of the area 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 an existing outfall location. There should be an understanding of the nature of the outfall because it has a significant influence on the storm-drainage system. Water-quality requirements should always be considered, particularly in an environmentally-sensitive area.

Actual surveys of these and other features are the most reliable means of gathering the required data. Photogrammetric mapping has become a method of obtaining the large amounts of data required for drainage design, particularly for a busy urban roadway with attendant urban development. Existing topographic maps, available from the U. S. Geological Survey, the Natural Resources Conservation Service, municipalities, county governments, or private developers, are also valuable sources of the kind of data needed for a proper storm-drainage design. Governmental planning agencies should be consulted regarding plans for the area in question. The physical characteristics of a rapidly-growing urban area to be served by a storm-drainage system may change drastically in a short time. The designer must anticipate these changes and consider them in the storm-drainage design.

Comprehensive stormwater-management plans or floodplain ordinances should be reviewed if they are available.

36-5.04 Preliminary Sketch

A preliminary sketch or schematic, showing the basic components of the intended design, is useful. Such a sketch should indicate watershed areas and land use, existing drainage patterns, plan and profile of the roadway, street or drive layout with respect to the project roadway, underground utility locations and elevations, locations of proposed retaining walls, bridge abutments and piers, logical inlet and manhole locations, preliminary lateral and trunkline layouts, and a definition of the outfall location and characteristics. The sketch should be reviewed with the traffic-staging plans and soils recommendations for an area which is incompatible with required construction staging. With the sketch or schematic, the designer is able to proceed with the detailed process of storm-drainage design calculations, adjustments, and refinements.

Unless the proposed system is simple and small, the designer should not ignore a preliminary plan as described above. Upon completion of the design, documentation of the overall plan is facilitated by the preliminary schematic.

36-5.05 Special Considerations

Consideration and planning should be directed toward avoidance of utilities and deep cuts. Traffic may be maintained or a temporary bypass may be constructed, and temporary drainage may be provided for during the construction phase. Further consideration should be given to the actual trunkline layout and its constructability. The proposed location of the storm drain may interfere with existing utilities or disrupt traffic. A trunkline may be required on each side of the roadway with few laterals, or only a single trunkline may be required. Such features are a function of economy but may be controlled by other physical features.

Pipe size should not be decreased in a downstream direction regardless of the available pipe gradient because of potential plugging with debris.

36-6.0 PAVEMENT DRAINAGE

36-6.01 Introduction

Roadway features considered during gutter, inlet, and pavement drainage calculations include the following:

1. longitudinal and cross slopes;
2. curb and gutter sections;
3. roadside and median ditches; and
4. bridge deck.

The pavement width, cross slope, and profile grade control the time required for stormwater to drain to the gutter section. The gutter cross section and longitudinal slope control the quantity of flow which can be carried in the gutter section.

36-6.02 Roadway Longitudinal Slope

A minimum longitudinal grade should be considered for a curbed pavement because of the spread of stormwater against the curb. A flat grade on an uncurbed pavement can also lead to a spread problem if vegetation is allowed to build up along the pavement edge.

The desirable minimum gutter grade for a curbed pavement is 0.5%, and the desirable minimum is 0.3%. A minimum grade in curbed sections can be maintained in flat terrain by rolling the longitudinal-gutter profile. On an uncurbed roadway, the minimum longitudinal grade is 0%.

To provide adequate drainage in a sag vertical curve, a minimum slope of 0.3% should be maintained within 50 ft 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 170. Although ponding is not a problem at a crest vertical curve, a similar minimum grade should be provided to facilitate drainage.

36-6.03 Cross Slope

The selection of pavement cross slope is a compromise between motorist comfort and safety (i.e., flatter cross slope) and drainage (i.e., steeper cross slope). Chapters Forty-five and Fifty-three provide INDOT criteria on cross slope for the traveled way, shoulder, and curb offset. The slope will vary according to the following:

1. facility of two-lanes, or facility of 4 or more lanes;
2. urban or rural location;
3. functional classification of the facility;
4. new construction or reconstruction, or 3R work; and
5. curbed or uncurbed facility.

See Chapters Forty-five and Fifty-three to determine the applicable roadway cross slope.

36-6.04 Pavement Texture

The pavement texture should be considered for roadway surface drainage. Although the designer will have little control over the selection of the pavement type or its texture, the pavement texture does have an impact on the buildup of water depth on the pavement during a storm. A high level of macrotexture provides a channel for water to escape from the tire-pavement interface and thus reduces the potential for hydroplaning.

A high level of macrotexture may be achieved by tining a new portland-cement-concrete pavement surface while it is still in the plastic state. Retexturing of an existing portland-cement-concrete surface can be accomplished through pavement grooving or cold milling. Longitudinal or transverse grooving is effective in achieving macrotexture in concrete pavement. Transverse grooving aids in surface runoff resulting in less wet pavement time. A combination of longitudinal and transverse grooving provides the most adequate drainage for high-speed conditions.

36-6.05 Curb and Gutter

A curb at the outside edge of a pavement is common for a low-speed, urban highway facility. It contains the surface runoff within the roadway and away from adjacent properties, prevents erosion, provides pavement delineation, and enables the orderly development of property adjacent to the roadway. See Section 45-1.0 for a discussion on curb types and usage.

A curb and gutter forms a triangular channel that can be an efficient hydraulic conveyance facility to convey runoff of a lesser magnitude than the design flow without interruption to traffic. If 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 of concern for curb-and-gutter flow. This width should be limited as discussed in Section 36-7.0.

Where practical, it is desirable to intercept runoff from a cut slope or other area draining toward the roadway before it reaches it, to minimize the deposition of sediment or other debris on the roadway and to reduce the amount of water which must be carried in the gutter section. A shallow swale section at the edge of the roadway pavement or shoulder offers advantages over a curbed section where curbs are not needed for traffic control. The advantages include a lesser hazard to traffic than a near-vertical curb, and hydraulic capacity that is not dependent on spread on the pavement. A swale section without a curb is appropriate where a curb has been used to prevent water from eroding a fill slope.

36-6.06 Roadside or Median Ditch

A roadside ditch is used with an uncurbed roadway section to convey runoff from the highway pavement and from areas which drain toward the highway. Due to right-of-way limitations, a roadside ditch should not be used on an urban arterial. It can be used in a cut section, depressed section, or other location where sufficient right of way is available, and drives or intersections are infrequent. Where practical, the flow from an area draining toward a curbed highway pavement should be intercepted in the ditch as appropriate.

A median area or inside shoulder should be sloped to a center swale to prevent drainage from the median area from flowing across the pavement. This should be considered for a high-speed facility, or for one with more than two lanes of traffic in each direction.

Chapter Thirty discusses the hydraulic design of a channel.

36-6.07 Bridge Deck

Drainage of a bridge deck is similar to that for a curbed roadway section. However, it can be 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 requires a constant cross slope. Because of the difficulties in providing and maintaining an adequate deck-drainage system, gutter flow from the roadway should be intercepted before it reaches a bridge. Runoff should be collected by inlets, although gutter turnouts may be used for a minor flow. Runoff should also be handled in compliance with applicable stormwater-quality regulations. Deck drainage can be carried several spans to the bridge end for disposal.

The gutter spread should be checked to ensure compliance with the design criteria described in Section 36-7.0. A flat grade or sag vertical curve is not allowed on a bridge on a new alignment. The desirable longitudinal slope for bridge-deck drainage is 0.5% or steeper. A flatter grade will be tolerated where it is not physically or economically desirable to satisfy the above criteria.

A bridge deck may not require drainage structures. To determine the length of deck permitted without drainage structures and without exceeding the allowable spread, see Chapter Thirty-three.

36-6.08 Shoulder Gutter or Curb

A shoulder gutter or sloping curb may be appropriate to protect a fill slope from erosion caused by water from the roadway pavement. It should be considered for a 2:1 fill slope higher than 20 ft. It should also be considered for a 3:1 fill slopes higher than 20 ft if the roadway grade is steeper than 2%. Where permanent vegetation cannot be established, the height criterion should be reduced to 10 ft regardless of the grade. Inspection of the existing and proposed site conditions and contact with maintenance and construction personnel should be made by the designer to determine if vegetation will survive. An erosion-control blanket can be effective to facilitate the establishment of vegetation.

A shoulder gutter or curb, or a riprap turnout should be utilized at a bridge end where concentrated flow from the bridge deck would otherwise flow down the fill slope. The section of gutter should be long enough to include the transitions. A shoulder gutter or riprap turnout is not required on the high side of a superelevated section or adjacent to a barrier wall on a high fill.

36-6.09 Median or Median Barrier

A median is used to separate opposing lanes of traffic on a divided highway. It is preferable to slope a median area or inside shoulder to a center depression to prevent stormwater in the median area from flowing across the traveled way. Where a median barrier is used, or on a horizontal curve with associated superelevations, it is necessary to provide inlets and connecting storm drains to collect the water which accumulates against the barrier. A slotted drain adjacent to the median barrier or weep holes in the barrier can also be used for this purpose.

36-6.10 Impact Attenuator

The location of an impact-attenuator system should be reviewed to determine the need for a drainage structure. It is necessary to have 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 where superelevation or other grade separation occurs, a grate inlet or a slotted drain may be needed to prevent water from flowing through the clear opening and crossing the highway lanes or ramp lanes. A curb, curb-type structure, or swale cannot be used to direct water across the clear opening because vehicular vaulting can occur once the attenuator system is impacted.

36-7.0 STRUCTURE-SIZING PROCESS

The following is a summary of the hydraulic processes for sizing a storm-drain system or slotted-drain inlet.

36-7.01 Storm-Drain System

36-7.01(01) Design Frequency and Spread

The design-storm frequency for pavement drainage should be consistent with the frequency selected for other components of the storm-drain system. For pavement drainage, the design frequency must include both the recurrence interval of the rainfall and the allowable spread of water in the gutter. See Figure 36-7A for INDOT practices.

The factor that governs how much water can be tolerated in the curb-and-gutter section and on the adjacent roadway is water spread. Water is allowed to spread onto the roadway area within tolerable limits because it is not economically feasible to limit it within a narrow gutter width.

The spread should be held to the specified width for the design frequency. For a storm of greater magnitude, the spread can be allowed to utilize most of the pavement as an open channel. For a curb-and-gutter section with 4 or more lanes, or a gutter-section roadway with no parking, it is not practical to avoid travel-lane flooding where the longitudinal grade is 0.2% to 1%. However, flooding should not extend beyond the lane adjacent to the gutter or shoulder for design conditions. INDOT design criteria for allowable water spread are shown in Figure 36-7A.

The median-inlet spacing for an Interstate route or other divided highway is also based on an allowable-spread width. Runoff collected by inlets in a grass or paved median must not encroach beyond the inside traveled way edge for the storm frequency shown in Figure 36-7A.

36-7.01(02) Inlet Spacing

Curb-inlet spacing must be in accordance with accepted engineering practice. The designer must contact the Hydraulics Team if the intended calculation method is acceptable. Gutter flow that bypasses curb inlets installed on grade must be accounted for at a downstream structure. Flanking inlets should be provided at a sag location to mitigate ponding problems resulting from grate clogging.

After calculating the required spacing, actual inlet locations must be determined. Section 36-10.0 provides the Department's hydraulic calculations for inlet spacing. Section 36-9.03 provides criteria for inlet locations independent of hydraulic calculations. Each curb inlet and its associated lateral line must be included in the system modeling required for the design- and check-storm evaluation discussed below.

1. Design Storm. Each storm-drain structure must be designed so that Q_{10} passes through each structure via gravity. See Section 36-12.0.

2. Check Storm. The storm-drain network must accommodate the Q_{50} storm event. The system may operate under pressure, but the Hydraulic Grade Line (HGL) must remain below the rim elevation at each system manhole, inlet, catch basin, or similar structure. See Section 36-13.0.

The design process for a storm-drain structure does not require two sets of hydraulic calculations, because each pipe material acceptable for use as a storm drain has a smooth-interior designation. Therefore, computer modeling or hand calculations for storm-drain pipe sizing can be based on a Manning's *n* value of 0.012.

36-7.01(03) Pipe Size, Cover, and Velocity

The minimum pipe size that can be used for a storm drain structure is 12 in. dia. or 1.11 ft^2. The cover provided over a storm-drain structure must be at least 1 ft and not greater than 100 ft. The minimum full-flow velocity for a storm drain structure is 2.5 ft/s, and the recommended maximum velocity is 6.5 ft/s. A storm-drain outlet structure also requires an energy dissipator to mitigate potential erosion. The dissipator riprap-gradation requirements are identical to those outlined for a culvert structure. See Chapter Thirty-four. Contact the Hydraulics Team for additional instructions if the required riprap gradation is prohibited due to clear zone or other issues.

If a satisfactory pipe type cannot be identified for a storm-drain structure, the only acceptable specialty-structure type is a precast-reinforced concrete box section. If a suitable precast reinforced-concrete box section size cannot be determined, contact the Hydraulics Team for additional instructions.

36-7.02 Slotted-Drain Pipe or Slotted-Vane-Drain Pipe

The design requirements for this structure type depend on the structure application. See Sections 36-9.02 and 36-10.0 for a discussion on the hydraulic design of a slotted-drain inlet. The following provides the applications and the associated design requirements.

1. Superelevated Traveled-Way-Edge Installation (Slotted-Drain Pipe). If installed adjacent to the edge of a superelevated section, the slotted-drain pipe sizing will be based on a 50-year storm frequency for an Interstate facility, or a 10-year storm frequency for another type of facility. The pipe sizing must be in accordance with accepted practices described in recognized engineering publications. See Section 36-10.06.

2. Gutter Installation at Sag Curb Inlet (Slotted-Drain Pipe). The design-storm requirement for this installation is identical to that for a storm drain. The length and size of pipe required must be determined in accordance with accepted practices described in recognized engineering publications. See Section 36-10.05.

3. Storm-Drain Structure. A slotted-drain pipe or slotted-vane-drain pipe installed as a component of a storm-drain system must adequately intercept sheet flow and also accommodate all upstream runoff collected by the storm-drain system. The structure is first

sized in accordance with the storm-drain sizing-procedure outlined in Section 36-7.01, except that Manning's n = 0.024 for slotted-drain pipe. The pipe size obtained from the process described above must be checked for adequacy for interception of sheet flow. The sheet-flow-interception design-storm frequency will be Q_{50} for an Interstate facility, or Q_{10} for another type of facility.

4. Culvert Structure. The sizing of a slotted-drain pipe with corrugated-interior designation or a slotted-vane-drain pipe with smooth-interior designation for a culvert application is also a two-step process. The structure is first sized as a culvert in accordance with the requirements for culvert sizing (see Chapter Thirty-one). After the appropriate culvert size is determined, it is necessary to verify whether the structure is adequate for intercepting sheet flow at the site.

If the required slotted-drain pipe or slotted-vane-drain pipe size exceeds the maximum size shown on the INDOT *Standard Drawings*, contact the Hydraulics Team for additional instructions.

36-7.03 Pipe Extension

The sizing of a pipe extension for a storm-drain structure is as follows.

1. Match Existing Pipe Size and Interior Designation. If practical, the pipe extension should be the same size and material as the existing pipe. However, at this stage, it is only necessary to identify the required interior designation for the extension.

2. Perform Appropriate Hydraulic Analysis. The hydraulic analysis must verify that all storm-drain design criteria described above are satisfied.

If the extended structure satisfies all of the required design criteria, the structure-sizing process is complete. If the extended structure does not satisfy the required design criteria, the designer must reevaluate whether the existing structure can be replaced with a new structure. If it is not practical to replace the existing pipe because of the construction method, traffic maintenance, or other concern, contact the Hydraulic Team for further instructions.

36-7.04 Sanitary Sewer and Water Utility Coordination

Coordination with each utility should begin as soon as possible once it is determined that the proposed construction will impact existing utility facilities. For an INDOT-route project, the coordination will be administered through the Utilities Team. For a project not on an INDOT route, the designer should contact each affected utility as soon as possible.

Preliminary inlet spacing and trunkline design determinations should be incorporated into the Preliminary Field Check Plans, as required for early coordination with the utility companies.

Final storm-drain design determinations should be incorporated into the Hearing Plans so that final utility coordination can begin upon design approval.

If it is determined that utility relocation work will be included in the contract, the designer must verify that all elements of the utility construction are included in the contract documents. For example, the INDOT *Standard Specifications* do not include material or testing requirements for sanitary-sewer or water-main pipe. Therefore, if construction of these facilities is required, the designer is responsible for including all applicable requirements in the contract via special provisions. If the utility has specific casting, manhole, or other facility requirements that differ from those included in the INDOT *Standard Specifications* or *Standard Drawings*, these requirements must be included in the contract via plan details or special provisions.

See Chapter Ten for more information on utility accommodation.

36-8.0 GUTTER-FLOW CALCULATIONS

36-8.01 Introduction

Gutter-flow calculations are necessary to relate the quantity of flow, *Q*, in a curbed channel to the spread of water on a shoulder, parking lane, or pavement section. Equations can be utilized to solve for a uniform cross-slope channel, composite gutter section, or V-shaped gutter section. Figure 36-8D can also be used to solve for a composite-gutter section. A computer program, such as the FHWA HEC 12 program, can be used to solve for this, or to determine inlet capacity. A composite gutter section has a greater hydraulic capacity for normal cross slopes than a uniform gutter section, and is therefore preferred. The following provides example problems for each gutter section.

36-8.02 Manning's *n* For Pavement

Figure 36-8A provides the value of Manning's *n* for a street or pavement gutter.

36-8.03 Uniform-Cross-Slope Procedure

Gutter capacity for a uniform cross slope can be determined from the equation as follows:

$$Q = \frac{0.56 S_x^{\,1.67} S^{0.5} T^{2.67}}{n} \qquad \text{(Equation 36-8.1)}$$

Where:

Q = flow in the gutter (ft^3/s)
S_X = cross slope
S = longitudinal slope
T = water spread, ft
n = Manning's n (see Figure 36-8A)

If the gutter geometrics are known, Q or T can be determined if one of these is known. Figure 36-8B illustrates the parameters shown in Equation 36-8.1.

36-8.04 Composite-Gutter-Section Procedure

To solve for composite gutter flow, use Equation 36-8.1, Equation 36-8.2, Equation 36-8.3 and Figure 36-8C as illustrated in the following procedure. Figure 36-8C can be used to determine the flow in a gutter section with width, W, less than the total spread, T. These calculations are used for evaluating a composite gutter section or frontal flow for a grate inlet.

1. Condition 1. Find spread, given flow.

   a. 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.

      Example: $S = 0.01$; $S_X = 0.02$; $S_W = 0.06$; $W = 2.0$ ft; $n = 0.016$; $Q = 2.0$ ft^3/s. Try $Q_S = 0.706$ ft^3/s.

   b. Calculate the gutter flow, Q_W, in W, using the equation as follows:

      $$Q_W = Q - Q_S \qquad \text{(Equation 36-8.2)}$$

      Therefore, $Q_W = 2.0 - 0.706 = 1.294$ ft^3/s

   c. Calculate the ratios Q_W/Q and S_W/S_X and use Figure 36-8C to find an appropriate value of W/T:

      $Q_W/Q = 1.294/2.0 = 0.65$. $S_W/S_X = 0.06/0.02 = 3$.
      From Figure 36-8C, $W/T = 0.27$.

d. Calculate the spread, T, by dividing the depressed-section width, W, by the value of W/T from Step 1.c., as follows:

$$T = 2.0/0.27 = 7.41 \text{ ft}$$

e. Find the spread above the depressed section, T_S, by subtracting W from the value of T obtained in Step 1.d., as follows: $T_S = 7.41 - 2.0 = 5.41$ ft

f. Use the value of T_S from Step 1.e., Manning's n, S, and S_X to find the actual value of Q_S from Equation 36-8.1 as follows: $Q_S = 0.494 \text{ ft}^3/\text{s}$

g. Compare the value of Q_S from Step 1.f. to the trial value from Step 1.a. If the values are not comparable, select a new value of Q_S and return to Step 1.a.

Compare 0.494 to 0.706. It is too low. Try $Q_S = 0.812$. Therefore $2.0 - 0.812 = 1.188$, and $1.188/2.0 = 0.6$. From Figure 36-8C, $W/T = 0.23$. Therefore $T = 2.0/0.23 = 8.70$ ft, and $T_S = 8.70 - 2.0 = 6.7$ ft. From Equation 36-8.1, $Q_S = 0.812 \text{ ft}^3/\text{s}$. Therefore OK.

Answer: Spread, $T = 8.70$ ft.

2. Condition 2. Find gutter flow, given spread.

a. 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.

Example: Allowable spread, $T = 10.17$ ft; $W = 2.0$ ft; $T_S = 10.17 - 2.0 = 8.17$ ft; $S_X = 0.04$; $S = 0.005$ ft/ft; $S_W = 0.06$; $n = 0.016$; $d = 0.f3$ ft

b. Use Equation 36-8.1 to determine the capacity of the gutter section above the depressed section, Q_S. Use the procedure for uniform cross slope, Condition 2, substituting T_S for T. From Equation 36-8.1, $Q_S = 3 \text{ ft}^3/\text{s}$.

c. Calculate the ratios W/T and S_W/S_X and, from Figure 36-8C, find the appropriate value of E_O, the ratio of Q_W/Q. $W/T = 2.0/10.17 = 0.2$. $S_W/S_X = 0.06/0.04 = 1.5$. From Figure 36-8C, $E_O = 0.46$.

d. Calculate the total gutter flow using the equation as follows:

$$Q = \frac{Q_S}{1 - E_O} \qquad \text{(Equation 36-8.3)}$$

Where: Q = gutter flow rate, ft^3/s

Q_S = flow capacity of the gutter section above the depressed section, ft^3/s

E_O = ratio of frontal flow to total gutter flow, Q_W/Q

Therefore, $Q = 3.00/(1 - 0.46) = 5.55\ ft^3/s$.

e. Calculate the gutter-flow width, *W*, using Equation 36-8.2 as follows:

$$Q_W = Q - Q_S = 5.55 - 3.00 = 2.55\ ft^3/s$$

Figure 36-8D can also be used to calculate the flow in a composite-gutter section.

36-8.05 V-Type Gutter Section Procedure

Equation 36-8.1 can also be used to solve for a V-type channel. 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 a median barrier. It assumes the effective flow is confined to the V-section with spread, T_1.

1. Condition 1. Given flow, *Q*, find spread, *T*.

a. 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*. Example: $S = 0.01$, $S_{X1} = 0.06$, $S_{X2} = 0.04$, $S_{X3} = 0.015$, $n = 0.016$, $Q = 2.0\ ft^3/s$, shoulder = 6.1 ft. See Figure 36-8E.

b. Calculate S_X as follows:

$$S_X = \frac{S_{X1}S_{X2}}{S_{X1} + S_{X2}} = \frac{(0.06)(0.04)}{0.06 + 0.04} = 0.024$$

c. Solve for T_1 using Equation 36-8.1. T_1 is a hypothetical width that is correct if it is contained within S_{X1} and S_{X2}. From Equation 36-8.1, $T_1 = 8.5$ ft. However, because the shoulder width of 6.1 ft is less than 8.5 ft, S_{X2} is 0.04 and the pavement cross slope S_{X3} is 0.015, *T* will actually be greater than 8.5 ft. Therefore, 8.5 – 2.0 = 6.5 ft, > 4.0 ft. Therefore, the spread is greater than 8.5 ft.

d. To find the actual spread, solve for depth at points B and C.

Point B: 6.5 ft at 0.04 = 0.26 ft. Point C: 0.26 ft – (4.0 ft at 0.04) = 0.10 ft

e. Solve for the spread on the pavement. Pavement cross slope = 0.015.

$T_{0.015}$ = 0.10/0.015 = 6.67 ft

f. Find the actual total spread, T. T = 6.10 + 6.67 = 12.77 ft

2. Condition 2. Given spread, T, find flow, Q.

a. Determine input parameters, longitudinal slope, S, cross slope, $S_X = S_{X1}S_{X2}/(S_{X1} + S_{X2})$, Manning's n, and allowable spread. Example: n = 0.016, S = 0.015, S_{X1} = 0.06, S_{X2} = 0.04, T = 6.10 ft

b. Calculate S_X as follows:

$$S_X = \frac{S_{X1}S_{X2}}{S_{X1} + S_{X2}} = \frac{(0.06)(0.04)}{0.06 + 0.04} = 0.024$$

c. Using Equation 36-8.1, solve for Q as follows:

For T = 6.10 ft, Q = 1.0 ft^3/s

36-9.0 INLETS

36-9.01 General

An inlet is a drainage structure which is utilized to collect surface water through a grate or curb opening and convey it to a storm drain or a direct outlet to a culvert. A grate inlet should be bicycle-safe unless located on a highway where bicycles are not permitted.

36-9.02 Types

Inlets used for the drainage of a highway surface can be divided into three major classes. These classes are discussed as follows. See the INDOT *Standard Drawings* for details on the inlet types used by the Department.

36-9.02(01) Grate Inlet

This consists of an opening in the gutter covered by one or more grates. It is best suited for use on a continuous grade. The grate is susceptible to clogging with debris and, thus, should be supplemented with a curb box and additional grate capacity to allow for partial clogging at a sag point. Flanking inlets are recommended at a major sag point. The grate should be bicycle safe where bicycle traffic is anticipated. It should be structurally designed to handle the appropriate loads if subject to traffic. The width of each inlet casting should match the width of the gutter. See Section 36-10.0 for additional discussion.

36-9.02(02) Combination Inlet

Various types of combination inlet are in use. A curb-box and grate combination is common with the curb opening adjacent to the grate. A slotted inlet is used in combination with a grate, located either longitudinally upstream of the grate, or transversely adjacent to the grate. Engineering judgment is necessary to determine if the total capacity of the inlet is the sum of the individual components or a portion of each. The gutter grade, cross slope, and proximity of the inlets to each other will be deciding factors. A combination inlet may be desirable in a sag because it can provide additional capacity if plugged.

36-9.02(03) Slotted-Drain Inlet

INDOT uses the slotted-drain-pipe inlet on a mainline roadway, and the slotted-vane-drain-pipe inlet on a drive. The slotted-drain is used to intercept sheet flow at the roadway edge. It can also be installed in a concrete gutter in conjunction with a curb inlet at a sag location. The slotted-vane-drain is used to intercept sheet flow on an urban drive. A slotted-drain inlet is used as a component of a storm-drainage system.

The slotted-drain pipe consists of a horizontal metal pipe with a continuous vertical riser and a slotted opening with bars perpendicular to the opening. The slotted-vane drain consists of a gray-iron casting which is placed on top of a horizontal PVC pipe encased in a low-grade concrete. Each type functions as a weir with flow entering from the side. It can be used to intercept sheet flow, collect gutter flow with or without a curb, modify an existing system to accommodate roadway widening or increased runoff, or reduce ponding depth and spread at a grate inlet.

36-9.03 Inlet Location

An inlet is required where needed to collect runoff within the design controls specified in the design criteria (Section 36-7.0). An inlet may be necessary where it contributes little to the drainage area. Such a location should be shown on the plans prior to performing computations regarding discharge, water spread, inlet capacity, or run-by. Location examples are as follows:

1. sag point in the gutter grade;
2. upstream of a median break, entrance- or exit-ramp gore, crosswalk, or street intersection;

3. immediately upstream and downstream of a bridge;
4. immediately upstream of a cross-slope reversal;
5. on a side street at an intersection;
6. at the end of a channel in a cut section;
7. behind a curb, shoulder, or sidewalk to drain a low area; or
8. where necessary to collect snowmelt.

An inlet should not be located in the path where a pedestrian is likely to walk.

36-10.0 INLET SPACING

36-10.01 General

A number of inlets are required to collect runoff at a location with little regard for contributing drainage area as discussed in Section 36-9.0. These should be plotted on the plan first. Locate inlets starting from the crest and working downgrade to the sag point. The location of the first inlet from the crest can be established by determining the length of pavement and the area back of the curb sloping toward the roadway which will generate the design runoff. The design runoff can be computed as the maximum allowable flow in the curbed channel which will satisfy the design criteria described in Section 13-7.0. Where the contributing drainage area consists of a strip of land parallel to and including a portion of the highway, the location of the first inlet can be calculated as follows:

$$L = \frac{43\,560 Qt}{CWi} \qquad \text{(Equation 36-10.1)}$$

Where:

	L	=	distance from the crest, ft
	Q_t	=	maximum allowable flow, ft^3/s
	C	=	composite runoff coefficient for contributing drainage area
	W	=	width of contributing drainage area, ft
	i	=	rainfall intensity for design frequency, in/h

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. Equation 36-10.1 is an alternative form of the Rational Equation.

To space successive downgrade 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 run-by. The run-by 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. Figure 36-10K is an inlet-spacing computation sheet which can be utilized to record the spacing calculations. An editable version of this form may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/.

36-10.02 Grate Inlet 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 a gutter inlet and a curb-opening inlet. At a low velocity, all of the water flowing in the section of gutter occupied by the grate, termed frontal flow, is intercepted by the grate inlet. A small portion of the flow along the length of the grate, termed side flow, is intercepted. On a steep slope, a portion of the frontal flow may tend to splash over the end of the grate. Figure 36-10A can be used to determine splashover velocity for a curved vane grate or reticuline grate. Data is not available for other grate types used by INDOT. An estimate of splashover velocity for a grate with a rectangular opening, such as the alternative grate for casting type 12, 13, or 14, is approximately 2.0 ft/s less than the splashover velocity for a reticuline grate.

INDOT recommends the curved vane grate for a curb-and-gutter application. Section 36-17.0 provides a hydraulic capacity chart for the curved vane grate inlet used by INDOT. The chart is based on a roadway cross section used by the Department. For another inlet type and roadway cross section, the procedure for determining the hydraulic performance is described below.

FHWA has developed computer software, HY12, which will analyze the flow in a gutter and the interception capacity of a grate inlet, curb-opening inlet, slotted-drain inlet, or combination inlet on a continuous grade. Both uniform and composite cross-slopes can be analyzed. The program can analyze a curb-opening, slotted-drain, or grate inlet in a sag. Enhanced versions by private vendors have made the program more user-friendly and have improved its usefulness. Not all INDOT grate configurations have been included in the HEC 12 program. The curved vane grate and the reticuline grate used in the program are similar to the INDOT grates and can be used by inputting the appropriate size. Other grate types, such as INDOT casting type 12, 13, or 14, are not included in HEC 12. However, grate-inlet-capacity curves are available from manufacturers and are recommended for use.

The ratio of frontal flow to total gutter flow, E_O, for a straight cross slope can be determined from the equation as follows:

$$E_O = \frac{Q_W}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67} \quad \text{(Equation 36-10.2)}$$

Where: Q = total gutter flow, ft^3/s
Q_W = flow in width W, ft^3/s

W = width of depressed gutter or grate, ft
T = total spread of water in the gutter, ft

Figure 36-8C provides a graphical solution for E_O for a straight cross slope or a depressed-gutter section.

The ratio of side flow, Q_S, to total gutter flow is as follows:

$$\frac{Q_S}{Q} = 1 - \frac{Q_W}{Q} = 1 - E_O \qquad \text{(Equation 36-10.3)}$$

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the equation as follows:

$$R_f = 1 - 0.09(V - V_O) \qquad \text{(Equation 36-10.4)}$$

Where: V = velocity of flow in the gutter, ft/s
V_O = gutter velocity where splashover first occurs, ft/s

This ratio is equivalent to frontal-flow-interception efficiency. Figure 36-10A provides a solution for Equation 36-10.4 which reflects grate length, bar configuration, and gutter velocity at which splashover occurs. The gutter velocity needed to use Figure 36-10A is total gutter flow divided by the area of flow.

The following equation may be used to solve for velocity in a triangular gutter section with known cross slope, slope, and spread.

$$V = \frac{1.12 S^{0.5} S_X^{\;0.67} T^{0.67}}{n} \qquad \text{(Equation 36-10.5)}$$

Where: V = velocity of flow in gutter, ft/s
S = longitudinal slope of gutter
S_X = cross slope
T = water spread, ft

Figure 36-10B illustrates the gutter cross section to which Equation 36-10.5 applies.

The ratio of side flow intercepted to total side flow, R_S, or side flow interception efficiency, is expressed as follows:

$$R_S = \frac{SL^{2.3}}{SL^{2.3} + 0.15V^{1.8}} \qquad \text{(Equation 36-10.6)}$$

Where: V = velocity of flow in gutter, ft/s
L = length of grate, ft
S_X = cross slope

Figure 36-10C provides a solution to Equation 36-10.6.

The efficiency, E, of a grate is expressed as follows:

$$E = R_f E_O + R_S(1 - E_O) \qquad \text{(Equation 36-10.7)}$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow, as follows:

$$Q_i = EQ = Q[R_f E_O + R_S(1 - E_O)] \qquad \text{(Equation 36-10.8)}$$

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Example 36-10.1

Given: Urban non-freeway; 4-lanes, undivided with crown at centerline
Drainage area: 200 ft residential strip, $C = 0.4$, $S = 0.005$
12 ft lane with 0.02 cross slope and 2 ft gutter at 0.025 cross slope
10-year design, IDF Curve for Indianapolis in Chapter Twenty-nine
Allowable spread $T = 8.0$ ft, $n = 0.016$
$S_O = 0.01$, $S_X = 0.02$, $S_W = 0.025$
Use curves and equations
Use INDOT standard grate types 10 and 11, 16 in. x 36 in.

Find: Maximum allowable flow, Q_T
Q_i intercepted by 16 in. x 36 in. vane grate
Q_r run-by
Location of first and second inlets from crest of hill

See Figure 36-10C(1) for sketch.

Solution:

Step 1. Solve for Q_S using Equation 36-8.1 as follows:

$$Q_S = \frac{0.56}{0.016}(0.02)^{1.67}(0.01)^{0.5}(8.0 - 2.0)^{2.67} = 0.6087 \text{ ft}^3/\text{s}$$

$$Q_S = 0.6087 \text{ ft}^3/\text{s}$$

Step 2. Use Figure 36-8C to find E_O as follows:

$$\frac{W}{T} = \frac{2.0}{8.0} = 0.25; \quad E_O = 0.55 = \frac{Q_W}{Q}; \quad \frac{S_W}{S_X} = \frac{0.025}{0.02} = 1.25$$

Step 3. Find total maximum allowable flow, Q_T,as follows:

$$Q_T = \frac{Q_S}{1 - E_O} = \frac{0.6087}{1 - 0.55} = 1.35 \text{ ft}^3\text{/s}$$

Step 4. Determine V from Equation 36-10.5 as follows:

$$V = \frac{1.12}{0.016}(0.01)^{0.5}(0.02)^{0.67}(8)^{0.67} = 2.05 \text{ ft/s}$$

Step 5. Determine Q_i from Equation 36-10.8 as follows:

$$Q_i = 1.35[(1.0)(0.55) + 0.35(1 - 0.55)] = 0.955 \ ft/s$$

Step 6. From Figure 36-10A, R_f = 1.0; from Figure 36-10C, R_S = 0.35.

Step 7. $Q_r = Q_T - Q_i$. Therefore, Q_r = 1.35 – 0.955 = 0.395.

Step 8. Locate first inlet from crest, use Equation 36-10.1. To find i in the equation, first solve for t_C. From Figure 29-7D, for a residential area, the following apply.

100-ft strip
C = 0.4
S = 0.5%, overland flow
t_C = 15 min
Gutter flow estimated at V = 2.03 ft/s from Step 4.

Try L = 330 ft. Therefore, $t_C = \frac{330}{(2.03)(60)} - 2.7$ min.

Total t_C = 15 + 2.7 = 17.7 min
From Figure 29-8C (IDF curve), i = 4.4 in./h
Solve for weighted C value as follows:

$$C = \frac{(100)(0.4) + (26)(0.9)}{126} = 0.50$$

Solve for L as follows:

$$Q_i = EQ = Q_T[R_f E_O + R_S(1 - E_O)]$$

$$L = \frac{43\,560(1.35)}{(0.50)(4.4)(126)} = 212 \text{ ft. No Good.}$$

Try i = 200 ft/h and recalculate L as follows:

$$L = \frac{43\,560(1.35)}{(0.50)(4.48)(126)} = 208 \text{ ft. OK.}$$

Therefore, place the first inlet 200 ft from the crest.

9. Step 9. Locate the second inlet.

Q_T = 1.35 ft^3/s, Q_r = 0.395 ft^3/s, $Q_{allowable}$ = 1.35 – 0.395 = 0.955 ft^3/s.
Assuming similar drainage area and t_C, i = 4.4 in/h.

$$L = \frac{43\,560(0.955)}{(0.50)(4.4)(126)} = 150 \text{ ft}$$

Therefore, place the second inlet 150 ft from the first inlet.

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36-10.03 Grate Inlet In Sag

36-10.03(01) Standard Practice

Standard practice is to install two curved vane grates, types 10 or 11, on one frame casting at the sag point. Each vane grate is positioned to receive water from each upstream direction. A curb box is combined with the grate to provide relief if the grate is plugged with debris. The curb box is ignored in the hydraulic-capacity calculations.

A grate inlet in a sag operates as a weir up to a depth of about 0.5 ft and as an orifice for a depth greater than 1.5 ft. Between these depths, a transition from weir- to orifice-flow occurs. The capacity of a grate inlet operating as a weir is as follows:

$$Q_i = CPd^{1.5} \qquad \text{(Equation 36-10.9)}$$

Where: P = perimeter of grate excluding bar widths and side against curb, ft
C = 3.0
d = depth of water at curb measured from the normal cross slope gutter flow line, ft

The capacity of a grate inlet operating as an orifice is determined as follows:

$$Q_i = CA(2gd)^{0.5} \qquad \text{(Equation 36-10.10)}$$

Where: C = orifice coefficient, 0.67
A = clear opening area of the grate, ft^2
g = acceleration due to gravity, 32.2 ft/s^2

Figure 36-10D is a plot of Equations 36-10.9 and 36-10.10 for various grate sizes. The effect of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either the weir or 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.

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Example 36-10.2

The following example illustrates the use of Figure 36-10D.

Given: A symmetrical sag vertical curve with equal bypass from inlets upgrade of the low point.

Q = 1.0 ft^3/s, design storm	Q_r = 0.35 ft^3/s
Q = 1.25 ft^3/s, check storm	Q_r = 0.53 ft^3/s
S_X = 0.02 ft/ft	T = 8.0 ft, design
$d = TS_X$ = 0.16 ft	n = 0.016

Use grate types 10 and 11 (16 in. x 36 in.)

Find: Grate size for design Q and depth at curb for check Q. Check spread at S = 0.003 on approaches to the low point.

Solution: Try one grate type 10 and one grate type 11.

1. <u>Design Storm</u>.

   a. P = 16 + 36 + 16 = 68 in. = 5.67 ft

   b. Using Equation 36-10.9, solve for allowable Q as follows:

   $$Q = (3.0)(5.67)(0.16)^{1.5} = 1.089 \qquad \text{OK}$$

In accordance with INDOT policy, one grate type 10 and one grate type 11, each with a curb box, will be placed at the sag point. The curb boxes will not be analyzed hydraulically, but are available for drainage if the inlet becomes plugged.

2. Check Storm.

a. P = 2(16) + 2(36) = 104 in. ≈ 8.67 ft

b. Determine d as follows:

$$d^{1.5} = \frac{Q}{3.0P} = \frac{1.089}{(3.0)(8.67)} = 0.42. \quad d = 0.119 \text{ ft} \qquad \text{OK}$$

INDOT practice is to provide a grade of 0.3% within 50 ft of the level point in a sag vertical curve. Check T at S = 0.003 for the design and check the flow as follows:

c. Use Equation 36-8.1. Determine T for the design storm as follows:

$$T^{2.67} = \frac{(0.35)(0.016)}{0.56(0.02)^{1.67}(0.003)^{0.5}} = 125.5 \quad T = 6.11 \text{ ft} \qquad \text{OK}$$

d. Determine T for the check storm as follows:

$$T^{2.67} = \frac{(0.53)(0.016)}{0.56(0.02)^{1.67}(0.003)^{0.5}} = 7.99. \quad T = 7.14 \text{ ft} \qquad \text{OK}$$

Thus, a standard castings type 10 and 11 are adequate to intercept the design flow at a spread which does not exceed the design spread. The standard INDOT practice of placing two grates with curb boxes will intercept a check storm within the design criteria and allow for some plugging with debris.

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36-10.03(02) Flanking Inlets

At a major sag point where significant ponding can occur, such as an underpass or sag vertical curve in a depressed section, a minimum of one flanking inlet should be placed on each side of the inlet at the sag point. The flanking inlets should be placed so that they will limit spread on a low-grade approach to the level point and act in relief of the sag inlet if it becomes clogged. Figure 36-10E shows the spacing required for depth-at-curb criteria and vertical curve length defined by $K = L/A$, where L is the length of the vertical curve in feet and A is the algebraic difference in approach grades. The INDOT geometrics specify a maximum K value for the design speed, and a maximum K of 170 considering drainage on a curbed facility.

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Example 36-10.3

Given: Data from Example 36-10.2:

Speed = 57 mi/h, K = 40 S_X = 0.02 T = 8.0 ft, design

Find: Location of flanking inlets so that they will function in relief of the inlet at the low point where the depth at the curb exceeds the design depth.

Solution: Allowable depth, d, at curb = 8.0 ft at 0.02 = 0.16 ft

Spacing to flanking inlet, x = 64.3 ft, from Figure 36-10E by interpolation.

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36-10.04 Slotted Inlet

36-10.04(01) Divided Facility with Median Barrier

Snow accumulation adjacent to a concrete barrier on the inside or outside shoulder can present a drainage problem. Therefore, Department practice is to use a slotted drain in conjunction with inlet type H-5 or HA-5 as follows.

1. Tangent Section. Use at every third inlet.

2. Low Side of Superelevated Curve. Use at all inlet sites.

3. Sag Vertical Curve. Use three, centered on the low point.

See the INDOT *Standard Drawings* for more-detailed information.

36-10.04(02) High-Side Shoulder

A slotted-drain pipe is used at locations as follows:

1. high-side shoulder of a superelevated section;
2. the high-side shoulder slopes toward the traveled way;
3. high-traffic-volume freeway; or
4. roadway that is either curbed or uncurbed.

See the INDOT *Standard Drawings* for more-detailed information.

36-10.05 Slotted Inlet on Grade

A slotted inlet, which uses a vertical riser, is an effective pavement drainage inlet which has a variety of applications. It can be used on a curbed or uncurbed section, and offers little interference to traffic operations. It can be placed longitudinally in the gutter or transversely to the gutter. A slotted inlet should be connected into an inlet structure so that it will be accessible to maintenance forces upon plugging or freezing.

36-10.05(01) Longitudinal Placement

Flow interception by a slotted-drain pipe and a curb-opening inlet 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. A slotted inlet can have economic advantages and can be useful in a widening or safety project where right of way is narrow and existing inlet capacity must be supplemented. A curb can be eliminated as a result of utilizing a slotted inlet.

The length of a slotted-drain pipe required for total interception of gutter flow on a pavement section with a straight cross slope is expressed as follows:

$$L_T = KQ^{0.42} S^{0.3} \left(\frac{1}{nS_X} \right)^{0.6} \qquad \text{(Equation 36-10.11)}$$

Where: K = 0.60
L_T = slotted inlet length required to intercept 100% of gutter flow, ft

Figure 36-10H illustrates the gutter cross section to which Equation 36-10-11 applies.

The INDOT standard slotted-drain-pipe slot width is 1-¾ in. and the length is 20 ft. The efficiency of a slotted inlet shorter than the length required for total interception is expressed as follows:

$$E = 1 - \left(1 - \frac{L}{L_T} \right)^{1.8} \qquad \text{(Equation 36-10.12)}$$

Where L = slotted-inlet length, ft

Figure 36-10 I provides a solution of Equation 36-10.12.

The length of inlet required for total interception by a slotted inlet in a composite section can be determined by using an equivalent cross slope, S_e, as follows:

$$S_e = S_X + S'_W E_O \qquad \text{(Equation 36-10.13)}$$

Where: S_X = pavement cross slope
S_W = gutter cross slope
$S'_W = S_W - S_X$
E_O = ratio of flow in the depressed gutter to total gutter flow, Q_W/Q (see Figure 36-8C)

The same equations are used for a slotted inlet or a curb-opening inlet. The following example illustrates the use of this procedure.

* * * * * * * * * *

Example 36-10.4

Given: Longitudinal placement of slotted inlet adjacent to curb.

$S_O = 0.01$ Allowable spread = 10.0 ft $n = 0.016$ $W = 2.0$ ft
Uniform cross slope, $S_X = 0.02$
Composite cross slope, $S_X = 0.02$, $S_W = 0.025$

Find:
(1) Maximum allowable Q
Q_i for a 20.0 ft slotted inlet on a straight cross slope.
(2) Maximum allowable Q
Q_i for a 10.0 ft slotted inlet on a composite cross slope.

Solution: (1) Determine maximum allowable Q from Equation 36-8.1 as follows:

$$Max\ Q = \frac{0.56}{0.016}(0.02)^{1.67}(0.01)^{0.5}(10)^{2.67} = 2.38 \text{ ft}^3/\text{s}$$

Determine L_T from Equation 36-10.11 as follows:

$$L_T - 0.60(2.38)^{0.42}(0.01)^{0.3}\left[\frac{1}{(0.016)(0.02)}\right]^{0.6} = 27.12 \text{ ft}$$

Therefore, $\frac{L}{L_T} = \frac{20}{27.12} = 0.74$

From Figure 36-10I, E = 0.91.

$Q_i = EQ$ = (0.91)(2.38) = 2.17 ft^3/s intercepted.

(2) Determine Q_S from Equation 36-8.1 as follows:

$$Q_S = \frac{0.560}{0.016}(0.02)^{1.67}(0.01)^{0.5}(10.0-2.0)^{2.67} = 1.312 \text{ ft}^3\text{/s}$$

$$\frac{W}{T} = \frac{2.0}{10} = 0.2. \quad \frac{S_W}{S_X} = \frac{0.025}{0.02} = 1.25.$$

From Figure 36-8C, E_O = 0.46.

$$MaxQ = \frac{1.312}{1-0.46} = 2.43 \text{ ft}^3\text{/s}$$

$S'_W = S_W - S_X$ = 0.025 – 0.02 = 0.005
$S_e = S_X + S'_W E_O$ = 0.02 + (0.005)(0.46) = 0.022

Determine L_T from Equation 36-10.11 as follows:

$$L_T - 0.60(2.38)^{0.42}(0.01)^{0.3}\left[\frac{1}{(0.016)(0.022)}\right]^{0.6} = 25.6 \text{ ft}$$

Therefore, $\frac{L}{L_T} = \frac{20}{25.6} = 0.78$

From Figure 36-10I, E = 0.92.

$Q_i = EQ$ = (0.92)(2.38) = 2.19 ft^3/s intercepted.

* * * * * * * * * *

36-10.05(02) Transverse Placement of Slotted Vane Drain

At a drive where it is desirable to capture virtually all of the flow (e.g., in a drive sloped toward the roadway), a slotted-vane drain can be installed in conjunction with a grate inlet. Tests have indicated that, if the slotted-vane drain is installed perpendicular to the flow, it will capture approximately 1.589 ft^3/s per running foot of drain on a longitudinal slope of 0% to 6%. Capacity curves are available from the manufacturers. The ideal installation utilizes a grate inlet to capture the flow in the gutter and the slotted-vane drain to collect the flow extending into the shoulder. A slotted-vane drain is shaped and rounded to increase inlet efficiency and should not be confused with a vertical-riser-type slotted inlet (i.e., a slotted-drain pipe).

36-10.06 Slotted Inlet In Sag Location

Except adjacent to a concrete barrier (Section 36-10.04), the use of a slotted-drain inlet in a sag configuration is discouraged because of the propensity of such an inlet to collect debris. However, there may be a location where it is desirable to supplement an existing low point inlet with the use of a slotted drain. A slotted inlet in a sag location performs as a weir to a depth of about 0.2 ft, dependent on slot width and length. At a depth greater than about 0.4 ft, it performs as an orifice. Between these depths, flow is in a transition stage. The interception capacity of a slotted inlet operating as an orifice can be computed from the equation as follows:

$$Q_i = 0.8\,L\,W\,(2gd)^{0.5} \qquad \text{(Equation 36-10.14)}$$

Where:
- W = width of slot, ft
- L = length of slot, ft
- d = depth of water at slot, ft
- g = acceleration due to gravity, 32.2 ft/s^2

For a slot width of 1-¾ in., the above equation becomes the following:

$$Q_i = 0.94Ld^{0.5} \qquad \text{(Equation 36-10.15)}$$

The interception capacity of a slotted inlet at a depth 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 36-10J provides solutions for weir flow and a plot representing data at depth between weir and orifice flow.

36-10.07 Inlet-Spacing Computations

To determine the locations of the inlets for a given project, information such as a layout or plan sheet suitable for outlining drainage area, road profile, typical cross section, grading cross section, superelevation diagram, and contour maps is necessary. The inlet computation sheet (Figure 36-10K) should be used to document the computations. The procedure is as follows.

1. Complete the blanks on top of the sheet to identify the job by project, route, date, and initials.

2. Mark on the plan the location of inlets which are necessary without considering specific drainage area. See Section 36-9.03 for additional information.

3. Start at one end of the project, at one high point, and work toward the low point, then space from the other high point back to the same low point.

4. Select a trial drainage area of approximately 300 ft to 500 ft below the high point and, using a drainage-area map, outline the area including drainage that may come over the curb. Where practical, a large area of behind-curb drainage should be intercepted before it reaches the highway. See Section 36-6.05.

5. Describe the location of the proposed inlet by number and station in columns 1 and 2. Identify the curb and gutter type in column 19. A sketch of the cross section should be provided in the open area of the computation sheet.

6. Compute the drainage area in acres and enter in column 3.

7. Select a C value from the appropriate table in Chapter Twenty-nine and enter in column 4.

8. 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 Chapter Twenty-nine. The minimum time of concentration should be 5 min. Enter value in column 5.

9. Using the Intensity-Duration-Frequency curves from Chapter Twenty-nine, select a rainfall intensity at the t_C for the design frequency. Enter in column 6.

10. Calculate Q by multiplying the values in columns 3, 4, and 6. Enter in column 7.

11. Determine the gutter slope at the inlet from the profile grade. Check the effect of superelevation. Enter in column 8.

12. Enter the cross slope adjacent to the inlet in column 9 and the gutter width in column 13. Sketch the composite cross slope and include dimensions.

13. For the first inlet in a series (high point to low point), enter the value from column 7 in column 11 because no previous run-by has occurred yet.

14. Using Equation 36-8.1 or the available computer model, determine the spread, *T*, and enter it in column 14. Calculate the depth *d* at the curb by multiplying *T* by the cross slopes, and enter in column 12. Compare with the allowable spread as determined by the design criteria described in Section 36-7.0. If the value in column 15 is less than the curb height, and the value in column 14 is near the allowable spread, continue on to Step 16. If not acceptable, expand or decrease the drainage area to satisfy the criteria and repeat Steps 5 through 14. Continue until the value in column 14 is near the allowable spread, and then proceed to Step 15.

15: Calculate *W*/*T* and enter in column 15.

16. Select the inlet type and dimensions and enter in column 16.

17. Calculate the intercepted flow, Q_i, and enter in column 17. Use Equation 36-8.1 and Figure 36-8C or Figure 36-8D to define the flow in the gutter. Use Figures 36-8C, 36-10A, and 36-10C, and Equation 36-10.8 to calculate Q_i for a grate inlet, and Equation 36-10.11 to calculate Q_i for a curb-opening inlet. See Section 36-10.02 for a grate-inlet example.

18. Calculate the run-by by subtracting the value in column 17 from that in column 11 and enter into column 18 and also into column 10 on the next line if an additional inlet exists downstream.

19. Proceed to the next inlet downgrade. Select an area approximately 300 ft to 400 ft below the first inlet as a first trial. Repeat Steps 5 through 7 considering only the area between the inlets.

20. Compute a time of concentration for the second inlet downgrade based on the area between the two inlets.

21. Determine the intensity based on the time of concentration determined in Step 20 and enter it in column 6.

22. Determine the discharge from this area by multiplying the values in columns 3, 4, and 6. Enter the discharge in column 7.

23. Determine total gutter flow by adding the values in columns 7 and 10, and enter in column 11. The value in column 10 is the same as that in column 18 from the previous line.

24. Determine *T* based on total gutter flow from column 11 by using Equation 36-8.1 or Figure 36-8D and enter in column 14. If *T* exceeds the allowable spread, reduce the area and repeat Steps 19 through 24. If *T* is substantially less than the allowable spread, increase the area and repeat Steps 19 through 24.

25. Select inlet type and dimensions and enter in column 16.

26. Determine Q_i and enter in column 17. See Step 17.

27. Calculate the run-by by subtracting the value in column 17 from that in column 7 and enter in column 16. This completes the spacing design for this inlet.

28. Return to Step 19 and repeat Steps 19 through 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.

36-11.0 MANHOLES

36-11.01 Location

A manhole is utilized to provide entry to a continuous underground storm drain for inspection and cleanout. An inlet box with a grate may be used in lieu of a manhole on the upper end of a storm-drain run to provide access to the system. In this manner, stormwater interception can be achieved with minimal additional cost. The locations where a manhole should be specified are as follows:

1. where two or more storm drains converge;
2. at an intermediate points along a tangent section;
3. where the pipe size changes;
4. where an abrupt change in alignment occurs; and
5. where an abrupt change of the grade occurs.

A manhole should not be located in a traffic lane. However, if this is impossible, it should not be in the normal vehicle wheel path. Where practical, a manhole should be located off the roadway.

36-11.02 Spacing

The spacing should be a maximum of 400 ft.

36-11.03 Types

The types of manholes used by INDOT are listed in Figure 36-11A. The type selected is dependent on the storm-drain pipe size and depth of the manhole.

36-11.04 Sizing

In determining the minimum round manhole size required for a given trunkline pipe size and location, the criteria to be satisfied are as follows:

1. The manhole or inlet structure must be large enough to accept the maximum pipe size shown in Figure 36-11B. In addition to accommodating the maximum pipe size, ensure that not too many pipes enter the manhole to threaten its structural capacity.

2. Knowing the relative locations of two pipes, compute the following:

$$K = \frac{R_1 + T_1 + R_2 + T_2 + 14.2\,in}{\Delta} \qquad \text{(Equation 36-11.1)}$$

Where: K is the in./deg (see Figure 36-11B)
R_1 and T_1 are the interior radius and wall thickness of Pipe No. 1, in.
R_2 and T_2 are the interior radius and wall thickness of Pipe No. 2, in.
Δ = angle between the pipes, deg

* * * * * * * *

Example 36-11.1

Given: Pipe No. 1 dia. = 54 in., Pipe No. 2 dia. = 48 in.
Δ = 140 deg

Solution:
$$\frac{27\,in + 5.6\,in + 24\,in + 5.08\,in + 14.2\,in}{140°}$$

K = 0.542 in./deg

The table indicates the minimum manhole barrel to be 66 in. For the 1650-barrel, the table indicates a maximum pipe size of 48 in. Because the maximum pipe size in the example is 54 in., a 72-in. manhole must be used.

For this example, spacing is not critical and the pipe size governs. If Δ is 115 deg or less, the spacing is critical and a larger manhole barrel is required. If pipes are located at substantially different elevations, pipes may not conflict and the above analysis is unnecessary.

See Figure 36-11C for pipe layout.

36-12.0 STORM DRAINS

36-12.01 Introduction

The design frequency for storm-drain design is 10 years utilizing gravity-flow techniques. The trunkline should be checked only utilizing HGL techniques for the 50-year storm.

After the preliminary locations of inlets, connecting pipes, and outfalls with tailwater have been determined, the next 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 grade 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. At a manhole where the pipe size is increased, the pipe crowns should match where grades permit.

The rate of discharge at a point in the storm drain is not necessarily the sum of the inlet flow rates of all inlets above that section of storm drain. It is less than this total. 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. Where a relatively large drainage area with a short time of concentration is added to the system, the peak flow may be larger using the shorter time, though the entire drainage area is not contributing. Flows may arrive at a manhole at which no additional flows enter. Although the time of concentration at this point is technically longer, the flow rate in the downstream pipe should not be reduced. The designer should be aware of unusual conditions and should determine which time of concentration controls for each pipe segment. See Chapter Twenty-nine for a discussion on time of concentration.

For ordinary conditions, a storm drain should be sized based on the assumption that it will flow full or practically full under the design discharge but will not flow under pressure head. The Manning's formula is recommended for capacity calculations. In a depressed section or underpass where ponded water can be removed only through the storm-drain system, a higher design frequency, 50 years, should be considered to design the storm drain which drains the sag point. See Section 36-10.08 for a discussion on the location of flanking inlets. The main storm drain downstream of the depressed section should be designed by computing the hydraulic grade line and keeping the water-surface elevations below the grates or the established critical elevations for the design storm.

36-12.02 Design Procedure

The storm-drainage system design procedure is as follows.

1. Determine inlet locations and spacing as outlined earlier herein.

2. Prepare a plan layout of the storm-drainage system establishing the design data as follows:

    a. location of storm drains;

    b. direction of flow;

    c. location of manholes; and

    d. locations of existing utilities such as water, gas, sanitary sewer, or underground cables.

3. Determine drainage area, runoff coefficient, and a time of concentration to the first inlet. Using an Intensity-Duration-Frequency (IDF) curve for 10-year recurrence interval, determine the rainfall intensity. Calculate the discharge as $1.008ACI$.

4. Size the pipe to convey the discharge by varying the slope and pipe size as necessary. A storm-drain system is designed for full gravity flow conditions using the design-frequency discharges.

5. Calculate travel time in the pipe to the next inlet or manhole by dividing the 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.

6. Calculate the new area, A, and multiply by the runoff coefficient, C. Add to the previous A time C product. Multiply by 1.008 and the new rainfall intensity, I, to determine the new discharge. Determine the size of pipe and slope necessary to convey the discharge.

7. Continue this process to the storm-drain outlet. Utilize the equations or nomographs to accomplish the design.

8. Complete the design by calculating the hydraulic grade line as described in Section 36-13.0 for the trunkline only for the 50-year recurrence interval. The design procedure should include the following.

   a. Storm-drain design computations can be made on forms as illustrated in Figure 36-12F. An editable version of this form may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/.

   b. All computations and design sheets should be identified. The designer's initials and date of computations should be shown on each sheet. Voided or superseded sheets should be so marked. The origin of data used on one sheet but computed on another should be identified.

36-12.03 50-Year Sag Point

As indicated above, the storm drain which drains a major sag point should be sized to accommodate the runoff from a 50-year frequency rainfall. This can be done by actually computing the run-by occurring at each inlet during a 50-year rainfall and accumulating it at the sag point. The inlet at the sag point as well as the storm-drain pipe leading from the sag point must be sized to accommodate this additional run-by within the criteria established. See Section 36-7.0. To design the pipe leading from the sag point, convert the additional run-by created by the 50-year rainfall into an equivalent CA which can be added to the design CA. This equivalent CA can be approximated by dividing the 50-year run-by by 1.008 x I_{10} in the pipe at the sag point.

A separate system may be designed to prevent the above-ground system from draining into the depressed area. This concept may be more costly but may be justified. Another method is to design the upstream system for a 50-year design to minimize the run-by to the sag point. Each method must be evaluated on its own merits and the impacts and risk of flooding a sag point must be assessed.

36-12.04 Hydraulic Capacity

The formula for determining the hydraulic capacity of a storm-drain for gravity and pressure flow is the Manning's formula, expressed by the equation as follows:

$$V = \frac{1.486}{n} R^{0.67} S^{0.5} \quad \text{(Equation 36-12.1)}$$

Where: V = mean velocity of flow, ft/s
n = Manning's roughness coefficient

R = hydraulic radius, ft. R = area of flow divided by the wetted perimeter
S = slope of the energy grade line

In terms of discharge, the above formula becomes the following:

$$Q = VA = \frac{1.486}{n} AR^{0.67} S^{0.5} \quad \text{(Equation 36-12.2)}$$

Where: Q = rate of flow, ft^3/s
A = cross-sectional area of flow, ft^2

For a storm drain flowing full, the above equations become the following:

$$V = \frac{0.59}{n} SD^{0.67} \qquad Q = \frac{0.463}{n} S^{0.5} D^{2.67} \quad \text{(Equation 36-12.3)}$$

Where D = diameter of pipe, ft

The nomograph solution of Manning's formula for full flow in a circular storm drain is shown in Figure 36-12A, Figure 36-12B, and Figure 36-12C. Figure 36-12D has been provided to assist in the solution of Manning's formula for part full flow in a storm drain.

36-12.05 Minimum Grade

A storm drain should be designed such that the velocity of flow will not be less than 2.67 ft/s at design flow. For a flat grade, the components should be designed so that the flow velocity will increase progressively throughout the length of the pipe system. The storm-drainage system should be checked to ensure that there is sufficient velocity in all of the drains to deter settling of particles. The minimum slope, S, required for a velocity of 2.67 ft/s can be calculated as shown below, or the value shown in Figure 36-12E can be used.

$$S = \frac{(nV)^2}{R^{1.33}} \quad \text{(Equation 36-12.4)}$$

The maximum velocity of flow should not exceed 8.3 ft/s.

36-13.0 HYDRAULIC GRADE LINE

36-13.01 Introduction

The hydraulic grade line (HGL) is the last feature to be established regarding the hydraulic design of a storm drain. This grade line aids the designer in determining the acceptability of the proposed system by establishing the elevations along the system to which the water will rise if the system is operating from a flood of design frequency. INDOT policy is that the maximum HGL is at the top of the inlet or manhole for Q_{50} flow.

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. A concern with a storm drain designed to operate under pressure-flow conditions is that inlet surcharging and possible manhole-lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on open-channel conditions must be planned, including evaluation of the potential for excessive and inadvertent flooding created if 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. A storm-drain system can often alternate between pressure and open-channel flow conditions from one section to another.

The detailed methodology employed in calculating the HGL through the system begins at the system outfall with the tailwater elevation. If the outfall is an existing storm-drain system, the HGL calculation must begin at the outlet end of the existing system and proceed upstream through the in-place system, then upstream through the proposed system to the upstream inlet. The same considerations apply to the outlet of a storm drain as to the outlet of a culvert. See Figure 31-5D for a sketch of a culvert outlet which depicts the difference between the HGL and the energy grade line (EGL). The EGL should be computed first, then the velocity head, $V^2/2g$, should be subtracted to obtain the HGL.

36-13.02 Tailwater

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 is a very large outfall with low tailwater for which a water-surface profile calculation is appropriate to determine the location where the water surface will intersect the top of the barrel and full-flow calculations can begin. The downstream water-surface elevation is based on critical depth or the tailwater, whichever is higher.

In estimating tailwater depth on the receiving stream, the designer will consider the joint or coincidental probability of two events occurring at the same time. For 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 which causes peak discharge on a small basin may not be critical for a larger basin. If the same storm causes peak discharge on both basins, the peaks will be out of phase. To aid in the evaluation of joint probabilities, see to Figure 36-13A.

36-13.03 Exit Loss

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 determined as follows:

$$H_o = 1.0\left(\frac{V^2}{2g} - \frac{V_d^2}{2g}\right) \qquad \text{(Equation 36-13.1)}$$

Where: V = average outlet velocity, ft/s
V_d = channel velocity downstream of outlet, ft/s

If $V_d = 0$ as in a reservoir, the exit loss is one velocity head. At a location with a flap gate at the outlet to prevent water from backing up into the system, an additional loss caused by the flap gate may need to be added. A manufacturer should be consulted for information. For part full flow where the pipe outlets into a channel with moving water, the exit loss may be reduced to virtually zero.

36-13.04 Bend Loss

The bend loss coefficient for storm-drain design is minor, but can be evaluated using the formula as follows:

$$H_b = \frac{0.0033\Delta V_o^{\ 2}}{2g} \qquad \text{(Equation 36-13.2)}$$

Where Δ = angle of curvature, deg.

36-13.05 Pipe-Friction Loss

The friction slope is the energy gradient for that run. The friction loss is the energy gradient multiplied by the length of the run in feet. Energy loss from pipe friction can be determined from the Manning's formula with terms as previously defined, as follows:

$$S_f = \left(\frac{Qn}{AR^{0.67}}\right)^2 \qquad \text{(Equation 36-13.3)}$$

The head loss due to friction can be determined from the formula as follows:

$$H_f = LS_f \qquad \text{(Equation 36-13.4)}$$

The Manning's formula can be used to determine friction loss as follows:

$$H_f = \frac{2.87n^2V^2L}{D^{1.33}} \qquad \text{(Equation 36-13.5)}$$

$$H_f = \left(\frac{64.4n^2L}{R^{1.33}}\right)\left(\frac{V^2}{2g}\right) \qquad \text{(Equation 36-13.6)}$$

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/s
- R = hydraulic radius, ft
- g = acceleration due to gravity, 32.2 ft/s^2
- S_f = slope of hydraulic grade line

36-13.06 Manhole Losses

The head loss encountered in flowing from one pipe to another through a manhole is represented as being proportional to the velocity head at the outlet pipe. Using K to identify this constant of proportionality, the energy loss is approximated as $K(V_o^2/2g)$. Experimental studies have determined that K can be approximated as follows:

$$K = K_O\, C_D\, C_d\, C_Q\, C_p\, C_B \qquad \text{(Equation 36-13.7)}$$

Where:
- K = adjusted loss coefficient
- K_O = initial head loss coefficient based on relative manhole 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

1. Relative Manhole Size. K_O is estimated as a function of the relative manhole size and the angle of deflection between the inflow and outflow pipes. See Figure 36-13B.

$$K_O = 0.1\left(\frac{b}{D_O}\right)(1-\sin\theta)+1.4\sin\theta\left(\frac{b}{D_O}\right)^{0.15} \qquad \text{(Equation 36-13.8)}$$

Where: θ = angle between the inflow and outflow pipes, deg
b = manhole diameter, in.
D_O = outlet-pipe diameter, in.

2. Pipe Diameter. A change in head loss due to differences in pipe diameter is significant only in a pressure-flow situation where the depth in manhole to outlet pipe diameter ratio, d/D_O, is greater than 3.2.

$$C_D = \left(\frac{D_O}{D_i}\right)^3 \qquad \text{(Equation 36-13.9)}$$

Where: D_i = incoming pipe diameter, in.
D_O = outgoing pipe diameter, in.

3. Flow Depth. The correction factor for flow depth is significant only in free-surface flow or low pressure, where the d/D_O ratio is less than 3.2. Water depth in the manhole is approximated as the level of the hydraulic-grade line at the upstream end of the outlet pipe. The correction factor for flow depth, C_d, is calculated as follows:

$$C_d = 0.5\left(\frac{d}{D_O}\right)^{0.6} \qquad \text{(Equation 36-13.10)}$$

Where: d = water depth in manhole above outlet pipe, ft
D_O = outlet pipe diameter, ft

4. 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:

$$C_Q = (1-2\sin\theta) \text{ x } \left(1-\frac{Q_i}{Q_O}\right)^{0.75} + 1 \qquad \text{(Equation 36-13.11)}$$

Where: C_Q = correction factor for relative flow
θ = angle between the inflow and outflow pipes, deg

Q_i = flow in the inflow pipe, ft^3/s
Q_O = flow in the outlet pipe, ft^3/s

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 manhole shown in the sketch and assume the following two discharge situations to determine the impact of pipe 2 entering the access hole.

$$C_{Q3-1} = (1 - 2\sin 180)\left(\frac{1-3.18}{4.24}\right)^{0.75} + 1 = 1.35$$

a. For discharge situation 1, Q_1 = 3.18 ft^3/s, Q_2 = 1.06 ft^3/s, and Q_3 = 4.24 ft^3/s. Therefore, C_Q = 1.35.

b. For discharge situation 2, Q_1 = 1.06 ft^3/s, Q_2 = 3.18 ft^3/s, and Q_3 = 4.24 ft^3/s. Therefore, C_Q = 1.81.

See Figure 36-13B(1) for relative-flow effect.

5. Plunging Flow. The correction factor for plunging flow, C_p, is calculated as follows:

$$C_p = 1 + 0.2\left(\frac{h}{D_O}\right) x \left[\frac{(h-d)}{D_O}\right] \qquad \text{(Equation 36-13.12)}$$

Where: C_p = correction for plunging flow
h = vertical distance of plunging flow from flowline of incoming pipe to the center of outlet pipe, ft
D_O = outlet pipe diameter, ft
d = water depth in manhole, ft

This correction factor corresponds to the effect of another inflow pipe or surface flow from an inlet, plunging into the manhole, on the inflow pipe for which the head loss is being calculated. Using the notations in the above sketch for the example, C_p is calculated for pipe No. 1 if pipe No. 2 discharges plunging flow. The correction factor is applied only if $h > d$.

6. Benching. The correction for benching in the manhole, C_B, is obtained from Figure 36-13C. Benching tends to direct flow through the manhole, resulting in reductions in head loss. For a flow depth between the submerged and unsubmerged conditions, a linear interpolation is performed.

7. Summary. To estimate the head loss through a manhole 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.

36-13.07 Hydraulic-Grade-Line Design Procedure

The equations and charts necessary to manually calculate the location of the hydraulic gradeline are included herein. The HYDRA computer program in the HYDRAIN system is recommended for design of a storm drain. It will include a HGL analysis and a pressure-flow simulation. A step-by-step procedure is described to manually compute the HGL. Figure 36-13D can be used to document the procedure. An editable version of this form may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/.

If the HGL is above the pipe crown at the next upstream manhole, pressure-flow calculations should be performed. If it is below the pipe crown, open-channel-flow calculations should be used at the upstream manhole. The process is repeated throughout the storm-drain system. If all HGL elevations are acceptable, the hydraulic design is adequate. If the HGL exceeds an inlet elevation, adjustments to the trial design must be made to lower the water-surface elevation.

1. Enter in column 1 the station for the junction immediately upstream of the outflow pipe. HGL computations begin at the outfall and are worked upstream taking each junction into consideration.

2. Enter in column 2 the tailwater elevation if the outlet will be submerged during the design storm. Otherwise, refer to the tailwater discussion in Section 36-13.02 for the procedure.

3. Enter in column 3 the diameter of the outflow pipe, D_O.

4. Enter in column 4 the design discharge for the outflow pipe, Q_O.

5. Enter in column 5 the length of the outflow pipe, L_O.

6. Enter in column 6 the outlet velocity of flow, V_O.

7. Enter in column 7 the velocity head, $V_O^2/2g$.

8. Enter in column 8 the exit loss, H_O.

9. Enter in column 9 the friction slope of the outflow pipe, SF_O. This can be determined by using Equation 36-13.3. This assumes full-flow conditions.

10. Enter in column 10 the friction loss, H_f, which is computed by multiplying the length, L_O, in column 5 by the friction slope, SF_O, in column 9. For a curved alignment, calculate curve loss by using the formula $H_b = 0.0033\theta V_O^2/2g$, where θ = angle of curvature, deg, and add to the friction loss.

11. Enter in column 11 the initial head loss coefficient, K_O, based on relative manhole size as computed from Equation 36-13.8.

12. Enter in column 12 the correction factor for pipe diameter, C_D, as computed from Equation 36-13.9.

13. Enter in column 13 the correction factor for flow depth, C_d, as computed from Equation 36-13.10. This factor is significant only where the d/D_O ratio is less than 3.2.

14. Enter in column 14 the correction factor for relative flow, C_Q, as computed from Equation 36-13.11.

15. Enter in column 15 the correction factor for plunging flow, C_p, as computed from Equation 36-13.12. This correction factor is applied only if $h > d$.

16. Enter in column 16 the correction factor for benching, C_B, as determined from Figure 36-13C.

17. Enter in column 17 the value of K as computed from Equation 36-13.7.

18. Enter in column 18 the value of the total manhole loss, $KV_O^2/2g$.

19. If the tailwater submerges the outlet end of the pipe, enter in column 19 the sum of the TW-elevation value in column 2 and the exit-loss value in column 7 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 $(d_C + D)/2$.

20. Enter in column 20 the sum of the friction-head value from column 10, the manhole-losses value from column 18, and the energy-grade-line value from column 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.

21. Determine the HGL for column. 21 throughout the system by subtracting the velocity-head value from column 7) from the EGL value from column 20.

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.

The above procedure applies to a pipe that is flowing full, as should be the condition for the design of a new system. If a part full flow condition exists, the EGL is located one velocity head above the water surface.

Figure 36-13E provides a summary of energy losses which should be considered. Figure 36-13F illustrates the proper and improper use of energy losses in developing a storm-drain system.

36-14.0 UNDERDRAINS

See Section 52-10.0 for INDOT's design criteria and application for underdrains.

36-15.0 COMPUTER PROGRAMS

To assist with storm-drain-system design, a microcomputer software module has been developed for the computation of the hydraulic grade line. The computer program, called HYDRA, is part of the HYDRAIN system. HYDRA can be used to check design adequacy and to analyze the performance of a storm-drain system under assumed inflow conditions. Section 36-16.0 provides an example problem using the HYDRA computer model.

If another commercial package is used, it must be capable of computing the trunkline size for gravity flow at a 10-year event and performing the hydraulic-grade-line computation for the 50-year event. Hand calculations which satisfy these requirements are also acceptable.

For slotted-drain pipe and slotted-vane-drain pipe, use manufacturers' publications with capture-rate information for a sag or on-grade installation.

36-16.0 EXAMPLE PROBLEM

The following is an example problem of inlet and storm-drain computations worked manually and using microcomputer software. The inlet computations utilized HEC 12, available from McTRANS. The storm-drain calculations utilized HYDRA.

Given: Sketch of roadway segment with inlets located as shown in Figure 36-16A. The drainage area is as indicated on the Inlet Computation sheet.

12-ft travel lane with $S_X = 0.02$, and 2.0-ft gutter with $S_W = 0.025$
10-year design, IDF curve for Indianapolis
Allowable spread, $T = 6.0$ ft; $n = 0.016$; $S_O = 0.012$
Curved vane grates type 10 and 11, size 16 in x 36 in

Find: Check spread at inlet locations. Design storm-drain size and slope. Calculate hydraulic grade line.

Solution:

1. Inlet Computations.

    a. Hand calculations documented in Figure 36-16B, Inlet Computation Sheet.

    b. Analysis computed by HEC 12 computer program and documented on printout labeled HEC 12, after Figure 36-16B.

2. Storm-Drain Calculations.

    a. Hand calculations for pipe sizing in Figure 36-16C, Storm-Drain Computation Sheet.

    b. HYDRA calculations for pipe sizing and Hydraulic-Grade Line shown on HYDRA printout, after Figure 36-16C.

36-17.0 INLET-CAPACITY CHART

Because of its frequency of usage by INDOT, Figure 36-17A provides a hydraulic-capacity chart for the curved vane grate, frame casting types 10 and 11. See the INDOT *Standard Drawings*. The inlet-capacity chart has been produced based on the following assumptions.

1. Grate dimensions: 36 in x 16 in
2. S_X = roadway slope = 0.02
3. S_W = gutter-pan slope = 0.025
4. W = gutter width = 2 ft
5. n = 0.016
6. S = longitudinal slope = 0.5% to 7%
7. Q = gutter flow = 0.35 ft^3/s to 8.48 ft^3/s

The assumed roadway conditions for S_X, S_W, and W are those that occur on a curbed facility. Figure 36-17A allows the user to determine the intercepted flow, Q_i, for a given longitudinal slope, S, and total gutter flow, Q. For example:

S = 1%
Q = 4.23 ft^3/s

Figure 36-17A yields Q_i = 1.87 ft^3/s.

36-18.0 REFERENCES

1. American Association of State Highway and Transportation Officials, Volume 9, *Highway Drainage Guidelines, Storm Drainage Systems*, 1992.

2. Federal Highway Administration, *Bridge Deck Drainage Guidelines*, FHWA Report No. RD-014, December 1986.

3. Federal Highway Administration, *Design of Bridge Deck Drainage,* Hydraulic Engineering Circular No. 21, 1993.

4. Federal Highway Administration, *Design of Urban Highway Drainage - The State of The Art*, FHWA-TS-79-225, 1979.

5. Federal Highway Administration, *Drainage of Highway Pavements*, Hydraulic Engineering Circular No. 12, 1984.

6. Federal Highway Administration, *Pavement and Geometric Design Criteria For Minimizing Hydroplaning*, FHWA Report No. RD-79-31, December 1979.

7. Dah-Chen Woo, *Bridge Drainage System Needs Criteria*, U.S. Department of Transportation, Public Roads, Vol.52, No. 2, September 1988.

Str.	Type	Casting Types												
		2	3	4	5	6	7	8	10	12	12A	13	14	15
Catch Basins	A	X	X					X						
	D					X								
	E						X							
	J								X					
	K								X					
	S												X	
	W [1]	X	X					X						
Inlets	A	X	X					X						
	B													X
	C													X
	D					X								
	E						X							
	F						X							
	G						X							
	H, HA				X									
	J								X					
	M								X					
	N									X				
	P										X			
	R											X		
	S												X	
	T												X	
Manholes	A	X		X				X						
	B	X		X				X						
	C [2]	X		X				X						
	D	X		X				X						
	E	X		X				X						
	F	X		X				X						
	G	X		X				X						
	H	X		X				X						
	J	X		X				X						
	K	X		X				X						
	L	X		X				X						
	M	X		X				X						
	N	X		X				X						

Notes: [1] May be substituted for catch basin type A.
[2] May be substituted for manhole type A or B.

COMPATIBILITY OF DRAINAGE STRUCTURES AND CASTINGS

Figure 36-2A

Symbol	Definition	Units
A	Area of cross section	ft^2
A	Watershed area	ac
a	Depth of depression	in.
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./h
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	lb/ft^2
Q	Rate of discharge in gutter	ft^3/s
Q_i	Intercepted flow	ft^3/s
Q_S	Gutter capacity above the depressed section	ft^3/s
Q_T	Total flow	ft^3/s
Q_W	Gutter capacity in the depressed section	ft^3/s
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
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/s
W	Width of depression for curb opening inlets	ft
W_d	Rotational velocity on dry surface	rpm
WD	Water depth	in.
W_W	Rotational velocity on flooded surface	rpm
y	Depth of flow in approach gutter	ft
Z	T/d, reciprocal of the cross slope	-

SYMBOLS AND DEFINITIONS

Figure 36-3A

Type of Facility	Design Frequency	Allowable Spread, *T*
Freeway	50 years	Edge of travel lane
Non-Freeway, ≥ 4 Lanes	10 years	Across one-half travel lane
Two-Lane Facility	10 years	4 ft onto travel lane
Bridge Deck, Non-Freeway		
V ≥ 50 mi/h	10 years	Edge of travel lane
V ≤ 50 mi/h	10 years	3 ft onto travel lane
Ramp, Roadway or Bridge		
V ≥ 50 mi/h	10 years	Edge of travel lane
V ≤ 50 mi/h	10 years	3 ft onto travel lane

DESIGN FREQUENCY AND ALLOWABLE WATER SPREAD

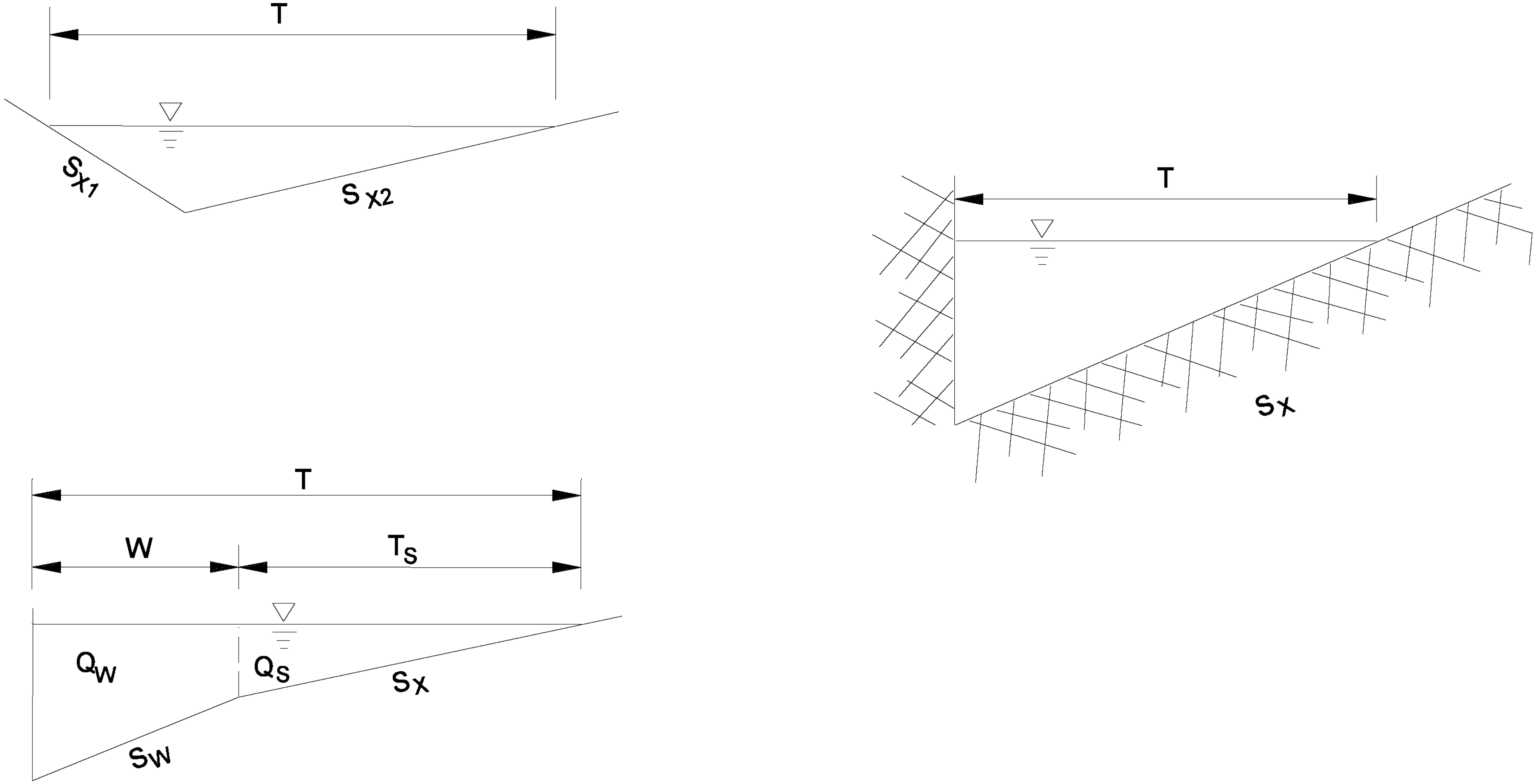
Figure 36-7A

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

Notes: 1. *For a gutter with a small slope where sediment may accumulate, increase* n *value by 0.002.*
2. *Reference: USDOT, FHWA, HDS-3 (1961)*

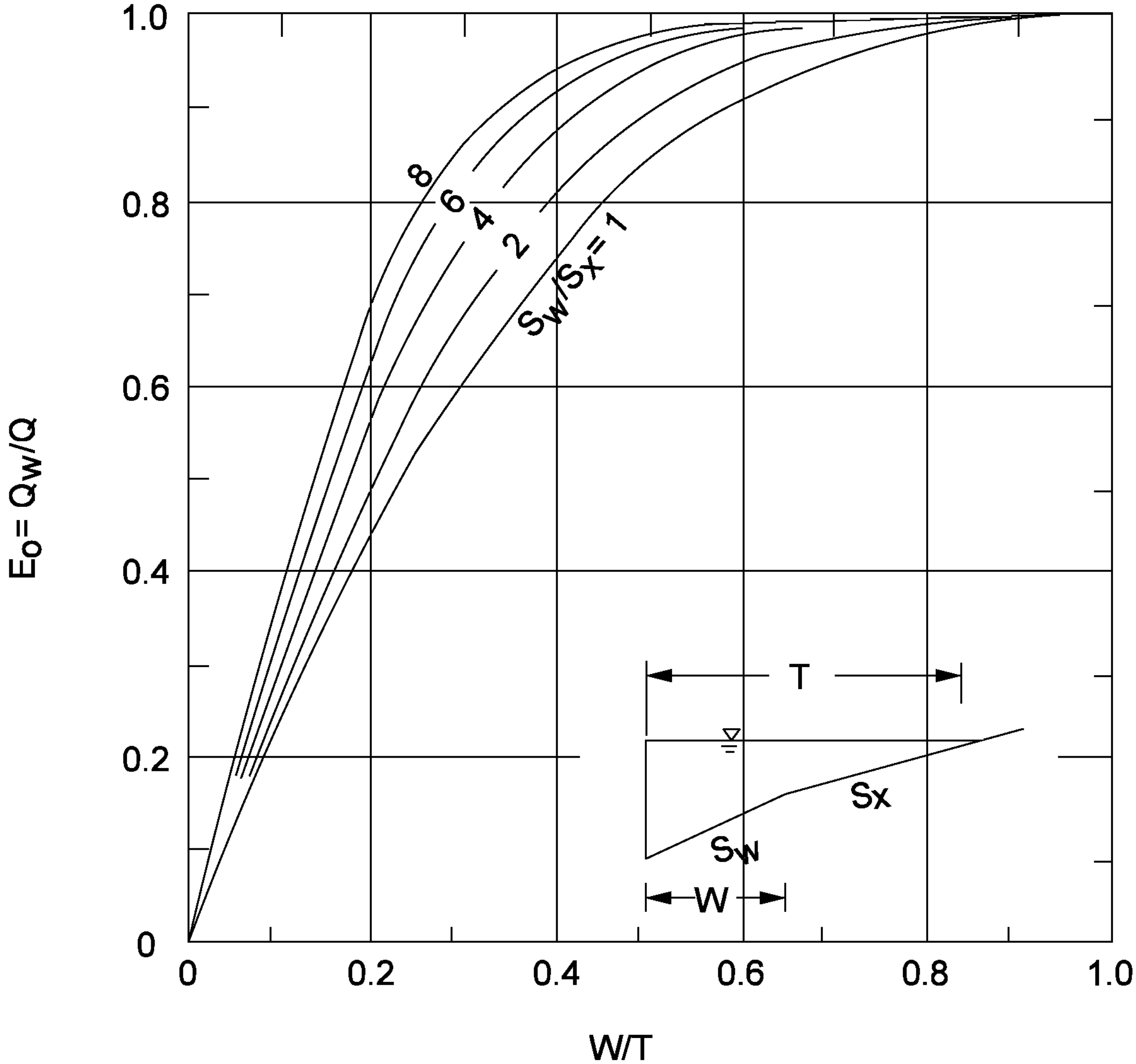
MANNING'S *n* FOR STREET OR PAVEMENT GUTTER

Figure 36-8A

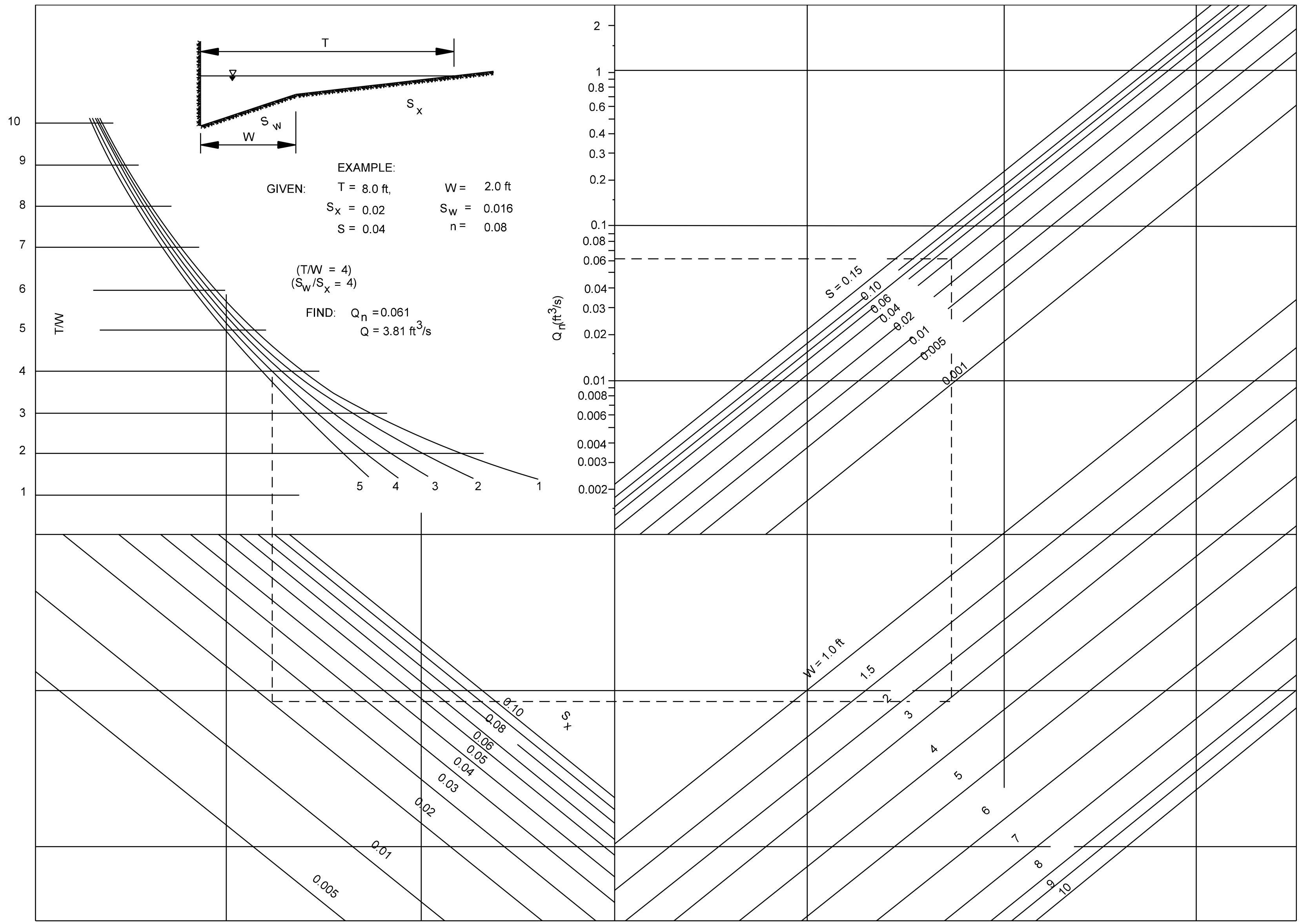


FLOW IN TRIANGULAR GUTTER SECTIONS

Figure 36-8B

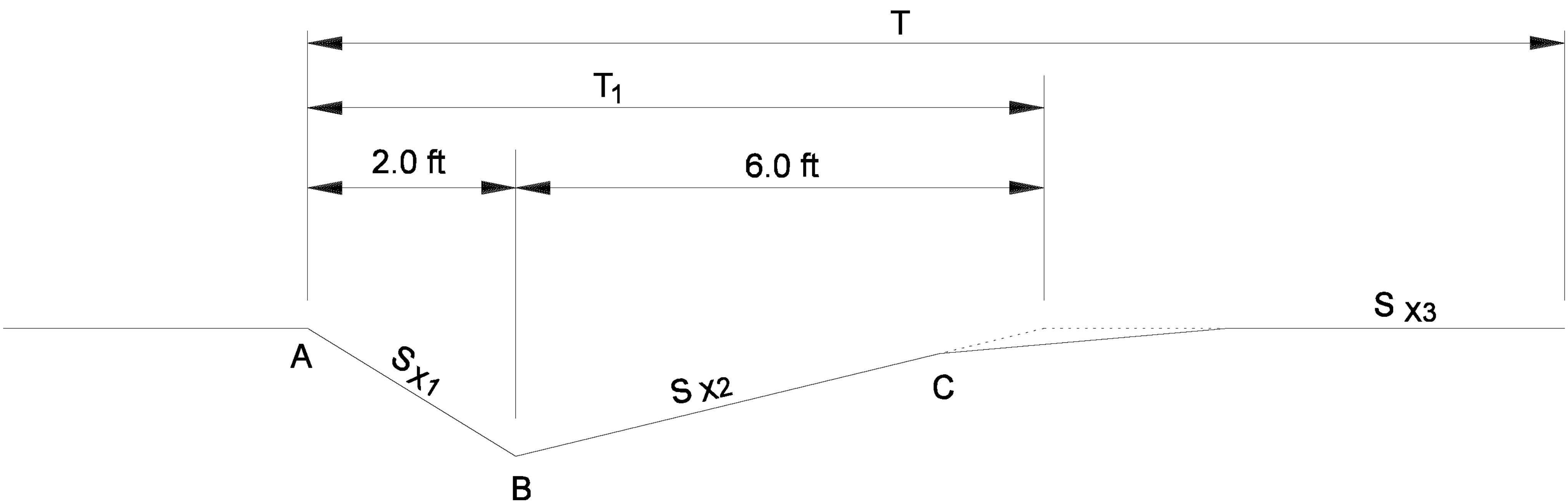


RATIO OF FRONTAL FLOW TO TOTAL GUTTER FLOW
Figure 36-8C



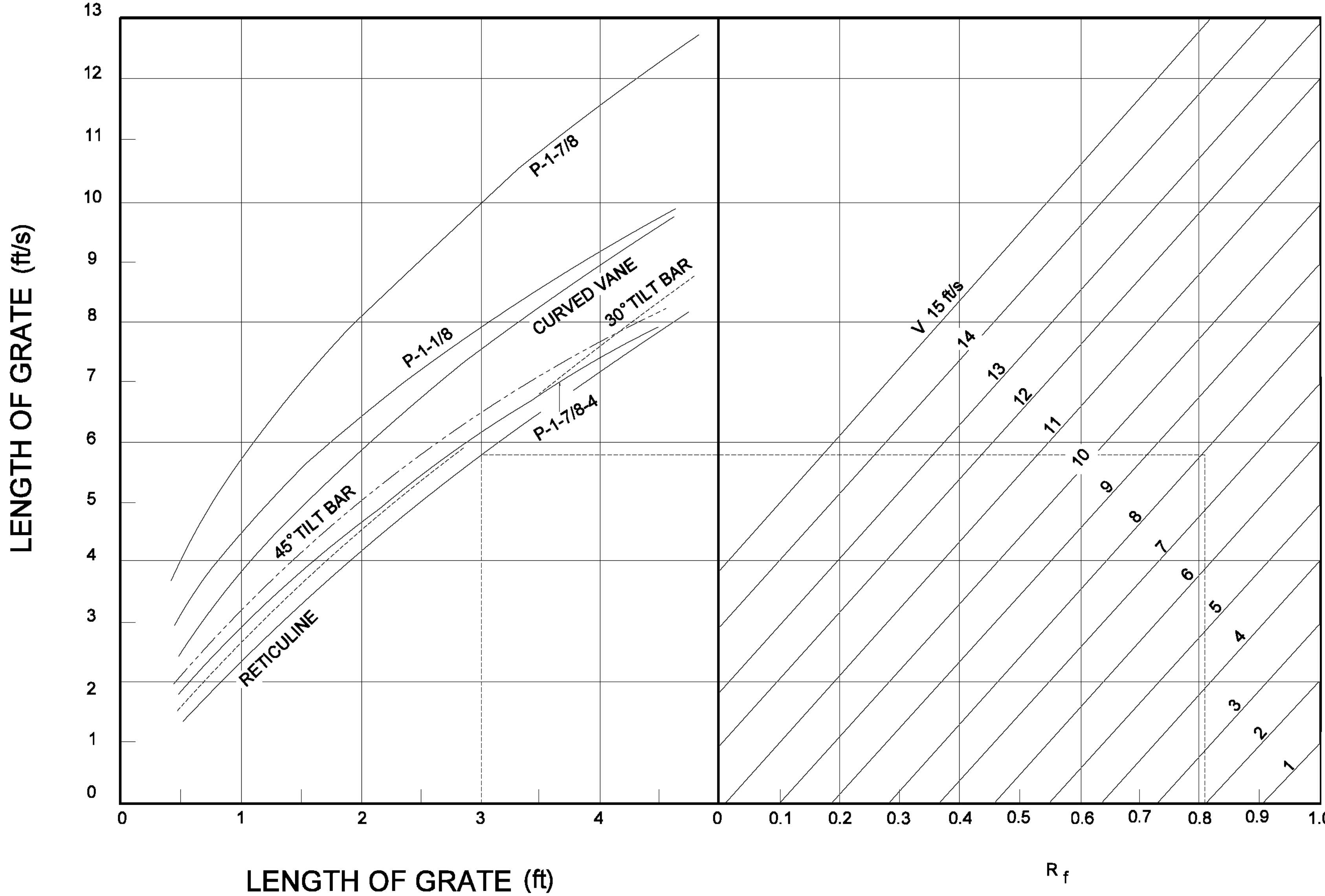
FLOW IN COMPOSITE GUTTER SECTIONS

Figure 36-8D



V-TYPE GUTTER

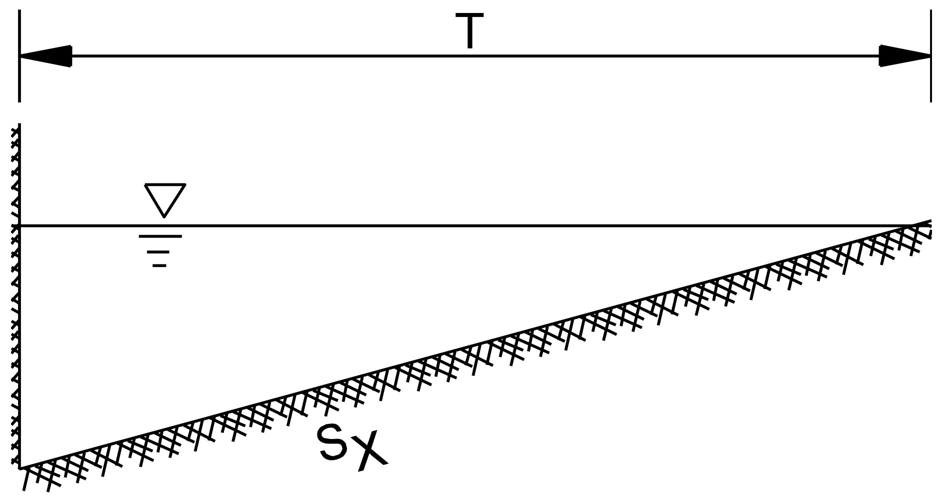
Figure 36-8E



Note: For grate types other that curved vane and reticuline. refer to the manufacture's data for nhydraulic characteristics.

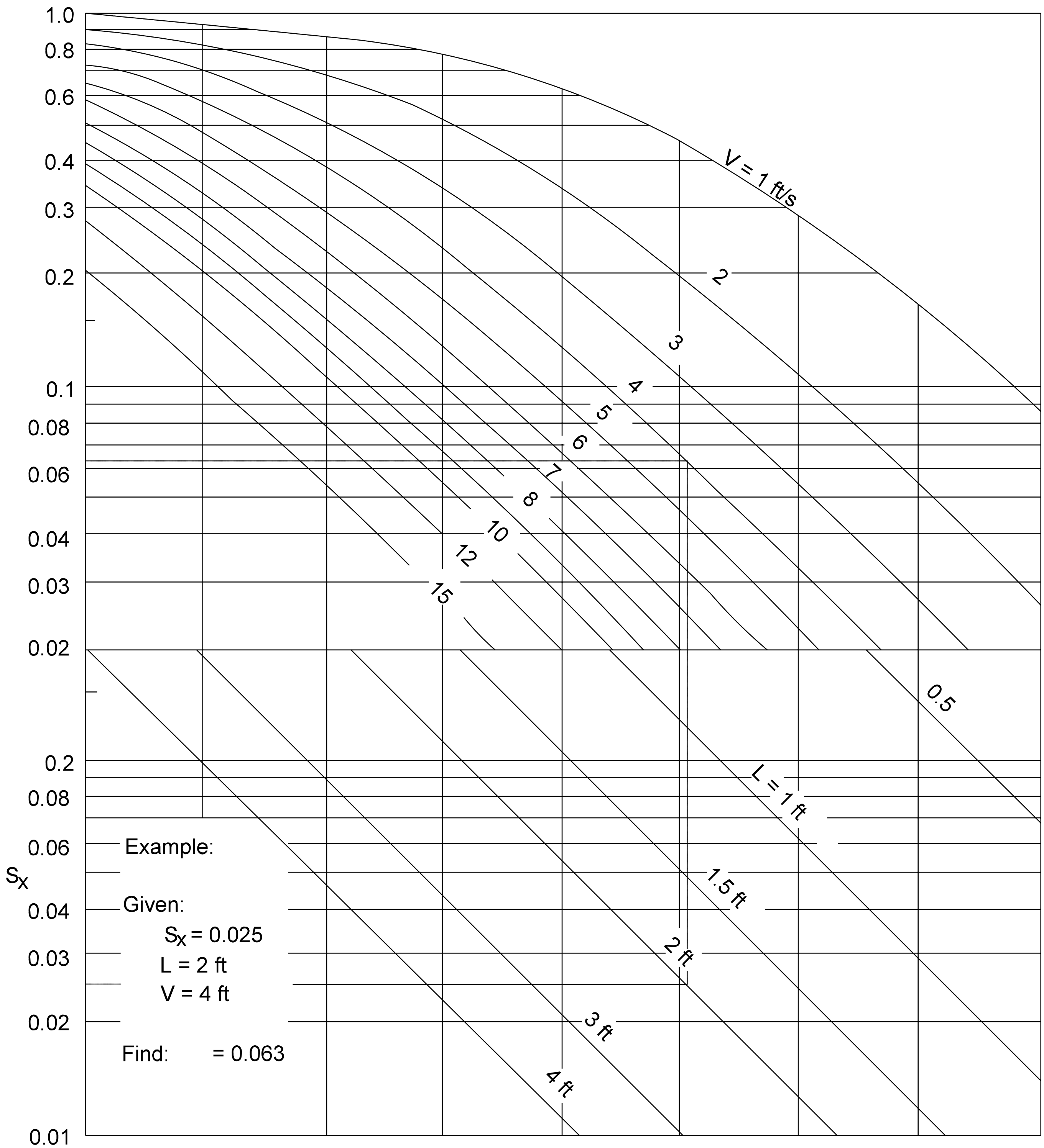
GRATE INLET FRONTAL FLOW INTERCEPTION EFFICIENCY

Figure 36-10A



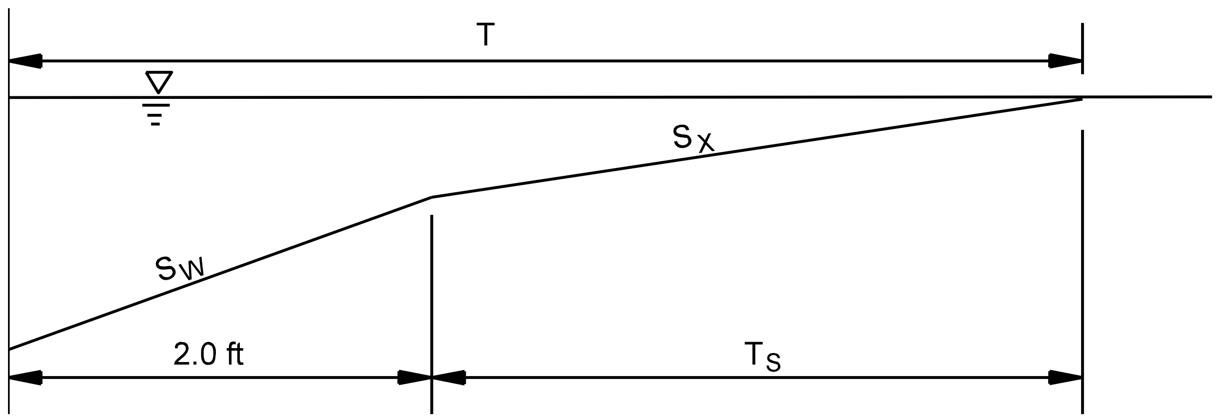
GUTTER CROSS SECTION
(Equation 36-10.5)

Figure 36-10B



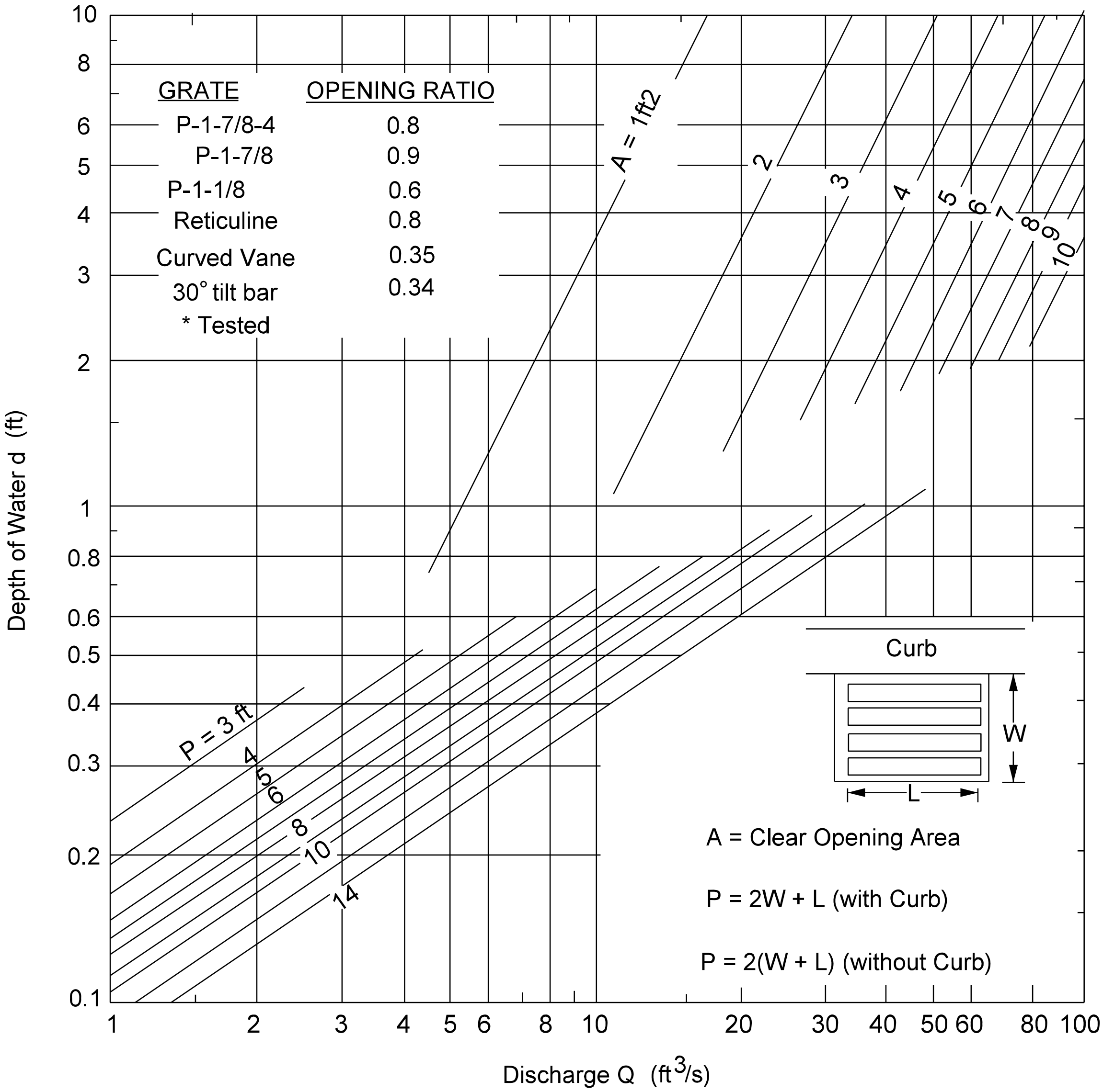
GRATE INLET SIDE FLOW INTERCEPTION EFFICIENCY

Figure 36-10C



EXAMPLE PROBLEM SKETCH

Figure 36-10C(1)



GRATE INLET CAPACITY IN SUMP CONDITIONS

Figure 36-10D

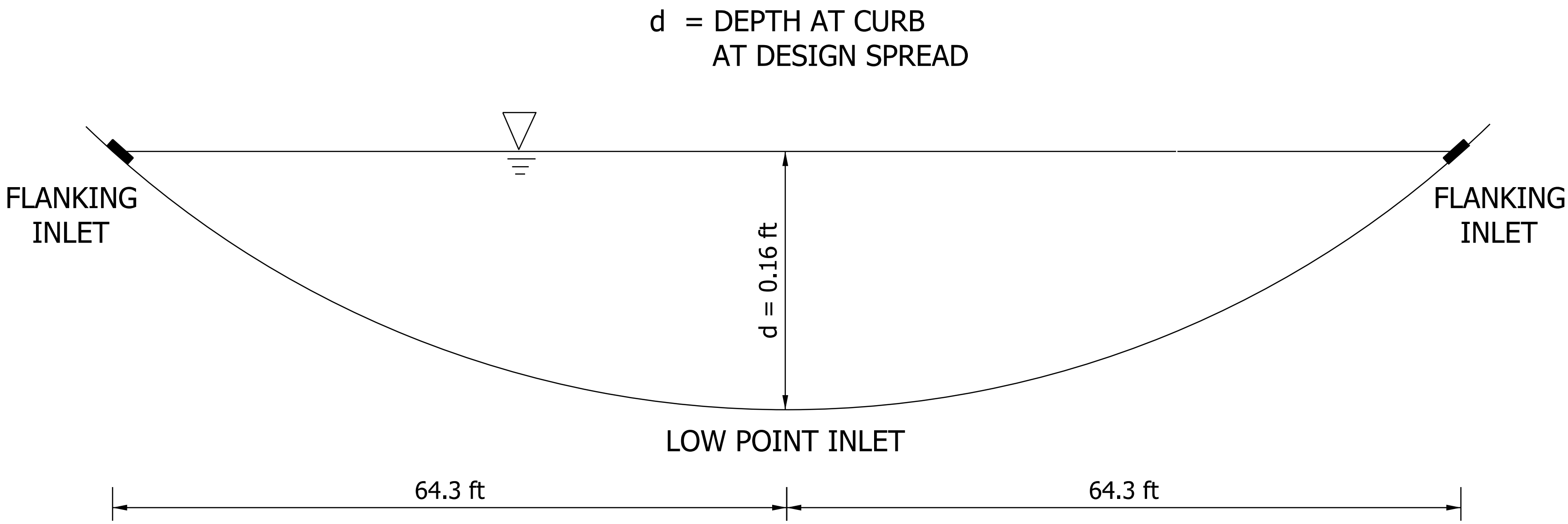
Speed (mph)	20	25	30	35	40	45	50	55	60	62	65	70
d↓ *K*→	20	30	40	50	70	90	110	130	160	167	180	220
0.1	20	24	28	32	37	42	47	51	57	58	60	66
0.2	28	35	40	45	53	60	66	72	80	82	85	94
0.3	35	42	49	55	65	73	81	88	98	100	104	115
0.4	40	49	57	63	75	85	94	102	113	116	120	133
0.5	45	55	63	71	84	95	105	114	126	129	134	148
0.6	49	60	69	77	92	104	115	125	139	142	147	162
0.7	53	65	75	84	99	112	124	135	150	153	159	176
0.8	57	69	80	89	106	120	133	144	160	163	170	188

Notes:

1. $x = (200dK)^{0.5}$, where x = distance from the low point to flanking inlet, ft, and
   d = depth at curb, ft
   Maximum K for drainage = 170 (ft/%A) for a curbed facility.
2. $K = L/A$, where L = length of vertical curve, ft, and A = algebraic difference in approach grades, %.
   Reference: *HEC 12* (modified).
3. See Figure 36-10E(1) for Example 36-10.3 details.

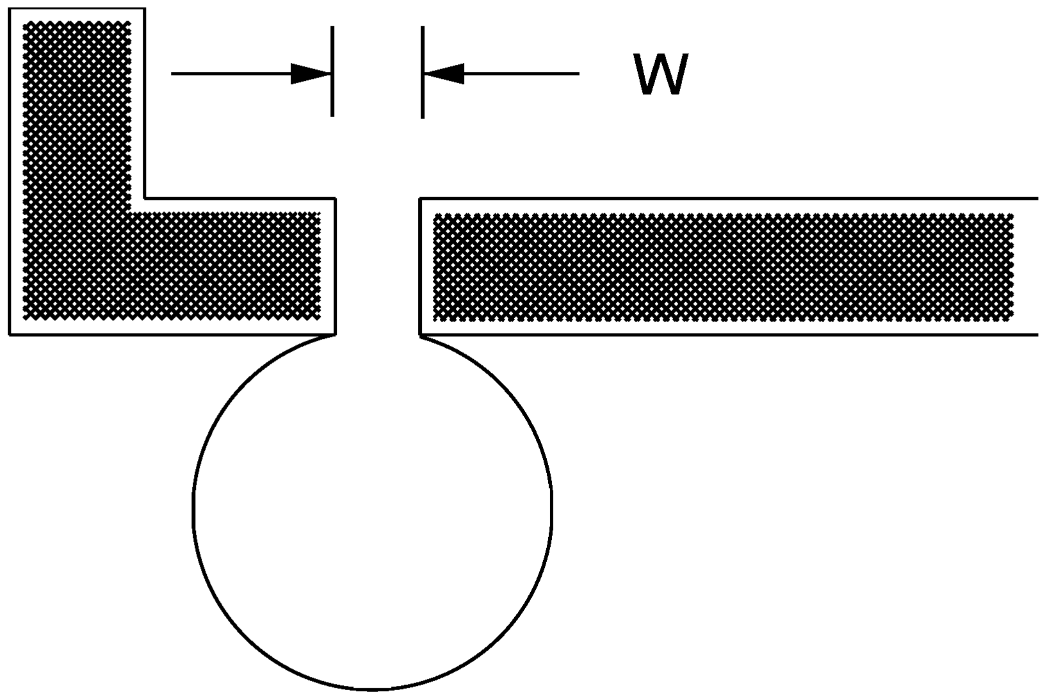
FLANKING-INLET LOCATIONS

Figure 36-10E



EXAMPLE PROBLEM 36-10.3
FLANKING INLETS AT SAG POINT

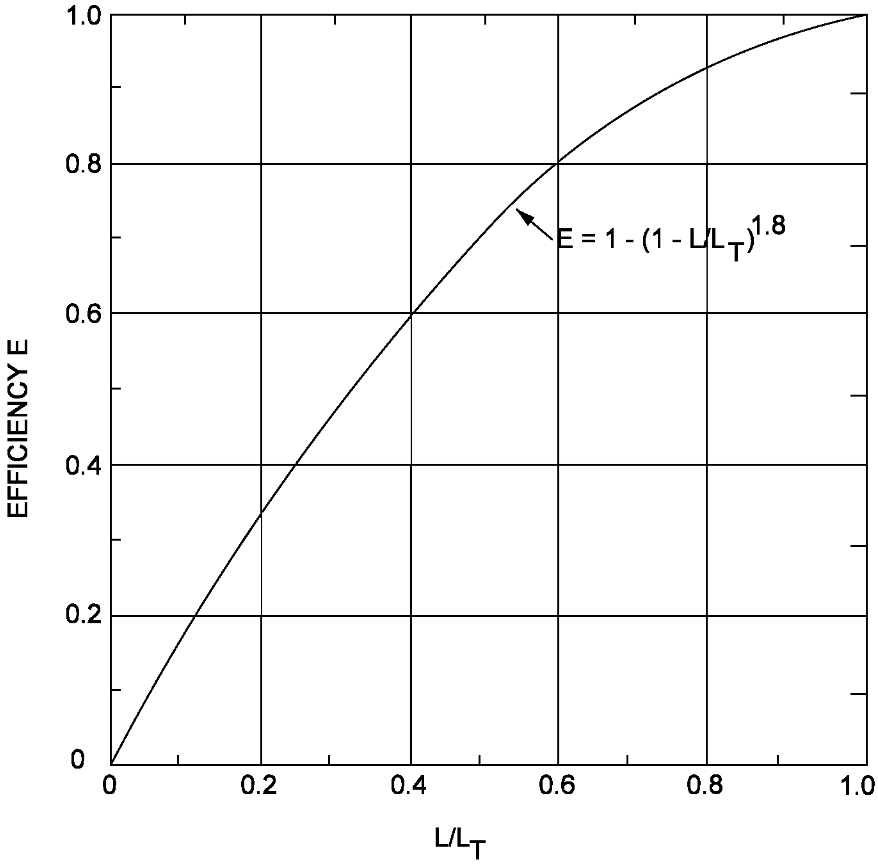
Figure 36-10E(1)



L = Length of Slotted Drain Inlet

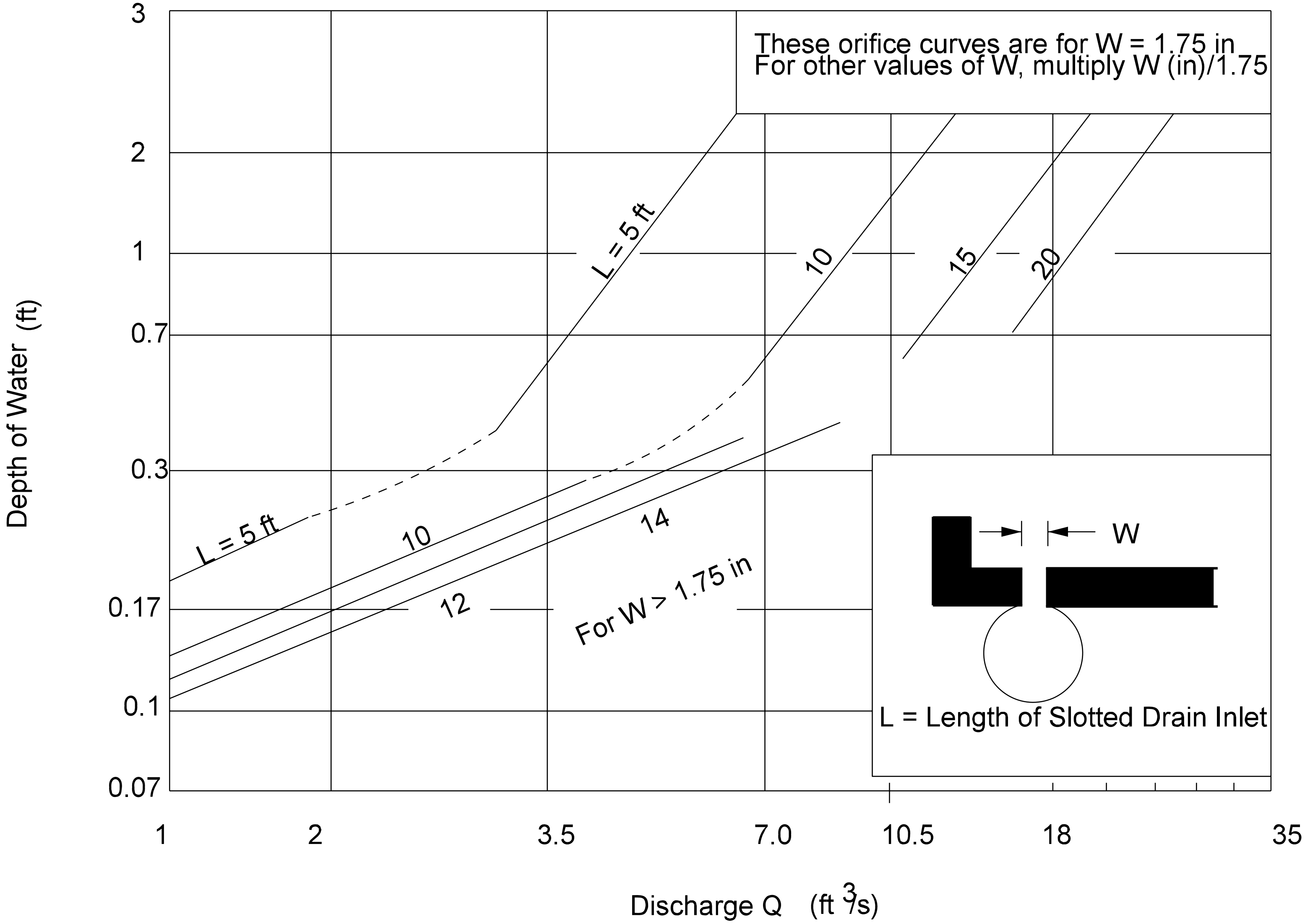
GUTTER CROSS SECTION
(Equation 36-10.11)

Figure 36-10H



CURB-OPENING AND SLOTTED DRAIN INLET INTERCEPTION EFFICIENCY

Figure 36-10I



SLOTTED DRAIN INLET CAPACITY IN SUMP LOCATIONS

Figure 36-10J

ROUTE: __________ PROJECT: __________________________ COMPUTED BY: __________________________ DATE: ___________ SHEET ____ OF ____

LOCATION		GUTTER DISCHARGE Design Frequency: _________					GUTTER DISCHARGE Allowable Spread: _________								INLET DISCHARGE			REMARKS
Inlet No.	Station	Drain Area A (ac)	Run-off Coef. C	Time of Conc. T_C (min.)	Rain Inten-sity, I (in./h)	Q = .002–78 x CIA (cfs)	Grade S_o (ft/ft)	Cross Slope S_x (ft/ft)	Prev. Run-by (cfs)	Total Gutt. Flow (cfs)	Depth d T/W (ft)	Gutt. Width W (ft)	Sprd. T (ft)	W/T	Inlet Type	Inter-cept Q_i (cfs)	Run-by Q_r (cfs)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19

INLET SPACING COMPUTATION SHEET

Manhole Type	Manhole Inside-Dia. Dimension (in.)	Maximum Trunkline Pipe Size (in.)	Minimum Trunkline Pipe Size (in.)
A	48 dia	24	12
B	36 dia	18	12
C	49 dia	24	12
D	58 x 74	42	27
E	80 x 74	60	48
F	108 x 74	84	66
G	136 x 74	108	90
H	49 dia	36	24
J	62 dia	36	24
K	74 dia	48	36
L	98 dia	54	48
M	104 dia	72	54
N	110 dia	84	72

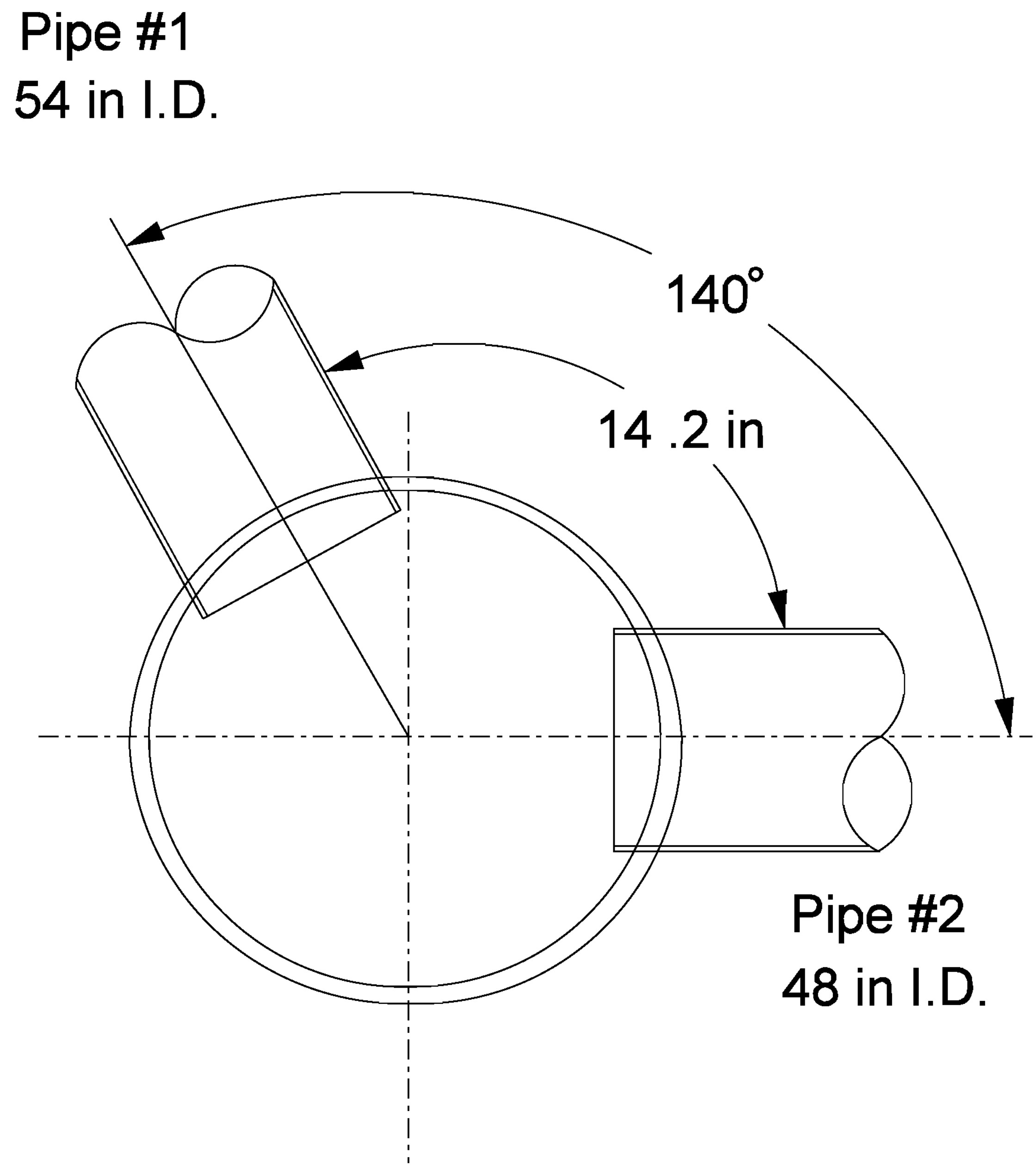
MANHOLE TYPES

Figure 36-11A

MH Dia. (in.)	*K* (in./deg)	Maximum Pipe Size (in.)
27	0.24	15
42	0.38	27
48	0.43	30
54	0.48	36
60	0.53	42
66	0.59	48
72	0.64	54
78	0.69	60
84	0.74	66
90	0.80	72
96	0.85	72
102	0.90	78
108	0.96	84

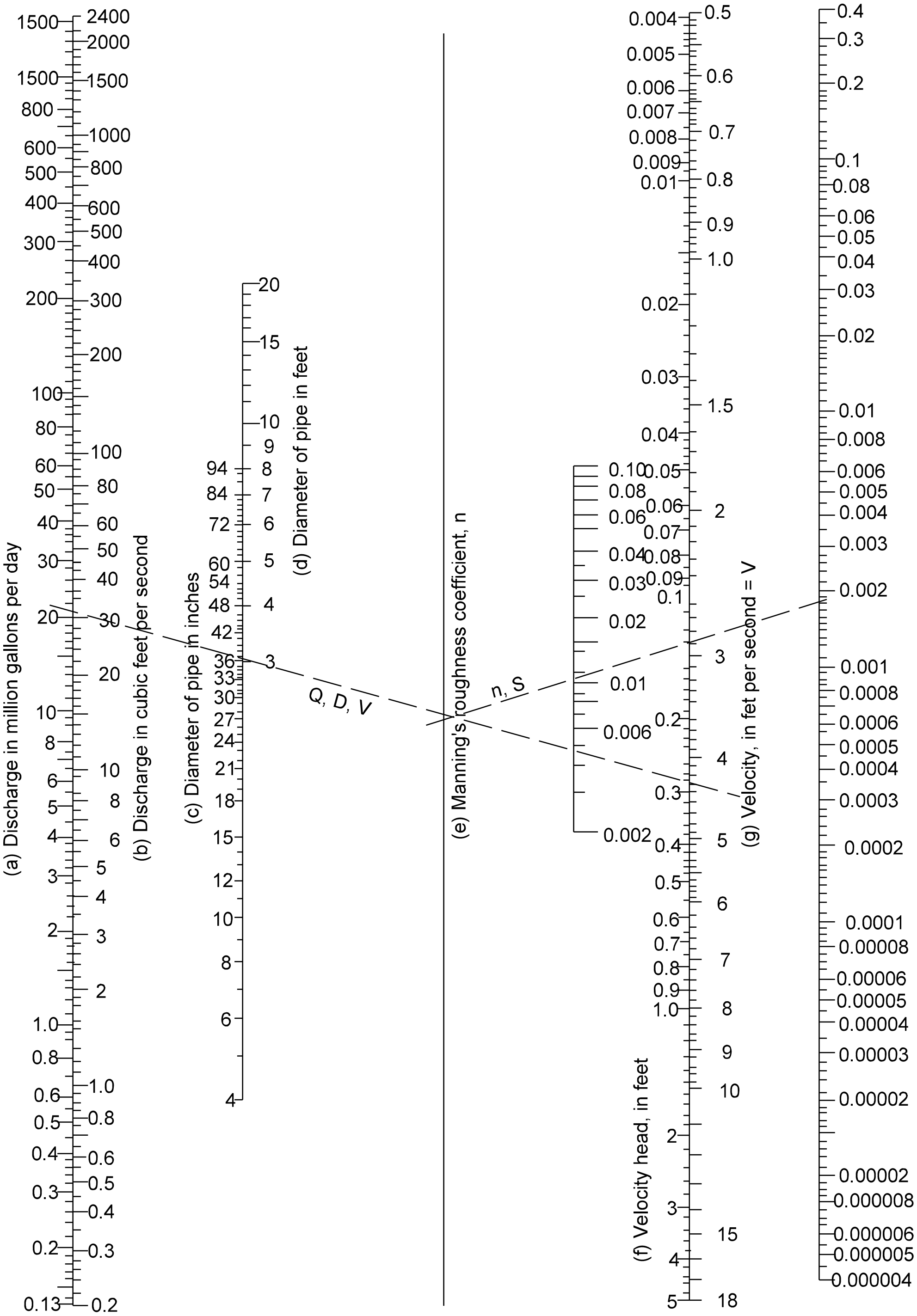
MANHOLE SIZING

Figure 36-11B



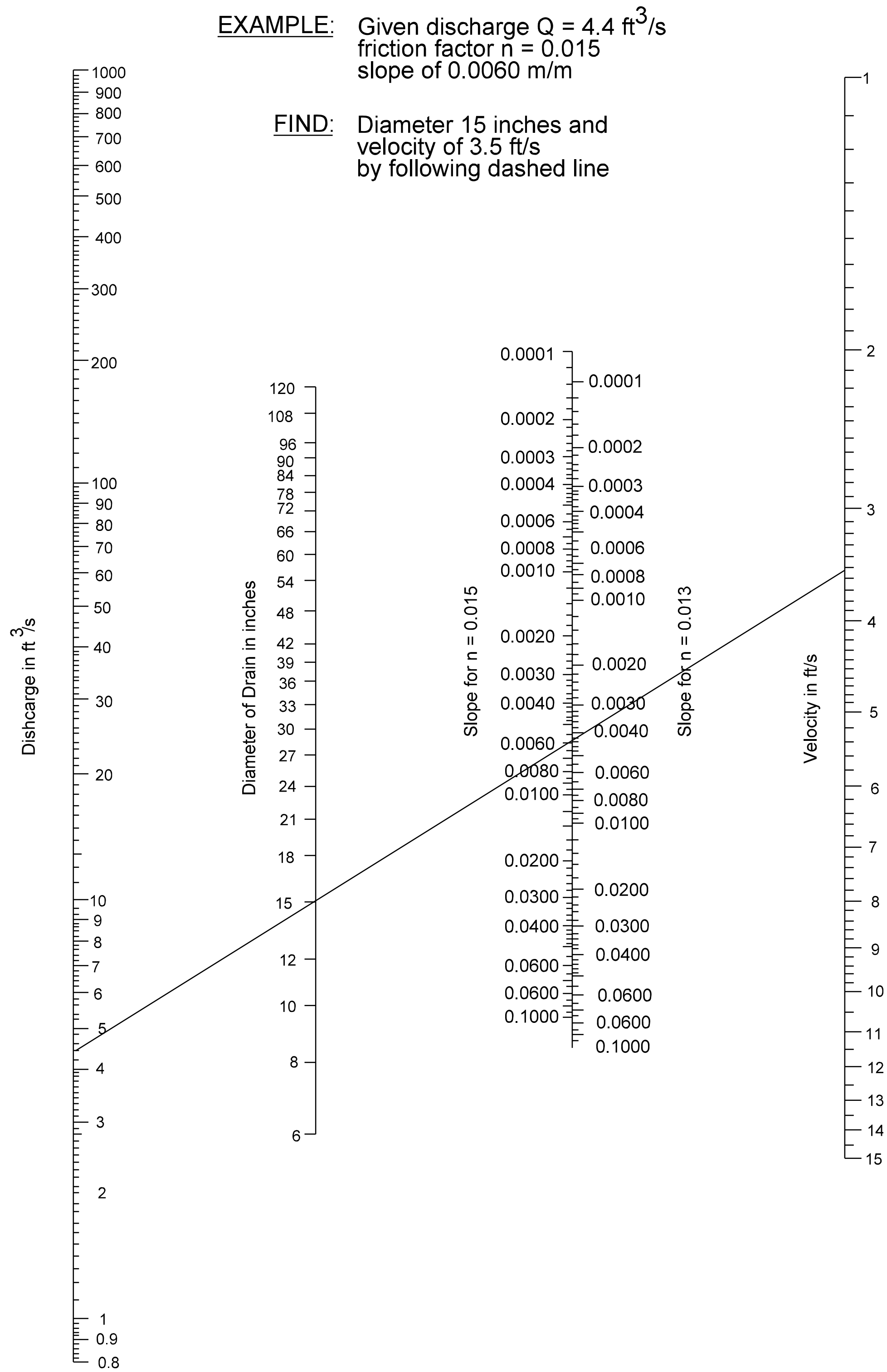
PIPE LAYOUT FOR EXAMPLE PROBLEM 36-11.1

Figure 36-11C



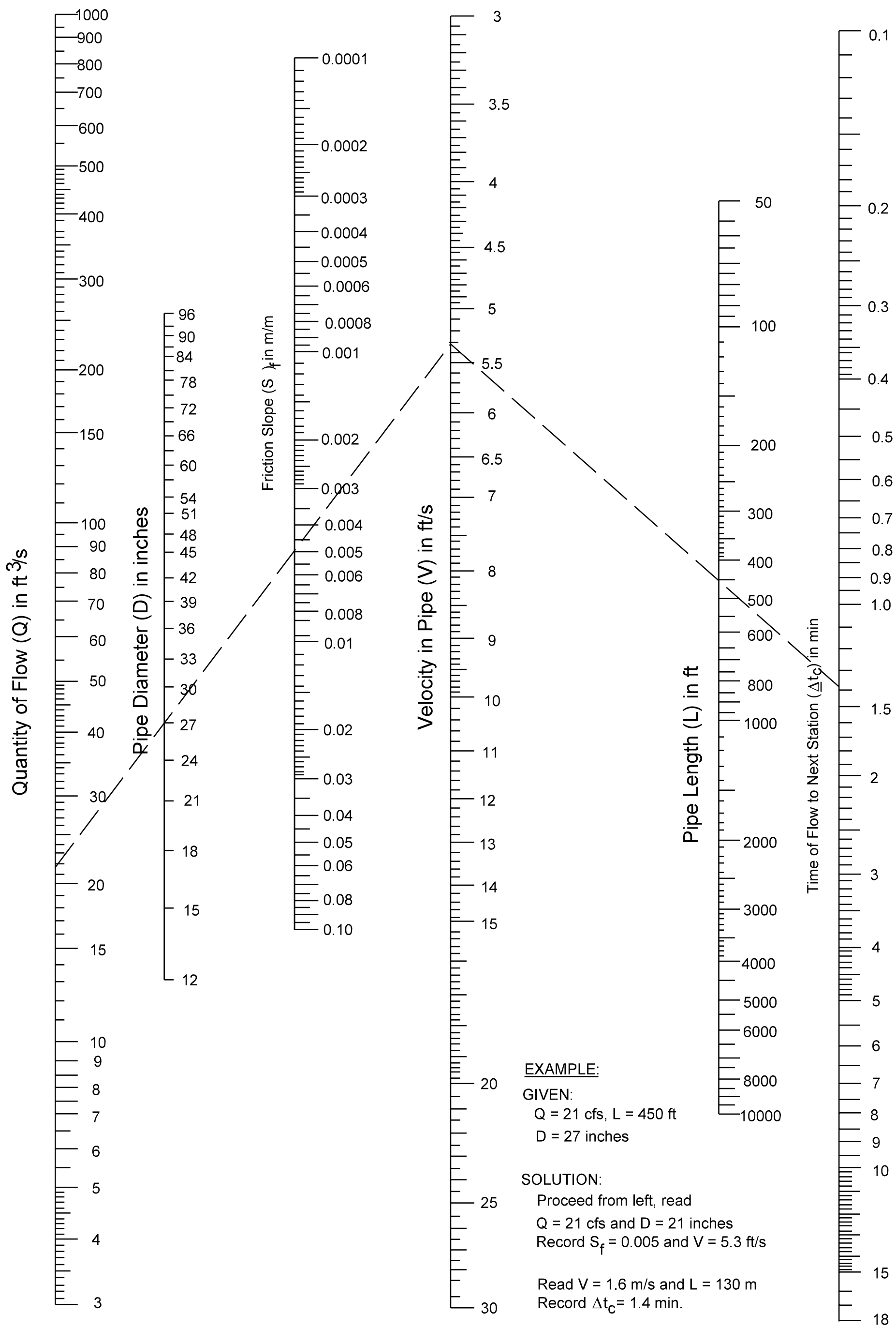
MANNINGS FORMULA FOR FLOW IN STORM DRAINS

Figure 36-12A



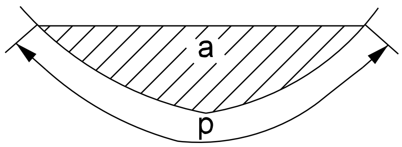
NOMOGRAPH FOR COMPUTING REQUIRED SIZE
OF CIRCULAR DRAIN FOR FULL FLOW
(n = 0.013 or n = 0.015)

Figure 36-12B

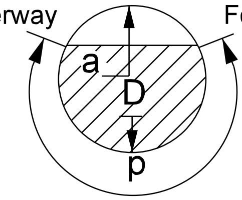


CONCRETE PIPE FLOW NOMOGRAPH

Figure 36-12C



A = Cross-sectional area of waterway
P = wetted perimeter
R = A/P = Hydraulic Radius



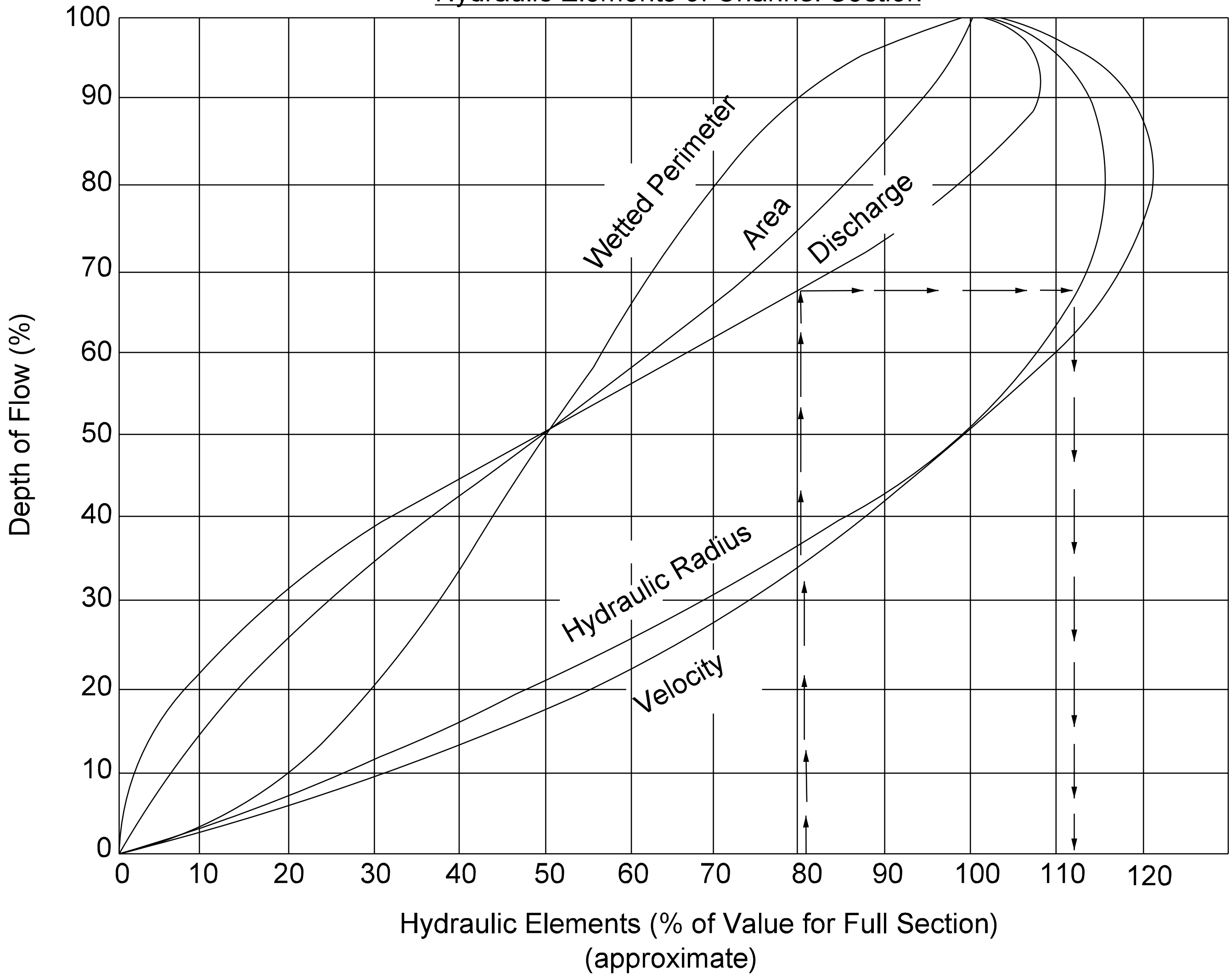
For pipes full or half full
R = D/4

Section of Any Channel

Section of Circular Pipe

V = Average or mean velocity in ft/s
Q =AV = Discharge of pipe or channel in ft^3/s
n = Coefficient of roughness of pipe or channel surface
S = Slope of hydraulic gradient (water surface in open channels or pipes not under pressure, same as slope of channel or pipe invert only when flow is uniform in constant section)

Hydraulic Elements of Channel Section



VALUES OF HYDRAULIC ELEMENTS OF CIRCULAR SECTION
FOR VARIOUS DEPTHS OF FLOW

Figure 36-12D

Pipe Size, in	Full-Pipe Flow, ft^3/s	Slope *
12	2.12	0.0030
15	3.14	0.0022
18	4.48	0.0017
21	6.14	0.0014
24	8.12	0.0012
27	10.17	0.0010
30	12.57	0.00087
33	15.11	0.00076
36	18.05	0.00068
42	24.47	0.00055
48	31.96	0.00046
54	40.29	0.00039
60	49.86	0.00034
66	60.39	0.00030
72	72.25	0.00027

* n = *0.012, coefficient of pipe roughness assumed by INDOT for storm drain*

**MINIMUM SLOPE NECESSARY TO ENSURE 3.0 ft/s
IN STORM DRAIN FLOWING FULL**

Figure 36-12E

Route __________ Section ____________ County ______________________
Computed ____________________ Date ____________
Checked _____________________ Date ____________

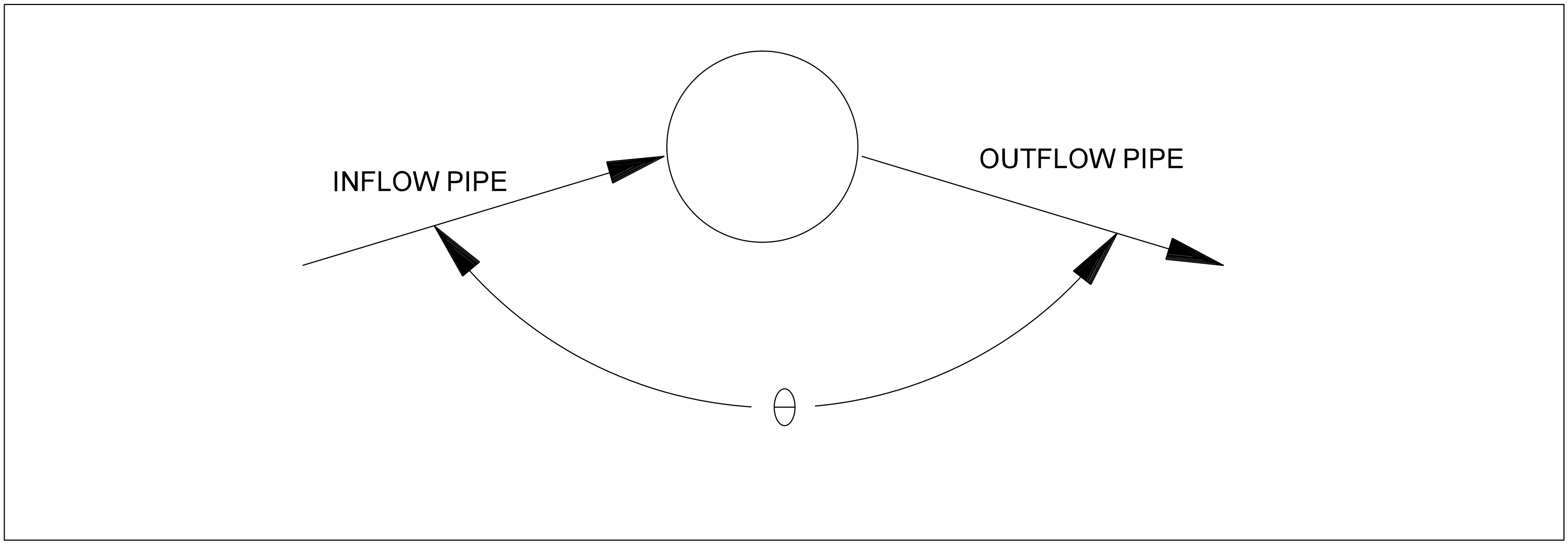
Station	Length (ft)	Drainage Area A (ac)		Runoff Coefficient C	(A)(C)		Flow Time (min)		Rainfall Intensity I (in./h)	Total Runoff 0.00278CIA = Q (cfs)	Diameter Pipe (in.)	Capacity Fill (cfs)	Velocity (ft/s)		Invert Elev.		Manhole Invert Drop	Slope of Drain (ft/ft)
		Increment	Total		Increment	Total	To Upper End	In Section					Flowing Full	Design Flow	Upper End	Lower End		

STORM DRAIN COMPUTATION SHEET

AREA RATIO	FREQUENCY FOR COINCIDENTAL OCCURRENCE			
	10-Year Design		100-Year Design	
	MAIN STREAM	TRIBUTARY	MAIN STREAM	TRIBUTARY
10 000 to 1	1 10	10 1	2 100	100 2
1000 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

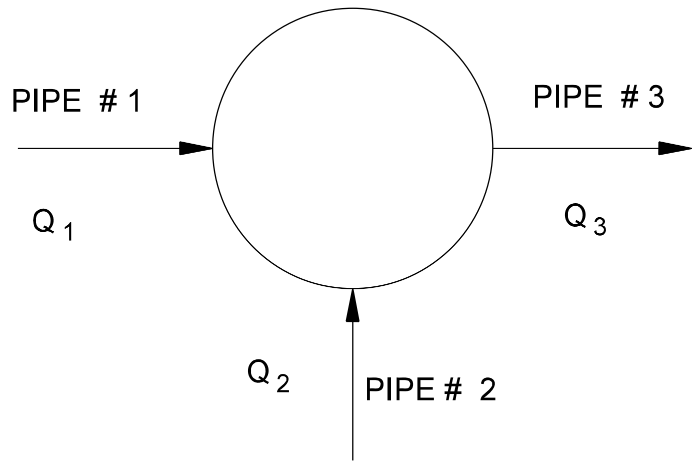
JOINT PROBABILITY ANALYSIS

Figure 36-13A



DEFLECTION ANGLE

Figure 36-13B



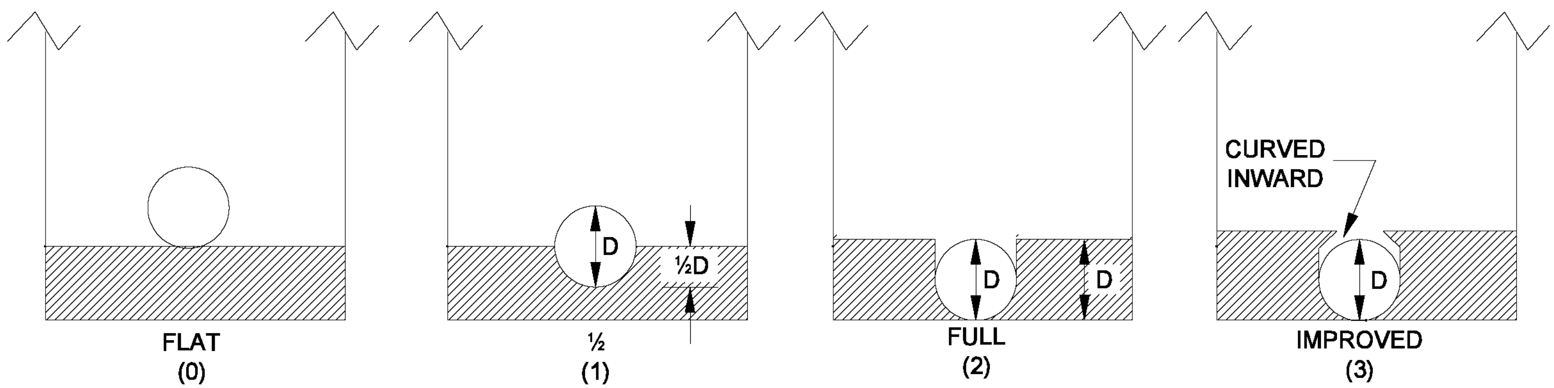
RELATIVE FLOW EFFECT

Figure 36-13B(1)

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, $d/D_0 > 3.2$

**free surface flow, $d/D_0 < 1.0$



SCHEMATIC REPRESENTATION OF BENCHING TYPES

CORRECTION FOR BENCHING
Figure 36-13C

Station	TW	D_O	Q_O	L_O	V_O	$V_O^2/2g$	H_O	Sf_O	H_f	K_O	C_D	C_d	C_Q	C_p	C_B	K	$K(V_O^2/2g)$	EGL_O 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)

HYDRAULIC GRADE LINE COMPUTATION FORM

$$H_{tm} = \frac{V^2}{2g}$$

TERMINAL JUNCTION LOSSES
(at beginning of run)

Where: g = gravitational constant, 32.2 ft/s^2

$$H_e = 0.5\frac{V^2}{2g}$$

ENTRANCE LOSSES
(for structure at end of run)
Assuming square - edge

$$H_{j1} = \frac{V^2}{2g}\text{ (Outflow)}$$

JUNCTION LOSSES

Use only where flows are identical to above; otherwise, use H_{j2} Equation.

$$H_{j2} = \frac{Q_4 V_4^2 - Q_1 V_1^2 - Q_2 V_2^2 + KQ_1 V_1^2}{2gQ_4}$$

JUNCTION LOSSES
(After FHWA)

Total losses to include H_{j2} plus losses for changes in direction of less than 90º (H_b).

Where:	K	=	Bend loss factor
	Q_3	=	Vertical dropped-in flow from an inlet
	V_3	=	Assumed to be zero

$$H_b = \frac{KV_1^2}{2g}$$

BEND LOSSES
(Change in direction of flow)

Where K	Degree of Turn (A) in Junction
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90

FRICTION LOSS (H_f)

$H_f = (S_f)\ (L)$

Where:	H_f	=	Friction head, ft
	S_f	=	Friction slope, ft/ft
	L	=	Length of conduit, ft

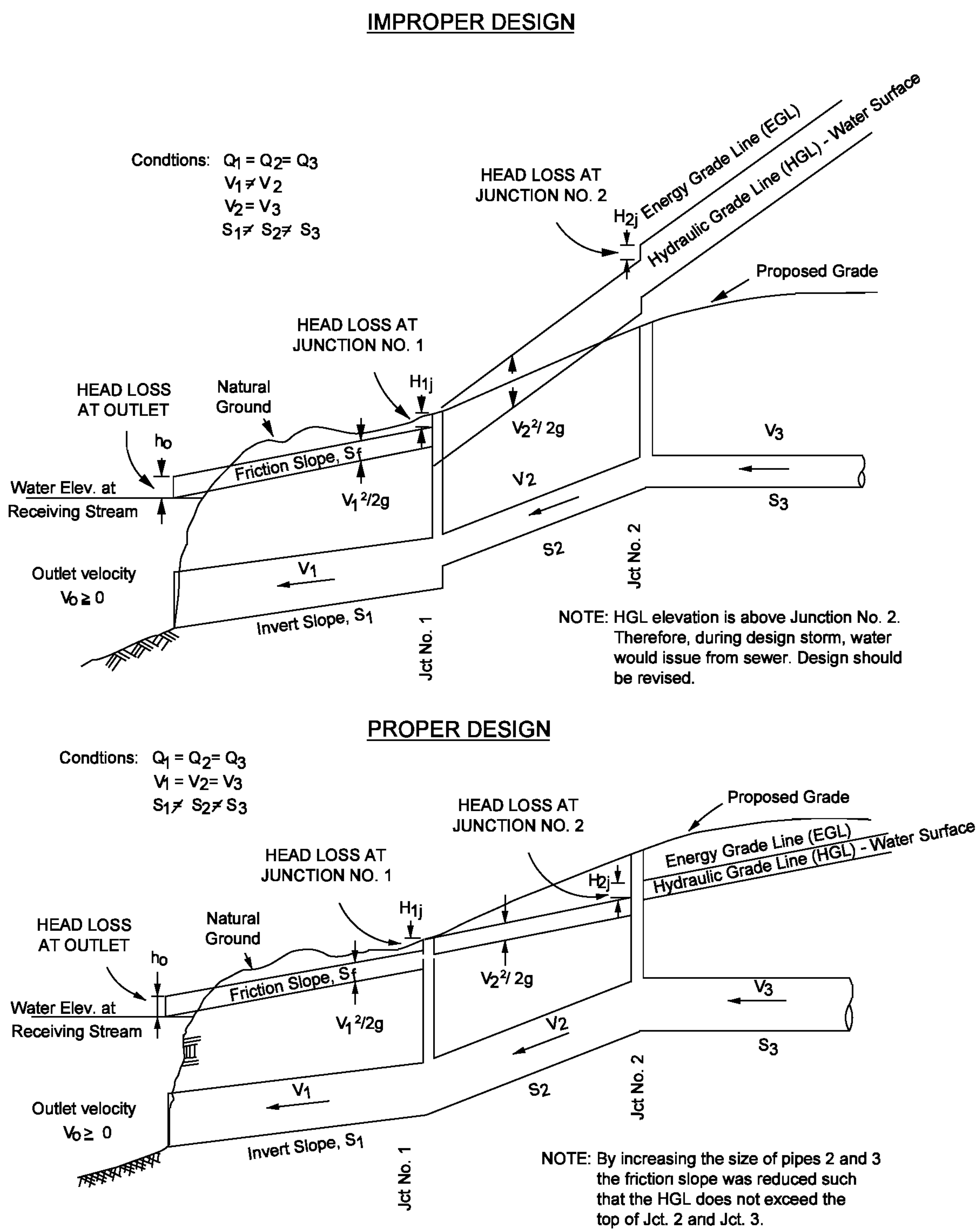
$$S_f = \left(\frac{Qn}{AR^{2/3}}\right)^2$$

Where:	Q	=	Discharge of conduit, ft^3/s
	n	=	Manning's coefficient of roughness (use 0.013 for R.C. Pipes)
	A	=	Area of conduit, ft^2
	R	=	Hydraulic radius of conduit (D/4 for round pipe), ft

TOTAL ENERGY LOSSES AT EACH JUNCTION

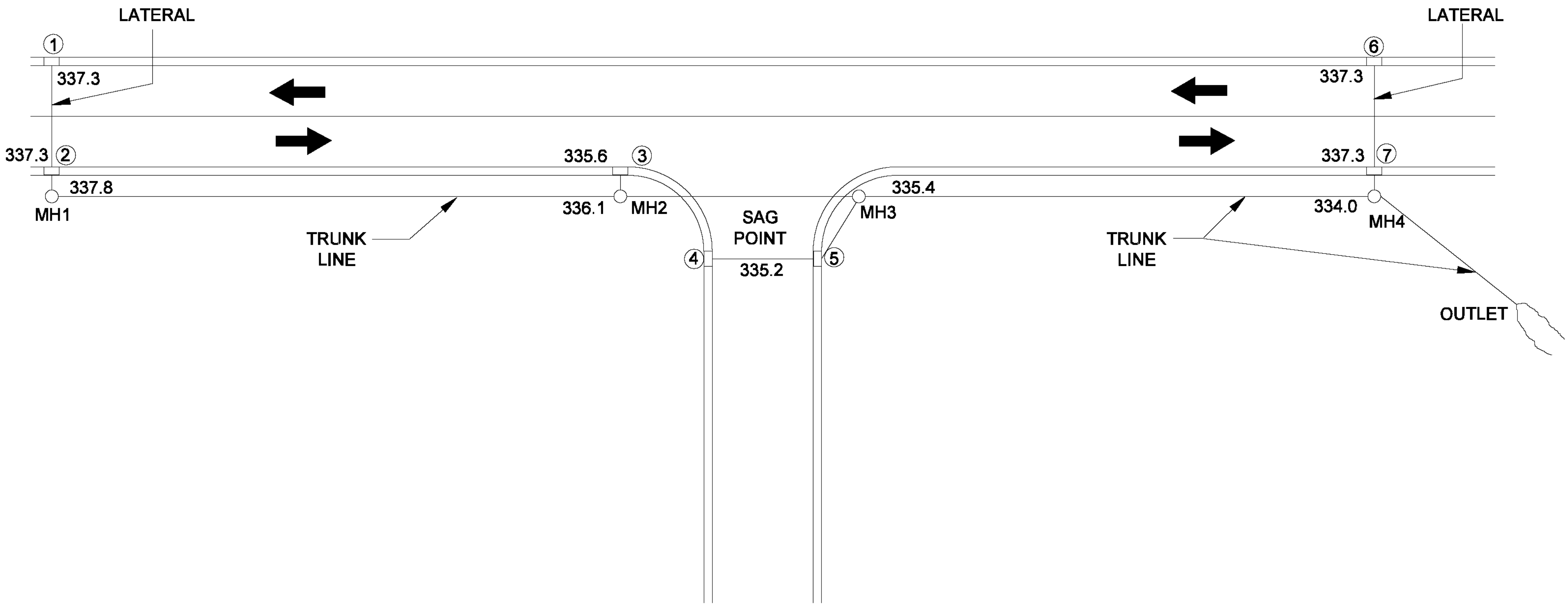
$$H_T = H_{tm} + He + (H_{j1} \text{ or } H_{j2}) + H_b + H_f$$

SUMMARY OF ENERGY LOSSES
Figure 36-13E



USE OF ENERGY LOSSES IN DEVELOPING A STORM DRAIN SYSTEM

Figure 36-13F



EXAMPLE PROBLEM

Figure 36-16A

INLET COMPUTATION SHEET

DATE_________PROJECT_________ROUTE______
COMPUTED BY________________SHEET_____OF____

LOCATION		GUTTER DISCHARGE DESIGN FREQUENCY 10					GUTTER DISCHARGE ALLOWABLE SPEED 6.0 ft (Roadway) 7.0 ft (Street)								INLET DISCHARGE			RE-MARKS
INLET No.	STAT.	DRAIN AREA "A" (acres)	RUNOFF COEF "C"	TIME OF C ONC. "T_C" (min)	Rain Intensity "I" (in/h)	Q=.002-78CIA (ft^3/s)	GRADE "S_O" (ft/ft)	CROSS SLOPE S_x (ft/ft)	PREV. RUNBY (ft^3/s)	TOTAL GUTTER FLOW (ft^3/s)	DEPTH "d" T/W (ft)	GUTTER WIDTH "W" (ft)	SPREAD "T" (ft)	W/T	INLET TYPE	INTER-CEPT "Q_i" (ft^3/s)	RUNBY "Q_r" (ft^3/s)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
1	10+00	0.125 0.100	0.4 0.9	10	5.28	0.706	0.012	0.02	0	0.706	0.133	2.00	6.00	0.33	10 & 11	0.530	0.177	
2	10+00	0.125 0.100	0.4 0.9	10	5.28	0.706	0.012	0.02	0	0.706	0.133	2.00	6.00	0.33	10 & 11	0.530	0.177	
3	10+42.7	0.050 0.150	0.9 0.4	11.2	5.08	0.530	0.012	0.02	0.177	0.706	0.133	2.00	6.00	0.33	10 & 11	0.530	0.177	
4	Street	0.100 0.125	0.9 0.9	7	6.00	1.200	Sag	0.02	0	1.200	0.140	1.32	6.67		Double 10 & 11	1.200	0	
5	Street	0.100 0.100	0.9 0.9	7	6.00	1.059	Sag	0.02	0.177	1.236	0.143	1.32	6.67		Double 10 & 11	1.236	0	Sag Point
6	11+00	0.100	0.9	11.5	5.04	0.424	0.012	0.02	0.177	0.600	0.123	1.32	5.73		10 & 11	0.459	0.141	Sag Point
7	11+00	0.050 0.250	0.9 0.4	11.5	5.04	0.706	0.012	0.02	0	0.706	0.133	1.32	6.00		10 & 11	0.530	0.177	

INLET SPACING COMPUTATION SHEET
(Example Problem)
Figure 36-16B

```
***************************************
*****     ROADWAY DRAINAGE DESIGN     *****
***************************************
```

DESIGNER: INDOT

DATE: 09-02-1998

PROJECT: Example

PROJECT NO.:

INLET NO.: 1

STATION: 10+00

DRAINAGE AREA: .09 Hectares

DESIGN FREQUENCY: 10 Years

ROADWAY & DISCHARGE DATA

Cross-Slope	S (m/m)	Sx (m/m)	n	Q (m^3/s)	T (m)
Composite	0.0120	0.0200	0.016	0.020	1.827

GUTTER FLOW

W (m)	Sw (m/m)	a (mm)	Eo	d (mm)	V (m/s)
0.397	0.0250	N/A	0.679	39.611	0.582

INLET INTERCEPTION

Inlet Type	L (m)	W (m)	E	Qi (m^3/s)	Qb (m^3/s)
Curved Vane	0.879	0.397	0.782	0.015	0.005

HIGHWAY DRAINAGE DESIGN EXAMPLE

Figure 36-16B(1)

```
****************************************
*****     ROADWAY DRAINAGE DESIGN     *****
****************************************
```

DESIGNER: INDOT

DATE: 09-02-1998

PROJECT: Example

PROJECT NO.:

INLET NO.: 2

STATION: 10+00

DRAINAGE AREA: .09 Hectares

DESIGN FREQUENCY: 10 Years

ROADWAY & DISCHARGE DATA

Cross-Slope	S (m/m)	Sx (m/m)	n	Q (m^3/s)	T (m)
Composite	0.0120	0.0200	0.016	0.020	1.827

GUTTER FLOW

W (m)	Sw (m/m)	a (mm)	Eo	d (mm)	V (m/s)
0.397	0.0250	N/A	0.679	39.611	0.582

INLET INTERCEPTION

Inlet Type	L (m)	W (m)	E	Qi (m^3/s)	Qb (m^3/s)
Curved Vane	0.879	0.397	0.782	0.015	0.005

HIGHWAY DRAINAGE DESIGN EXAMPLE
(continued)

Figure 36-16B(1)

****
*****   ROADWAY DRAINAGE DESIGN   *****
****

DESIGNER: INDOT

DATE: 09-02-1998

PROJECT: Example

PROJECT NO.:

INLET NO.: 3

STATION: 10+42.7

DRAINAGE AREA: .08 Hectares

DESIGN FREQUENCY: 10 Years

ROADWAY & DISCHARGE DATA

Cross-Slope	S (m/m)	Sx (m/m)	n	Q (m^3/s)	T (m)
Composite	0.0120	0.0200	0.016	0.020	1.827

GUTTER FLOW

W (m)	Sw (m/m)	a (mm)	Eo	d (mm)	V (m/s)
0.397	0.0250	N/A	0.679	39.611	0.582

INLET INTERCEPTION

Inlet Type	L (m)	W (m)	E	Qi (m^3/s)	Qb (m^3/s)
Curved Vane	0.879	0.397	0.782	0.015	0.005

HIGHWAY DRAINAGE DESIGN EXAMPLE
(continued)

Figure 36-16B(1)

```
*************************************************************
*****    FHWA URBAN DRAINAGE DESIGN PROGRAMS     *****
*****           ROADWAY DRAINAGE DESIGN          *****
*************************************************************
```

DESIGNER: INDOT

DATE: 09-02-1998

PROJECT: Example

PROJECT NO.:

INLET NO.: 4

STATION: Street

DRAINAGE AREA: .17 Hectares

DESIGN FREQUENCY: 10 Years

ROADWAY & DISCHARGE DATA

Cross-Slope	Sx (m/m)	Sw (m/m)	n	W (m)	a (mm)
Composite	0.020	0.025	0.016	0.397	0.00

INLET INTERCEPTION

Inlet Type * Sag *	L (m)	W (m)	T (m)	d (m)	Qi (m^3/s)
Curved Vane	1.760	0.397	2.004	0.042	0.034

HIGHWAY DRAINAGE DESIGN EXAMPLE
(continued)

Figure 36-16B(1)

```
**************************************************
*****    FHWA URBAN DRAINAGE DESIGN PROGRAMS  *****
*****         ROADWAY DRAINAGE DESIGN         *****
**************************************************
```

DESIGNER: INDOT

DATE: 09-02-1998

PROJECT: Example

PROJECT NO.:

INLET NO.: 5

STATION: Street

DRAINAGE AREA: .18 Hectares

DESIGN FREQUENCY: 10 Years

ROADWAY & DISCHARGE DATA

Cross-Slope	Sx (m/m)	Sw (m/m)	n	W (m)	a (mm)
Composite	0.020	0.025	0.016	0.397	0.00

INLET INTERCEPTION

Inlet Type * Sag *	L (m)	W (m)	T (m)	d (m)	Qi (m^3/s)
Curved Vane	1.760	0.397	2.043	0.043	0.035

HIGHWAY DRAINAGE DESIGN EXAMPLE
(continued)

Figure 36-16B(1)

****
*****   ROADWAY DRAINAGE DESIGN   *****
****

DESIGNER: INDOT

DATE: 09-02-1998

PROJECT: Example

PROJECT NO.:

INLET NO.: 6

STATION: 11+00

DRAINAGE AREA: .04 Hectares

DESIGN FREQUENCY: 10 Years

ROADWAY & DISCHARGE DATA

Cross-Slope	S (m/m)	Sx (m/m)	n	Q (m^3/s)	T (m)
Composite	0.0120	0.0200	0.016	0.017	1.715

GUTTER FLOW

W (m)	Sw (m/m)	a (mm)	Eo	d (mm)	V (m/s)
0.397	0.0250	N/A	0.708	37.358	0.560

INLET INTERCEPTION

Inlet Type	L (m)	W (m)	E	Qi (m^3/s)	Qb (m^3/s)
Curved Vane	0.879	0.397	0.807	0.013	0.004

HIGHWAY DRAINAGE DESIGN EXAMPLE
(continued)

Figure 36-16B(1)

```
****************************************
*****    ROADWAY DRAINAGE DESIGN    *****
****************************************
```

DESIGNER: INDOT

DATE: 09-02-1998

PROJECT: Example

PROJECT NO.:

INLET NO.: 7

STATION: 11+00

DRAINAGE AREA: .12 Hectares

DESIGN FREQUENCY: 10 Years

ROADWAY & DISCHARGE DATA

Cross-Slope	S (m/m)	Sx (m/m)	n	Q (m^3/s)	T (m)
Composite	0.0120	0.0200	0.016	0.020	1.827

GUTTER FLOW

W (m)	Sw (m/m)	a (mm)	Eo	d (mm)	V (m/s)
0.397	0.0250	N/A	0.679	39.611	0.582

INLET INTERCEPTION

Inlet Type	L (m)	W (m)	E	Qi (m^3/s)	Qb (m^3/s)
Curved Vane	0.879	0.397	0.782	0.015	0.005

HIGHWAY DRAINAGE DESIGN EXAMPLE
(continued)

Figure 36-16B(1)

Computed _______ Date _______

Checked ________ Date _______

Route ____________________
Section ____________________
County ____________________

Station		Length (ft)	Drainage Area A (acres)		Runoff Coefficient C	A x C		Flow Time (min)		Rainfall Intensity I (in/h)	Total Runoff 0.00278CIA = Q (ft^3/s)	Diameter Pipe (in)	Capacity Fill (ft^3/s)	Velocity (ft/s)		Inert Elev.		Manhole Invert Drop	Slope of Drain (ft/ft)
From	To		Increment	Total		Increment	Total	To Upper End	In Section					Flowing Full	Design Flow	Upper End	Lower End		
1	2	24.3	0.125		0.4	0.05													
			0.100	0.225	0.9	0.9	0.14	10		5.28	0.745	12	3.53	5.67	5.00	333.33	333.10		0.01
2	MH1	3.33	0.125		0.4	0.050													
			0.100	0.45	0.9	0.09	0.28	10.2		5.28	1.48	12	3.53	5.67	5.00	333.10	333.06		0.01
MH1	MH2	143.3		0.45			0.28	10.2		5.28	1.48	12	4.24	6.67	6.00	333.06	331.33		0.012
3	MH2	3.33	0.05 0.15	0.20	0.9 0.4	0.045 0.6	0.105	11.2		5.08	0.533	12	3.53	5.67	5.00	331.60	331.57		0.01
MH2	MH3	73.33		0.65			0.385	11.2		5.08	1.96	12	4.24	6.67	6.00	331.33	330.47		0.012
4	5	24.33	0.300		0.4	0.12													
			0.125	0.425	0.9	0.113	0.232	10		5.28	1.22	12	2.83	4.00	3.67	330.80	330.67		0.005
5	MH3	30.0	0.325		0.4	0.13													
			0.125	0.875	0.9	0.113	0.475	10.1		5.28	2.51	12	3.53	4.67	4.67	330.67	330.47		0.007
MH3	MH4	116.7		1.525			1.09	11.4		5.08	5.54	15	8.12	7.00	6.76	330.20	328.80		0.012
6	7	24.3	0.100		0.9		0.09	11.5		5.04	0.454	12	2.83	4.00	2.67	329.33	329.20		0.005
7	MH4	3.33	0.050		0.9	0.045													
			0.250	0.30	0.4	0.100	0.145	11.5		5.04	0.731	12	2.83	4.00	3.33	329.20	329.17		0.005
MH4	Outlet	81.7		1.825			1.23	11.7		5.04	6.20	18	8.48	5.33	5.00	328.53	328.13		0.005

STORM DRAIN COMPUTATION SHEET
FOR EXAMPLE PROBLEM

Figure 36-16C

```
        ******  HYDRA  *******  (Version 6.0)  *****            Date **-04-98
                                                                 Page No   1

   Example Storm Drain Analysis  -  Rational Method


 +++ Commands Read From File C:\HYDRA\EXSTO.HDA
   JOB
   SWI 2
   PDA 0.012 300  .914 .610 0.8 .0030
   RAI 5 170 10 132 15 112 20 100 25 90 30 81
                              IDF CURVE

 .17E+03*..................................................................
        .                                                                 .
        .                                                                 .
        .                                                                 .
 .13E+03.          *                                                      .
        .                                                                 .
        .                    *                                            .
        .                                                                 .
        .                              *                                  .
 .85E+02.                                        *                        .
        .                                                                 .
        .                                                                 .
        .                                                                 .
 .43E+02.                                                                 .
        .                                                                 .
        .                                                                 .
        .                                                                 .
        .                                                                 .
 .00E+00*..................................................................
        5.       9.       12.       16.       19.       23.       26.     3

                           PLOT-DATA (TIME Vs.VALUE)
 ---------------------------------------------------------------------------
     5.   170.00    25.    90.00    0.     .00    0.     .00    0.     .00
    10.   132.00    30.    81.00    0.     .00    0.     .00    0.     .00
    15.   112.00    0.       .00    0.     .00    0.     .00    0.     .00
    20.   100.00    0.       .00    0.     .00    0.     .00    0.     .00
 ---------------------------------------------------------------------------
   HGL 1
   REM Begin Mainline 1 to OUTLET
   NEW Start Lateral 1 to MH1
   REM Link 1  Lateral 1 to 2
   STO .05 .4 10
   STO .04 .9 10
   PIP 7.3 101.2 101.2 100.00 99.93 300
 +++ Tc =  10.0 min
 +++ CA =    .1
 +++ Link #  1,  Flow depth =    .093 m
```

HYDRA EXAMPLE

Figure 36-16D

```
****** HYDRA ******* (Version 6.0) *****                Date **-04-98
                                                         Page No   2

Example Storm Drain Analysis  -  Rational Method

  PNC 1 2 .92 0 0 1
  REM Link 2 Lateral 2 to MH1
  STO .05 .4 10.2
  STO .04 .9 10.2
  PIP 1.0 101.2 101.35 99.93 99.92
+++ Tc =  10.2 min
+++ CA =    .1
+++ Link #  2,  Flow depth =    .132 m
  PNC 2 21 1.22 90 0 1
  REM LINK 3 MAINLINE MH1 to MH2
  PIP 43.0 101.35 100.83 99.92 99.40 300
+++ Tc =  10.2 min
+++ CA =    .1
+++ Link #  3,  Flow depth =    .123 m
  PNC 21 22 1.22 180 0 1
  HOL 3
  NEW Lateral 3 to MH2
  REM Link 4 Lateral 3 to MH2
  STO .02 .9 11.2
  STO .06 .4 11.2
  PIP 1.0 100.68 100.83 99.48 99.47 300
+++ Tc =  11.2 min
+++ CA =    .0
+++ Link #  4,  Flow depth =    .078 m
  PNC 3 22 1.22 0 0 1
  REM Link 5 MH2 to MH3
  REC 3
  PIP 22.0 100.83 100.62 99.40 99.13 300
+++ Tc =  11.2 min
+++ CA =    .2
+++ Link #  5,  Flow depth =    .144 m
  PNC 22 23 1.22 180 0 1
  NEW Lateral 4 to 5
  REM Link 6 Street Inlets 4 to 5
  STO .12 .4 10
  STO .05 .9 10
  PIP 7.3 100.56 100.56 99.24 99.20 300
+++ Tc =  10.0 min
+++ CA =    .1
+++ Link #  6,  Flow depth =    .141 m
  PNC 4 5 .76 0 0 1
  REM Link 7 Street Inlet 5 to MH 3
  HOL 5
  STO .13 .4 10.1
  STO .05 .9 10.1
  PIP 9.0 100.56 100.62 99.20 99.14 300
+++ Tc =  10.1 min
+++ CA =    .2
+++ Link #  7,  Flow depth =    .201 m
```

HYDRA EXAMPLE
(continued)

Figure 36-16D

```
****** HYDRA ******* (Version 6.0) *****                 Date **-04-98
                                                          Page No    3

 Example Storm Drain Analysis  -  Rational Method

  PNC 5 23 1.22 135 0 1
  REM Link 8 MH3 to MH4
  REC 5
  PIP 35.0 100.62 100.20 99.06 98.64 375
+++ Tc =  10.2 min
+++ CA =    .3
+++ Link #  8,  Flow depth =    .188 m
  PNC 23 24 1.22 180 0 1
  NEW Lateral 6 to 7
  REM Link 9 Inlets 6 to 7
  STO .04 .9 11.5
  PIP 7.3 100.00 100.00 98.80 98.76 300
+++ Tc =  11.5 min
+++ CA =    .0
+++ Link #  9,  Flow depth =    .084 m
  PNC 6 7 .92 0 0 1
  REM Link 10 Inlet 7 to MH4
  HOL 7
  STO .02 .9 11.5
  STO .10 .4 11.5
  PIP 1.0 100.00 100.20 98.76 98.75 300
+++ Tc =  11.7 min
+++ CA =    .1
+++ Link # 10,  Flow depth =    .117 m
  PNC 7 24 1.22 180 0 1
  REM Outfall Link:  MH4 to Outlet
  REC 7
  PIP 24.5 100.2 98.95 98.56 98.44 450
+++ Tc =  11.7 min
+++ CA =    .1
+++ Cover at lower manhole    .023 m
+++ Link # 11,  Flow depth =    .140 m
  PNC 24 25 0 135 2
  END
 END OF INPUT DATA.
```

**HYDRA EXAMPLE
(continued)**

Figure 36-16D

****** HYDRA ******* (Version 6.0) *****

Date **-04-98
Page No 4

Example Storm Drain Analysis - Rational Method

*** Start Lateral 1 to

Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
1	7	300	100.000 99.930	.010	1.200 1.270	.875	1.131 1.455	.020 .103	0.
2	1	300	99.930 99.920	.010	1.270 1.430	.945	1.389 1.486	.040 .105	0.
3	43	300	99.920 99.400	.012	1.430 1.430	1.105	1.490 1.634	.040 .116	0.

Length = 51. m Total length = 51. m
Cost = 0. Total Cost = 0.

*** Lateral 3 to MH2

Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
4	1	300	99.480 99.470	.010	1.200 1.360	.875	1.046 1.486	.015 .105	0.
5	22	300	99.400 99.130	.012	1.430 1.490	1.105	1.615 1.646	.054 .116	0.

Length = 23. m Total length = 74. m
Cost = 0. Total Cost = 0.

HYDRA EXAMPLE
(continued)

Figure 36-16D

****** HYDRA ******* (Version 6.0) *****

Date **-04-98
Page No 5

Example Storm Drain Analysis - Rational Method

*** Lateral 4 to 5

Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
6	7	300	99.240 99.200	.005	1.320 1.360	.995	1.061 1.100	.034 .078	0.
7	9	305	99.200 99.140	.007	1.360 1.480	1.030	1.351 1.226	.069 .089	0.
8	35	375	99.060 98.640	.012	1.560 1.560	1.154	1.875 1.889	.102 .209	0.

Length = 51. m Total length = 59. m
Cost = 0. Total Cost = 0.

*** Lateral 6 to 7

Pipe Design

Link	Length (m)	Diam (mm)	Invert Up/Dn (m)	Slope (m/m)	Depth Up/Dn (m)	Min. Cover (m)	Velocity Act/Full (m/s)	--Flow-- Act/Full (m^3/s)	Estimated Cost ($)
9	7	300	98.800 98.760	.005	1.200 1.240	.875	.807 1.100	.012 .078	0.
10	1	300	98.760 98.750	.010	1.240 1.450	.915	1.312 1.486	.032 .105	0.
11	24	450	98.560 98.440	.005	1.640 .510	.023	1.074 1.363	.045 .217	0.

HYDRA EXAMPLE
(continued)

Figure 36-16D

****** HYDRA ******* (Version 6.0) *****

Date **-14-98
Page No 6

Example Storm Drain Analysis - Rational Method

Hydraulic Gradeline Computations

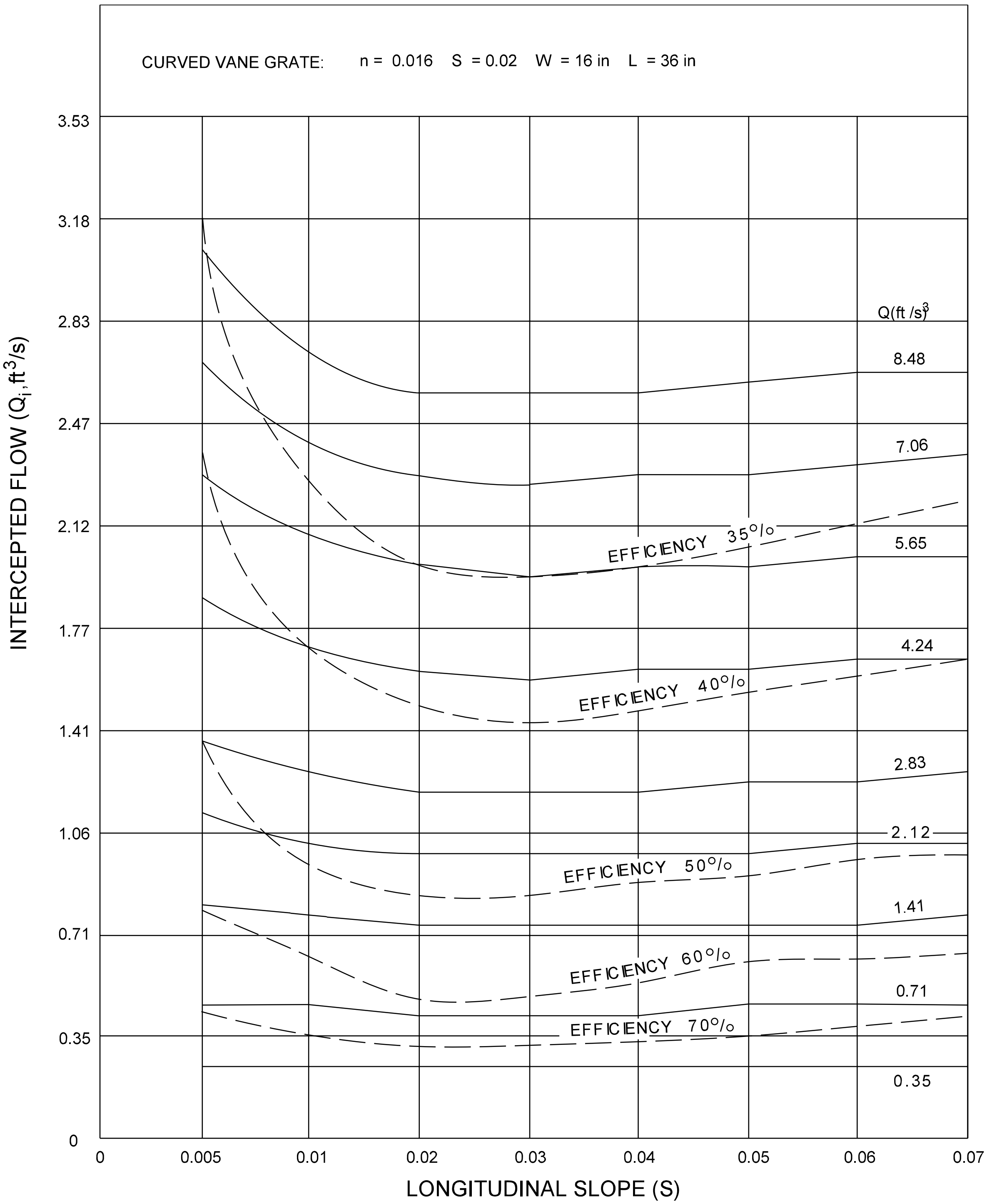
Link #	Down-stream Node #	Hydraulic Gradeline Elevation	Crown Elev.	Possible Surcharge	Ground Elev.	Super-crit.?	Manhole Depth	Loss Coef
1	2	100.075	100.235	N	101.200	Y	.138	.15
2	21	100.059	100.225	N	101.350	Y	.130	.15
3	22	99.564	99.705	N	100.830	Y	.153	.17
4	22	99.553	99.775	N	100.830	Y	.153	.17
5	23	99.272	99.435	N	100.620	Y	.188	.05
6	5	99.415	99.505	N	100.560	Y	.208	.15
7	23	99.341	99.445	N	100.620	Y	.188	.05
8	24	98.824	99.021	N	100.200	Y	.139	.03
9	7	98.887	99.065	N	100.000	Y	.120	.15
10	24	98.863	99.055	N	100.200	Y	.139	.03
11	25	98.578	98.897	N	98.950	Y	.000	.00

Link #	Terminal Node #	Hydraulic Gradeline Elevation	Ground Elevation	Loss Coef.
1	1	100.188	101.200	1.50
4	3	99.704	100.680	1.50

NORMAL END OF HYDRA

HYDRA EXAMPLE
(continued)

Figure 36-16D



INLET CAPACITY CHART
(Curved Vane Grate)

Figure 36-17A

TABLE OF CONTENTS

LIST OF FIGURES

CHAPTER THIRTY-EIGHT

BANK PROTECTION

38-1.0 INTRODUCTION

38-1.01 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.

The available methods of protecting a highway embankment from bank erosion are as follows:

1. relocating the highway away from the stream or water body;
2. moving the water body away from the highway (channel change);
3. changing the direction of the current with training works; and
4. protecting the embankment from erosion.

This Chapter provides procedures for the design of revetment to be used as channel-bank protection, and channel lining on a larger stream or river (i.e., that having a design discharge greater than 50 ft^3/s). Procedures are also provided for riprap protection at a bridge pier or abutment. For a small discharge, the procedures provided in Chapter Thirty should be used. Emphasis in this Chapter has been placed on rock riprap revetment due to its cost, environmental considerations, flexible characteristics, and widespread acceptance. Other channel-stabilization methods such as spurs, guide-bank retard structures, longitudinal dikes, and bulkheads are discussed in *Stream Stability at Highway Structures*, Hydraulic Engineering Circular No. 20.

38-1.02 Erosion Potential

Channel and bank stabilization is essential to the design of a 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 Hydraulic Engineering Circular No. 20. This procedure is described in Chapter Thirty. The three-level analysis provides a procedure for determining the geomorphological characteristics, evaluating the existing conditions through field observations, and determining the hydraulic and sediment transport properties of the stream. If sufficient information is obtained at a given level of the analysis to solve the problem, the procedure may be stopped without proceeding to the other levels.

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, surveying notes, bridge-design files, and river-survey data are available at the INDOT Central Office and at Federal agencies. Gaging-station records and interviews of long-time residents can provide documentation of recent and potentially current channel movement or bank instabilities.

Current site conditions can be used to evaluate stability. If 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 a 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. The presence of one of these conditions does not in itself indicate an erosion problem. Bank erosion is common in each channel if the channel is stable.

38-1.03 Symbols and Definitions

To provide consistency within this Chapter and throughout this *Manual*, the symbols in Figure 38-1A will be used. These symbols were selected because of their wide use in bank- and shore-protection publications. Where the same symbol is used for more than one definition, the symbol will be defined where it is used.

38-2.0 POLICY

A highway alignment or improvement can 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. Department policy requires that the facility, including bank protection, will perform without significant damage or hazard to people and property for flood and flow conditions experienced on a 100-year recurrence interval. The facility, to the maximum extent possible, should perpetuate natural drainage conditions thus protecting and maintaining the environment.

38-3.0 BANK- AND LINING-FAILURE MODES

38-3.01 Potential Failures

Prior to designing a bank-stabilization scheme, the common erosion mechanisms and revetment-failure modes, and the causes or driving forces behind bank erosion processes should be known. Inadequate recognition of potential erosion processes at a particular site may lead to failure of the revetment system.

Many causes of bank erosion and revetment failure have been identified. The more-common causes include abrasion, debris flows, water flow, eddy action, flow acceleration, unsteady flow, freeze-and-thaw, human actions on the bank, ice, precipitation, waves, toe erosion, and subsurface flow. However, it is most often a combination of mechanisms which cause bank or revetment failure, and the actual mechanism or cause is difficult to determine. Failures are classified by mode as follows:

1. particle erosion;
2. translational slide;
3. modified slump; and
4. slump.

38-3.02 Particle Erosion

Particle erosion is the most commonly considered erosion mechanism. Particle erosion results if the tractive force exerted by the flowing water exceeds the bank material's ability to resist movement. If displaced stones are not transported from the eroded area, a mound of displaced rock will develop on the channel bed. The mound has been observed to cause flow concentration along the bank, resulting in further bank erosion.

One type of particle erosion results in loss of the underlying material, resulting in undermining and eventual collapse of the revetment protection. The underlying material is lost through the revetment, or is piped under the toe of the revetment protection. This failure is common in and is damaging to a rigid type of protective lining. Providing a suitable filter, either natural or fabrics, in conjunction with a hydrostatic relief feature, will prevent this failure.

Another type of particle erosion failure occurs at the edges of the protective feature. The interface creates turbulence which in turn increases the tractive stresses placed on the protective layer, underlying layers and the natural bank material beyond the revetment. Extension of the protective feature moves, but does not eliminate, the failure.

38-3.03 Translational 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 riprap bank that extend parallel to the channel. As the slide progresses, the lower part of the riprap 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.

38-3.04 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 to the translational slide, but the geometry of the damaged riprap is similar in shape to initial stages of failure caused by particle erosion.

38-3.05 Slump

Slump is a rotational-gravitational movement of material along a surface of rupture that has a concave upward curve. The cause of a slump failure is related to shear failure of the underlying base material that supports the riprap revetment. The primary feature of a slump failure is the localized displacement of base material along a slip surface, which is caused by excess pore pressure that reduces friction along a fault line in the base material.

38-4.0 REVETMENT TYPES

38-4.01 Common Types

The types of slope protection or revetment used for bank or shore protection and stabilization include the following:

1. rock and rubble riprap;
2. wire-enclosed rock (gabions);
3. preformed blocks;
4. grouted rock;
5. grouted fabric;
6. sand or cement bags; and
7. soil cement.

38-4.02 Riprap

Riprap is a layer or facing of rock, dumped or hand-placed to prevent erosion, scour, or sloughing of a structure or embankment. Materials other than rock are also referred to as riprap. These include rubble, broken concrete slabs, or preformed-concrete shapes such as 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. The depth of riprap should be taken as 18 in.

38-4.03 Wire-Enclosed Rock

A wire-enclosed rock, or gabion, revetment consists of rectangular wire mesh baskets filled with rock. This revetment is formed by filling pre-assembled wire baskets with rock and anchoring them to the channel bottom or bank. A wire-enclosed rock revetment is either a rock-and-wire mattress, or blocks. In a mattress, 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 gabion baskets comprising the mattress have a depth dimension which is much smaller than its width or length. A block gabion is more equal-dimensional, having a depth that is approximately the same as its width and of the same order of magnitude as its length. It is rectangular or trapezoidal in shape. A block gabion revetment is formed by stacking individual gabion blocks in a stepped fashion.

38-4.04 Precast-Concrete Block

The preformed sections which comprise the revetment system are butted together or joined. As such, they form a continuous blanket or mat. The concrete blocks which make up the mats differ in shape and method of articulation but share certain common features. The features include flexibility, rapid installation, and provisions for establishment of vegetation within the revetment. The permeable nature of this revetment permits free draining of the bank materials. The flexibility, although limited, allows the mattress to conform to minor changes in the bank geometry. Its limited flexibility, however, subjects it to undermining in an environment characterized by large and relatively rapid fluctuations in the surface elevation of the channel bed or bank. Unlike wire-enclosed rock, the open nature of precast-concrete blocks does promote volunteering of vegetation within the revetment.

38-4.05 Grouted Riprap

Grouted riprap consists of rock-slope protection having voids filled with concrete grout to form a monolithic armor. Grouted riprap is a rigid revetment. It will not conform to changes in the bank geometry due to settlement. As with a monolithic revetment, grouted riprap is susceptible to failure from undermining and the subsequent loss of the supporting bank material. Although it is rigid, grouted riprap is not extremely strong. Therefore, the loss of a small area of bank support can cause failure of large portions of the revetment. See the INDOT *Standard Specifications* for more information.

38-4.06 Grouted Fabric Slope Pavement

A grouted fabric slope pavement revetment is constructed by injecting sand-cement mortar between two layers of double-woven fabric which has first been positioned on the slope to be protected. Mortar may be 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. The fabric, to which the mortar remains tightly bonded, then acts as tensile reinforcing to hold the mortar in place on the slope. This revetment is analogous to slope paving with reinforced concrete. The bottom layer of fabric acts as a filter cloth underlayment to prevent loss of soil particles through cracks which may develop in the revetment as a result of soil subsidence. Greater relief of hydrostatic uplift is provided by weep holes or filter points which are woven into the fabric and remain unobstructed by mortar during the filling operation.

38-4.07 Sand-Cement Bags

Sand-cement bags 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. A filter fabric is placed underneath this type of revetment. Adequate protection of the terminals and toe is essential. The revetment has little flexibility, low tensile strength, and is susceptible to damage, particularly on a relatively flat slope where the area of contact between the bags is less.

38-4.08 Soil Cement

Soil cement consists of a dry mix of sand, cement, and admixtures batched in a central mixing plant. It is transported, placed with 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 stable provided toe scour protection has been incorporated into the design.

38-5.0 DESIGN CONCEPTS

38-5.01 Introduction

Concepts related to the design of bank protection are discussed below.

38-5.02 Design Discharge

The design flow rate for the design or analysis of a highway structure in the vicinity of a river or stream has a 10- to 100-year recurrence interval. This discharge level will also be applicable to the design of a revetment system. However, a lower discharge may produce hydraulically-worse conditions with respect to riprap stability. 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 a structure associated with the riprap scheme.

38-5.03 Flow Types

Open-channel flow can be classified as follows:

1. uniform, gradually-varying, or rapidly-varying flow;
2. steady or unsteady flow; and
3. subcritical or supercritical flow.

The design relationships described herein are based on the assumption of uniform, steady, subcritical flow. The relationships are also valid for gradually-varying flow conditions. Although the individual hydraulic relationships are not in themselves applicable to rapidly-varying, unsteady, or supercritical flow conditions, procedures are provided for extending their use to these flow conditions. See Chapter Thirty for more information related to channel design.

A rapidly-varying, unsteady flow condition is common in an area of flow expansion, flow contraction, or reverse flow. These conditions are common at and immediately downstream of a bridge. A supercritical or near-supercritical flow condition is common at a bridge constriction or a steeply-sloped channel.

Non-uniform, unsteady, and near-supercritical flow conditions create stresses on the channel boundary that 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 provided in Section 38-6.0 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 the 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.

38-5.04 Section Geometry

The design procedures described herein require as input channel cross-section geometry. The cross-section geometry is necessary to establish the hydraulic design parameters (flow depth, top width, velocity, hydraulic radius, etc.) required by the riprap-design procedures, and to establish a construction cross section for placement of the revetment material. Where the entire channel perimeter will be stabilized, the selection of appropriate channel geometry is only a function of the desired channel conveyance properties and limiting geometric constraints. However, where the channel bank alone will be protected, the design must consider the existing channel-bottom geometry.

The development of an appropriate channel section for analysis is subjective. The intent is to develop a section which reasonably simulates a worst-case condition with respect to riprap stability. Information which can be used to evaluate channel geometry includes current channel surveys, past channel surveys (if available), and current and past aerial photos. The effect that channel stabilization will have on the local channel section must be considered.

The first problem arises if an attempt is made to establish an existing channel bottom profile for use in design. A single channel profile is 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 stability of the site and a history of past channel-geometry changes. Past surveys for a particular site may not be available. If so, past surveys at other sites in the vicinity of the design location may be used to evaluate past changes in channel geometry.

38-5.05 Flow In Channel Bend

Flow conditions in a channel bend are complicated by the distortion of flow patterns in the vicinity of the bend. In a long, relatively straight channel, the flow conditions are uniform and symmetrical about the centerline of the channel. However, in a channel bend, the centrifugal forces and secondary currents produced can lead to non-uniform and non-symmetrical flow conditions.

The increased velocities and shear stresses that are generated as a result of non-uniform flow should be considered in a bend.

Superelevation of flow in a channel bend should be considered in the revetment design. Although the magnitude of superelevation is small if compared with the overall flow depth in the bend (less than 1 ft), it should be considered in establishing freeboard limits for a bank-protection scheme on a sharp bend. The magnitude of superelevation at a channel bend may be estimated for subcritical flow from the equation as follows:

$$Z = \frac{CV_a^{\,2}T}{gR_o} \qquad \text{(Equation 38-5.1)}$$

Where: Z = superelevation of the water surface, ft

C = coefficient that relates free vortex motion to velocity streamline for unequal radii of curvature

V_a = mean channel velocity, ft/s

T = water-surface width at section, ft

g = gravitational acceleration, 32.2 ft/s^2)

R_o = 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 from 0.5 to 3.0, with an average value of 1.5.

38-5.06 Flow Resistance

The hydraulic analysis performed as a part of the riprap-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. Seasonal changes in these factors must also be considered. See Chapter Thirty for a discussion on the selection of Manning's *n* value.

38-5.07 Extent of Protection

Extent of protection refers to the longitudinal and vertical extent of protection required to adequately protect the channel bank.

38-5.07(01) Longitudinal Extent

The longitudinal extent of protection required for a particular bank protection scheme is dependent on local site conditions. The revetment should be continuous for a distance greater than the length that is impacted by channel-flow forces severe enough to cause dislodging or transport of bank material. Although this is a vague criterion, it should be considered. Review of existing bank-protection sites has revealed that a common misconception in stream-bank 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 38-5A. As illustrated, the minimum distances recommended for bank protection are an upstream distance of 1 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 an analysis of flow conditions in symmetric channel bends under ideal laboratory conditions. Real-world conditions are not as simplistic.

Many site-specific factors have an effect on the actual length of bank that should be protected. The designer will determine that the above criteria are difficult to apply on a mildly-curving bend or on a channel having irregular, non-symmetric bends. Other channel controls such as bridge abutments can be producing a stabilizing effect on the bend so that only a part of the channel bend needs to be stabilized. The magnitude or nature of the flow event can only cause erosion problems in a 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 useful for the evaluation of the longitudinal extent of protection required, particularly if the channel is actively eroding. In a straight channel reach, scars on the channel bank may be useful to help identify the limits required for channel-bank protection. The upstream and downstream limits of the protection scheme should be extended a minimum of 1 channel width beyond the observed erosion limits.

In a curved channel reach, the scars on the channel bank can be used to establish the upstream limit of erosion. A minimum of 1 channel width should be added to the observed upstream limit to define the limit of protection. The downstream limit of protection required in a curved channel reach is more difficult to define. Because the natural progression of bank erosion is in the downstream direction, the present visual limit of erosion may not define the ultimate downstream limit. Additional analysis based on consideration of flow patterns in the channel bend may be required.

38-5.07(02) Vertical Extent

The vertical extent of protection required of a revetment includes design height and foundation or toe depth.

1. Design Height. The design height of a riprap installation should be equal to the design highwater elevation plus an allowance for freeboard. Freeboard is provided in a

causeway situation to ensure that the desired degree of protection will not be reduced due to unaccounted factors, including the following:

a. wave action from wind or boat traffic;
b. superelevation in channel bends;
c. hydraulic jumps; and
d. flow irregularities due to piers, transitions, or flow junctions.

Erratic phenomena such as unforeseen embankment settlement, the accumulation of silt, trash, debris in the channel, aquatic or other growth in the channel, and ice flows should be considered in setting the freeboard height. Wave runup on the bank must be considered.

The prediction of wave height from a boat-generated wave is not as straightforward as other wave sources. Figure 38-5B provides a definition sketch for the wave-height discussion to follow. The height of a boat-generated wave must be estimated from observations.

It is necessary to estimate the magnitude of wave runup which results if waves impact the bank. Wave runup is a function of the design-wave height, the wave period, bank angle, and the bank-surface characteristics (as represented by different revetment materials). For a wave height of less than 2 ft, wave runup can be computed using Figures 38-5C and 38-5D. The runup height, R, shown in Figure 38-5D is for concrete pavement. Correction factors are provided in Figure 38-5C for reducing the runup magnitude for other revetment materials. The correction factor is multiplied by the wave height to obtain R.

The factors to be considered in the selection of an appropriate freeboard height are described below. As a minimum, a freeboard elevation of 1 ft to 2 ft should be used in an unconstricted reach, or 2 ft to 3 ft in a constricted reach. The Federal Emergency Management Agency requires 3 ft for levee protection, or 4 ft at a bridge for a 100-year flood. If computational procedures indicate that additional freeboard may be required, the greater height should be used. Wave and flow conditions should be observed during various seasons of the year, if possible. Consult existing records, and interview persons who have knowledge of past conditions in establishing the necessary vertical extent of protection required for a particular revetment installation.

2. Toe Depth. The undermining of revetment-toe protection has been identified as one of the 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 and natural scour and fill processes.

The relationships provided in Equations 38-5.2 and 38-5.3 below can be used to estimate the probable maximum depth of scour due to the natural scour and fill phenomenon in a straight channel or in a channel having mild bends. In application, the depth of scour, d_s, should be measured from the lowest elevation in the cross section. The low point in the cross section may eventually move adjacent to the protection if this is not shown in the current survey.

$d_s = 12.2$ ft for $D_{50} < 0.005$ ft (Equation 38-5.2)

$d_s = 6.5\ D_{50}^{-0.11}$ for $D_{50} > 0.005$ ft (Equation 38-5.3)

Where: d_s = estimated probable maximum depth of scour, ft
D_{50} = median diameter of bed material, in.

If D_{50} is in inches, $d_s = 1.738D_{50}^{-0.11}$.

The depth of scour predicted from Equations 38-5.2 and 38-5.3 must be added to the magnitude of predicted degradation and local scour to arrive at the total required toe depth.

38-6.0 DESIGN GUIDELINES

38-6.01 Rock Riprap

Guidelines are provided for bank slope, rock size, rock gradation, riprap layer thickness, filter design, edge treatment, and construction considerations. Construction details are illustrated. The guidelines apply equally to rock or rubble riprap.

38-6.01(01) Bank Slope

A primary consideration in the design of a stable riprap bank-protection scheme is the slope of the channel bank. For a riprap installation, the maximum recommended face slope is 2H:1V. Although not recommended, the steepest slope acceptable for rubble revetment is 1.5H:1V. To be stable under an identical wave attack or lateral velocity, a rubble revetment with a steep slope will need larger rubble sizes and greater thicknesses than one with a flatter slope.

38-6.01(02) 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. Relationships are provided for evaluating the riprap size required to resist particle and wave-erosion forces.

1. Particle Erosion. The methods used to evaluate a material's resistance to particle erosion are the permissible-velocity approach and the permissible tractive-force (shear stress) approach. In 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.

2. Design Relationship. A riprap-design relationship that is based on tractive-force theory with velocity as its primary design parameter is provided in Equation 38-6.1. The design relationship in Equation 38-6.1 below is based on the assumption of uniform, gradually-varying flow. Figure 38-6A provides a graphical solution to Equation 38-6.1. Equation 38-6.2 can be solved using Figures 38-6B and 38-6C.

$$D_{50} = \frac{0.001 V_a^{\ 3}}{\left(d_{avg}\right)^{0.5} \left(K_1\right)^{1.5}} \qquad \text{(Equation 38-6.1)}$$

Where: D_{50} = median riprap particle size, ft
C = correction factor, described below
V_a = average velocity in the main-flow channel, ft/s
d_{avg} = average flow depth in the main-flow channel, ft

K_1 is defined as follows:

$$K_1 = \left(1 - \frac{\sin^2\theta}{\sin^2\phi}\right)^{0.5} \qquad \text{(Equation 38-6.2)}$$

Where: θ = bank angle with the horizontal
φ = riprap material's angle of repose.

The average flow depth and velocity used in Equation 38-6.1 are main channel values. The main channel is defined as the area between the channel banks (see Figure 38-6D).

Equation 38-6.1 is based on a rock-riprap specific gravity of 2.65 and a stability factor of 1.2. Equations 38-6.3 and 38-6.4 provide correction factors for other specific gravities and stability factors as follows:

$$C_{sg} = \frac{2.12}{(S_s - 1)^{1.5}} \qquad \text{(Equation 38-6.3)}$$

Where S_s = specific gravity of rock riprap.

$$C_{sf} = \left(\frac{SF}{1.2}\right)^{1.5} \qquad \text{(Equation 38-6.4)}$$

Where SF = stability factor to be applied.

The correction factors computed using Equations 38-6.3 and 38-6.4 are multiplied together to form a single correction factor, C. This correction factor is then multiplied by the riprap size computed from Equation 38-6.1 to arrive at a stable riprap size. Figure 38-6E provides a solution to Equations 38-6.3 and 38-6.4 using correction factor C.

The stability factor, SF, used in Equation 38-6.4 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, and the riprap is considered to be stable. A stability factor of 1.2 was used in the development of Equation 38-6.1.

The stability factor is used to reflect the level of uncertainty in the hydraulic conditions at a particular site. Equation 38-6.1 is based on the assumption of uniform or gradually varying flow. This assumption can be violated or other uncertainties can occur, such as debris or ice impacts, the cumulative effect of high shear stresses, or forces from boat-generated waves. The stability factor is used to increase the design rock size if these conditions must be considered. Figure 38-6F provides guidelines for the selection of an appropriate value for the stability factor.

3. Application. Application of the relationship in Equation 38-6.1 is limited to a uniform or gradually-varying flow condition that is in a straight or mildly-curving channel reach of relatively uniform cross section. The relationship is also applicable in a non-uniform, rapidly-varying flow condition exhibited in a natural channel with sharp bends and steep slopes, or in the vicinity of a bridge pier or abutment.

   To fill the need for a design relationship that can be applied at a sharp bend or on steep slopes in a natural channel or at a bridge abutment, Equation 38-6.1 should be used with appropriate adjustments in the velocity or stability factor as described below.

4. Steep Slopes. The flow condition in a steep-sloped channel is rarely uniform. It is characterized by high flow velocity and significant flow turbulence. In applying

Equation 38-6.1 to a steep-sloped channel, the appropriate velocity must be determined. In determining the flow velocity, Equation 38-6.5 must be used to determine the channel's roughness coefficient. The selection of the stability factors shown in Figure 38-6F must be considered.

For a high-gradient stream, it is difficult to obtain an estimate of the median-bed material size. If the stream slope is steeper then 0.2% and the bed material is larger than 0.20 ft (gravel, cobble, or boulder-size material), the relationship shown in the following equation be used to evaluate the base Manning's *n*.

$$n = 0.39\, S_f^{0.38}\, R^{-0.16} \qquad \text{(Equation 38-6.5)}$$

Where: S_f = friction slope
R = hydraulic radius, ft

5. Bridge Pier. For recommendations, see Chapter Thirty-two.

6. Wave Erosion. Waves generated by boat traffic have also been observed to cause bank erosion on an inland waterway. The most widely-used measure of riprap's resistance to waves is that developed by R. Y. Hudson, *Laboratory Investigations of Rubble-Mound Breakwaters*, 1959. The Hudson relationship is described in the equation as follows:

$$W_{50} = \frac{\gamma_s H^3}{2.2 \cot\theta (S_s - 1)^3} \qquad \text{(Equation 38-6.6)}$$

Where: W_{50} = weight of median particle, lb

γ_s = unit weight of riprap (solid) material, lb/ft^3 (other parameters are as defined previously)

H = wave height, ft

S_s = specific gravity of riprap material

θ = bank angle with the horizontal

Assuming S_s = 2.65 and γ_s = 165 lb/ft^3, Equation 38-6.6 can be reduced to the following:

$$W_{50} = \frac{16.7 H^3}{\cot\theta} \qquad \text{(Equation 38-6.7)}$$

In terms of an equivalent diameter, Equation 38-6.7 can be reduced to the following:

$$D_{50} = \frac{0.75H}{\cot^{0.33}\theta} \qquad \text{(Equation 38-6.8)}$$

Where D_{50} = median riprap size, ft.

Methods for estimating a design wave height are provided in Section 38-5.07. Equation 38-6.8 is provided in nomograph form in Figure 38-6G. Equations 38-6.7 and 38-6.8 can be used for preliminary or final design where H is less than 5.0 ft, and there is no major overtopping of the embankment.

7. Ice Damage. Ice can affect riprap linings. Moving surface ice can cause crushing and bending forces and large impact loadings. The tangential flow of ice along a riprap-lined channel bank can also cause excessive shearing forces. Quantitative criteria for evaluating the impact ice has on a channel-protection scheme are unavailable. However, historic observations of ice flows in New England rivers indicate that riprap sized to resist a design flow event will also resist ice forces.

   For design, consideration of ice forces should be evaluated as required for each project. Ice flows are not of sufficient magnitude to warrant detailed analysis. Where ice flows have historically caused problems, a stability factor of 1.2 to 1.5 should be used to increase the design rock size. The selection of an appropriate stability factor to account for ice-generated erosive problems should be based on local experience.

38-6.01(03) Rock Gradation

The gradation of stones in riprap revetment affects the riprap's resistance to erosion. The stone should be well-graded throughout the riprap-layer thickness. The gradation limits should not be so restrictive that production costs are excessive. Figure 38-6H provides suggested guidelines for establishing gradation limits. Figure 38-6J can be used as an aid in selecting appropriate gradation limits.

The use of a four-point gradation as specified in Figure 38-6H can be too harsh a requirement for a smaller quarry. If so, the 85% requirement can be dropped as is done in Figure 38-6 I. A uniform gradation between D_{50} and D_{100} will likely result in an appropriate D_{85}.

38-6.01(04) Layer Thickness

All stones should be contained within the riprap-layer thickness to provide maximum resistance against erosion. Oversized stones in isolated locations 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 oversized stones should be removed individually and replaced with proper-size stones. The following criteria apply to the riprap layer thickness.

1. It should not be less than the spherical diameter of the D_{100} (W_{100}) stone, or less than two times the spherical diameter of the D_{50} (W_{50}) stone, whichever results in the greater thickness.

2. It should not be less than 12 in. for practical placement.

3. The thickness determined by either Item 1 or 2 above should be increased by 50 percent if the riprap is placed underwater to provide for uncertainties associated with this type of placement.

4. An increase in thickness of 6 in. to 12 in., 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.

38-6.01(05) Filter Design

A filter is a transitional layer of gravel, small stones, 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. A filter should be used where the riprap is placed on non-cohesive material subject to significant subsurface drainage, such as where the water-surface level frequently fluctuates, or in an area of high groundwater level.

1. Granular Filter. For rock riprap, a filter ratio of 5 or less between layers will result in a stable condition. The filter ratio is defined as the ratio of the 15-percent particle size, D_{15}, of the coarser layer to the 85-percent particle size, D_{85}, of the finer layer. The ratio of the 15-percent particle size of the coarser material to the 15-percent particle size of the finer material should exceed 5 but should be less than 40. This requirement can be stated as follows:

$$\frac{D_{15}\,\textit{(coarser layer)}}{D_{85}\,\textit{(finer layer)}} < 5 < \frac{D_{15}\,\textit{(coarser layer)}}{D_{15}\,\textit{(finer layer)}} < 40 \qquad \text{(Equation 38-6.9)}$$

The first test of the inequality is intended to prevent piping through the filter. The second test provides for adequate permeability for structural bedding layers. The right-hand portion provides a uniformity criterion.

If a single layer of filter material will not satisfy the filter requirements, one or more additional layers of filter material must be used. The filter requirement applies between the bank material and the filter blanket, between successive layers of filter material if more than one layer is used, and between the filter blanket and the riprap cover. In addition to the filter requirements, the grain-size curves for the various layers should be approximately parallel to minimize the infiltration of fine material from the finer layer to the coarser layer. Not more than 5 percent of the filter material should pass the 75-µm sieve. Figures 38-6J and 38-6K can be used as an aid in designing an appropriate granular filter. An editable version of Figure 38-6K may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/.

The thickness of the filter blanket should range from 6 in. to 18 in. for a single layer, or from 4 in. to 8 in. for individual layers of a multiple layer blanket. Where the gradation curves of adjacent layers are approximately parallel, the thickness of the blanket layers should approach the minimum. The thickness of individual layers should be proportionately increased above the minimum as the gradation curve of the material comprising the layer departs from a parallel pattern.

2. Geotextile Filter. A synthetic geotextile filter may be used use as an alternative to a granular filter. See Figure 38-6L. Since the original geotextile erosion control application in 1957, thousands of successful projects have been completed. The advantages relevant to using a geotextile filter are as follows.

   a. Installation is quick and labor efficient.

   b. A geotextile filter is more economical than a granular filter.

   c. A geotextile filter has consistent and more-reliable material quality.

   d. A geotextile filter has high inherent tensile strength.

   e. Local availability of suitable granular-filter material is no longer a design consideration in using a fabric filter.

   Disadvantages include the following.

   a. Geotextiles can be difficult to install underwater.

   b. Geotextiles have widely-variable hydraulic properties and must be designed based on project-specific conditions and performance requirements.

c. Geotextile-filter performance is sensitive to construction procedures.

d. Special installation or inspection procedures may be necessary in using a geotextile filter.

3. Geotextile-Filter Design. The design follows traditional graded granular-filter design principles and should consider the following:

a. soil retention (piping resistance);
b. permeability;
c. clogging; and
d. survivability.

Individual site conditions and performance requirements should be established in conjunction with the geotextile design. Generalized geotextile requirements should be used only for a very small or non-critical/non-severe installation where a detailed analysis is not warranted. AASHTO has developed materials and construction specifications (AASHTO Specification M 288) for a routine, non-critical/non-severe geotextile application. Geotextile-filter design requirements, for all levels of project severity and criticality, are provided in the Federal Highway Administration publication *Geosynthetic Design and Construction Guidelines*, (FHWA-HI-95-038). This reference provides guidance on specifying and installing geotextiles for a variety of transportation applications. The American Society for Testing and Materials 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 the design if using geosynthetics.

The following design steps are necessary for geotextile design in riprap or another permanent-erosion-control application.

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 as follows:

(1) soil retention;
(2) permeability/permittivity;

(3) clogging; and
(4) survivability.

Step 6: Estimate cost and prepare specifications.

4. Geotextile Installation Procedure. A properly-selected cloth should be installed with regard to the following precautions.

   a. Grade the area and remove debris to provide a smooth, fairly even surface.

   b. Place geotextile loosely, and lay in the direction of anticipated water flow or movement.

   c. Seam or overlap the geotextile as required.

   d. 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 or armor materials. Additional reductions in slope may be necessary due to hydraulic considerations and possible long-term stability. For a slope steeper than 2.5H:1V, special construction procedures will be required.

   e. For a stream bank or wave-action application, the geotextile must be keyed in at the bottom of the slope. If the system cannot be extended about 10 ft above the anticipated high-water level, the geotextile should also be keyed in at the crest of the slope.

   f. Place the revetment cushion layer or riprap over the geotextile width while avoiding puncturing it.

38-6.01(06) Edge Treatment

The edges of a riprap revetment (flanks, toe, and head) require a treatment to prevent undermining. The flanks of the revetment should be designed as illustrated in Figure 38-6M. The upstream flank is illustrated in section (a) and the downstream flank in section (b) of the figure. A more constructible flank section uses riprap rather than compacted fill.

Undermining of the revetment toe is one of the primary mechanisms of riprap failure. The toe of the riprap should be designed as illustrated in Figure 38-6N. The toe material should be placed in a toe trench along the entire length of the riprap blanket.

Where a toe trench cannot be dug, the riprap blanket should terminate in a thick, stone toe at the level of the streambed (see alternate design in Figure 38-6N). The toe material should not mound and form a low dike. A low dike along the toe can result in flow concentration along the revetment face which can stress the revetment to failure. The channel's design capability should not be impaired by placement of too much riprap in a toe mound.

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, the stone in the toe will launch into the eroded area as illustrated in Figure 38-6 O. Observation of the performance of this type of rock toe 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 from the scour evaluation. The alternate location can be used if the amount of rock required does not constrain the channel. Establishing a design scour depth is described in Section 38-5.07.

38-6.01(07) Construction Considerations

Construction considerations related to a riprap revetment include bank slope or angle, bank preparation, and riprap placement.

1. <u>Bank Preparation</u>. The bank should be prepared by clearing all trees and debris from the bank and grading the bank surface to the desired slope. The graded surface should not deviate from the specified slope line by more than 6 in. However, a local depression larger than this can be accommodated because initial placement of filter material or rock for the revetment will fill such a depression. Large boulders or debris found buried near the edges of the revetment should be removed.

2. <u>Riprap Placement</u>. The common methods of riprap placement are hand placing; machine placing, such as from a skip, dragline, or bucket; and dumping from trucks and spreading by bulldozer. Hand placement produces the best riprap revetment, but it is the most expensive method except if labor is inexpensive. Steeper side slopes can be used with hand-placed riprap more so than with another placement method. Where steep slopes are unavoidable (the channel width is constricted by an existing bridge opening or other structure, and right of way is costly), hand placement should be considered.

In the machine-placement method, sufficiently small increments of stone should be released as close to their final positions as practical. Rehandling 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 because this may result in the same undesirable conditions. Riprap placement by dumping and spreading is satisfactory, provided the required layer thickness is achieved. However, this is the least-desirable method, as a large amount of segregation and breakage can occur. It may be economical to increase the layer thickness and stone size somewhat to offset the shortcomings of this placement method.

38-6.01(08) Design Procedure

The rock-riprap design procedure described below consists of preliminary data analysis, rock sizing, and revetment-detail design. Figures 38-6P and 38-6Q provide a useful format for recording data at each step of the analysis. Editable versions of these forms may also be found on the Department's website at www.in.gov/dot/div/contracts/design/dmforms/.

1. Compile all necessary field data including channel cross section surveys, soils data, aerial photographs, history of problems at site, etc.

2. Determine design discharge.

3. Develop design cross sections. The rock-sizing procedure described herein is intended to prevent riprap failure from particle erosion.

4. Compute design water surface as follows.

    a. In 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 *n*. See Section 38-5.06.

    b. If the design section is a regular trapezoidal shape, and flow can be assumed to be uniform, use the design procedure described in Chapter Thirty.

    c. If the design section is irregular or flow is not uniform, backwater procedures must be used to determine the design water surface.

    d. The backwater analysis must be based on conveyance weighing of flows in the main channel, right bank, and left bank.

5. Determine design average velocity and depth as follows.

a. Average velocity and depth should be determined for the design section in conjunction with the computations from Step 4. The average depth and velocity in the main flow channel should be used.

b. If riprap is being designed to protect the channel banks, abutments, or piers located in the floodplain, the average floodplain depth and velocity should be used.

6. Compute the bank angle correction factor K_1 from Equation 38-6.2 and Figures 38-6B and 38-6C.

7. Determine the riprap size required to resist particle erosion from Equation 38-6.1 and Figure 38-6A as follows.

   a. Initially assume no corrections.

   b. Evaluate the correction factor for rock-riprap specific gravity and stability factor $C = C_{sg}C_{sf}$.

   c. If designing riprap for a pier or abutment, see Chapter Thirty-two.

8. If the entire channel perimeter is being stabilized, and an assumed D_{50} was used in determination of Manning's *n* for backwater computations, repeat Steps 4 through 7.

9. If in a causeway situation, determine the wave height.

10. Select the final D_{50} riprap size, set material gradation from Section 38-6.01(03) and Figure 38-6J, and determine riprap-layer thickness from Section 38-6.01(05).

11. Determine the longitudinal extent of protection required (Section 38-5.07).

12. Determine the appropriate vertical extent of revetment (Section 38-5.07).

13. Design filter layer from Section 38-6.01(05), Figure 38-6K as follows:

   a. determine the appropriate filter material size and gradation; and
   b. determine layer thickness.

14. Design edge flanks and toe. See Section 38-6.01(07).

38-6.01(09) Design Examples

The following design examples illustrate the use of the design methods and procedures outlined above. Example 38-6.1 illustrates the design of a riprap-lined channel section. Example 38-6.2 illustrates the design of riprap as bank protection. In the examples, the steps correlate with the design procedure described above. Computations are also shown on the appropriate figures.

* * * * * * * *

1. Example 38-6.1. A 1267-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 1267 ft to 1017 ft. The channel is to be sized to carry 5000 ft^3/s within its banks. Additional site conditions are as follows:

   a. flow conditions can be assumed to be uniform or gradually varying;

   b. the existing channel profile dictates that the straightened reach be designed at a uniform slope of 0.0049;

   c. the natural soils are gap graded from medium sands to coarse gravels, with the distribution as follows:

      D_{85} = 0.107 ft D_{50} = 0.065 in. D_{15} = 0.0046 ft
      K (permeability) = 1.17×10^{-3} ft/s

   d. Available rock riprap has a specific gravity of 2.65, and D_{50} = 1.0 ft.

   Design a stable trapezoidal riprap-lined channel for this site. Design figures used to summarize the data in this example are reproduced in Figures 38-6R and 38-6S.

   a. Compile Field Data.

      (1) See the given information for this example.

      (2) Other field data include site history, geometric constraints, roadway-crossing profiles, site topography, etc.

   b. Design Discharge.

      (1) Given as 5000 ft^3/s.

(2) Discharge in main channel equals the design discharge because the entire design discharge is to be contained in the channel as specified.

c. Design Cross Section.

(1) As specified, a trapezoidal section is to be designed.

(2) Initially assume a trapezoidal section with a 20 ft bottom width and 2H:1V side slopes (see Figure 38-6R).

d. Compute Design Water Surface.

(1) Determine roughness coefficient, $n = 0.04$.

(2) Compute flow depth. Assume $R = 7.0$ ft

(3) Solve Manning's equation for normal depth or see Chapter Thirty.

$$Q = \frac{1.486 A R^{0.67} S^{0.5}}{n}$$

d = 11.8 ft

(4) Compute the hydraulic radius to compare with the assumed value used in Step d(1). Use computer programs, available charts and tables, or manually compute.

$R = A/P$
$R = 531 / 74$
$R = 7.17$ ft, which is approximately equal to R (assumed); therefore,
$d = 12.0$ ft OK

e. Determine design parameters.

$A = 11.8(20) + 2\,(11.8)^2 = 514.5\ \text{ft}^2$
$V_a = Q/A = 5000 / 514.5 = 9.7$ ft/s
$d_a = d = 11.8$ ft (uniform channel bottom)

f. Bank Angle Correction Factor.

θ = 2H:1V
φ = 41 (from Figure 38-6B)
$K_1 = 0.73$ (from Figure 38-6A)

g. Determine riprap size. See Section 38-6.01(08).

(1) Using Figure 38-6S.

for channel bed, D_{50} = 0.28 ft
for channel bank, D_{50} = 0.44 ft

(2) Riprap specific gravity = 2.65 (given).

Stability factor = 1.2 (Figure 38-6P, column 9)
Uniform flow, little or no uncertainty in design
$C = 1$

(3) No piers or abutments to evaluate for this example, therefore $C_{p/a} = 1$.

(4) Corrected riprap size.

For channel bed, $D'_{50} = D_{50}$ = 0.28 ft
For channel banks, $D'_{50} = D_{50}$ = 0.44 ft

h. This step is not applicable.

i. Surface waves. Surface waves determined not to be a problem at this site.

j. Select design riprap size, gradation, and layer thickness.

D_{50} size: Recommend AASHTO Face Class riprap
D_{50} = 0.97 ft for entire perimeter

Layer thickness, T.

$T = 2D_{50}$ = 2(0.97 ft) = 1.94 ft, or $T = D_{100}$ = 1.32 ft
Use T = 2.0 ft

k. Longitudinal Extent of Protection. Riprap lining is to extend along the entire length of the straightened reach plus an additional upstream and downstream distance.

l. Vertical Extent of Protection. Riprap entire channel perimeter to top of bank.

m. Filter layer design.

(1) Filter material size.

$$\frac{D_{15}\ coarser\ layer}{D_{85}\ \ finer\ layer} < 5 < \frac{D_{15}\ coarser\ layer}{D_{15}\ \ finer\ layer} < 40 \qquad \text{(Equation 38-6.10)}$$

(2) Riprap-to-soil interface.

$$\frac{D_{15}\ riprap}{D_{85}\ soil} = \frac{0.6}{0.10} = 6 > 5 \qquad \text{(Equation 38-6.11)}$$

and,

$$\frac{D_{15}\ riprap}{D_{15}\ soil} = \frac{0.60}{0.0046} = 130 > 40 \qquad \text{(Equation 38-6.12)}$$

Therefore, a filter layer is needed. Try a 2 in. uniformly-graded coarse gravel filter.

(3) Filter-to-soil interface.

$$\frac{D_{15}\ \ filter}{D_{85}\ soil} = \frac{0.1}{0.107} = 0.94 < 5 \qquad \text{(Equation 38-6.13)}$$

and,

$$\frac{D_{15}\ riprap}{D_{15}\ soil} = \frac{0.1}{0.0046} = 21.9 > 5\ and\ < 40 \qquad \text{(Equation 38-6.14)}$$

Therefore, filter-to-soil interface is OK.

(4) Riprap-to-filter interface.

$$\frac{D_{15}\ riprap}{D_{85}\ \ filter} = \frac{0.6}{0.2} = 3 < 5 \qquad \text{(Equation 38-6.15)}$$

and,

$$\frac{D_{15}\ riprap}{D_{15}\ \ filter} = \frac{0.6}{0.1} = 6 > 5\ and\ < 40 \qquad \text{(Equation 38-6.16)}$$

Therefore, the 2-in. filter material is adequate.

(5) Filter layer thickness. Because the soil-gradation curve and the filter-layer gradation curve are not approximately parallel, use layer thickness of 8 in.

n. Edge Details. Line the entire perimeter. The edge flanks and toe should be as shown in Figure 38-6M. Also see the sketch in Figure 38-6R.

* * * * * * * *

2. Example 38-6.2. The site illustrated in Figure 38-6T and discussed below is migrating laterally towards Route 1; see Figure 38-6T(a). Design a riprap revetment to stabilize the active bank erosion at this site.

The process of developing appropriate channel geometry is illustrated in Figure 38-6T. Figure 38-6T(a) illustrates the location of the design site at position 2 along Route 1. The section illustrated in Figure 38-6T(c) was surveyed at this location and represents the current condition. No previous channel surveys are available at this site. However, data from some old surveys are available in the vicinity of an upstream railroad crossing (location 1). Figure 38-6T(b) 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. Because locations 1 and 2 are along bends of similar radii, it can be assumed that a similar phenomenon occurs at location 2. A thalweg located immediately adjacent to the channel bank represents the hydraulically-worst situation section at location 2. Therefore, the surveyed section at location 2 is modified to reflect this. 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. 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 38-6T(c).

Additional site conditions are as follows:

a. flow conditions are gradually varying;

b. channel characteristics are as described above;

c. topographic survey indicates the following:

(1) channel slope = 0.0024
(2) channel width = 300 ft; and

(3) bend radius = 1200 ft;

d. channel bottom is armored with cobble-sized material with D_{50} of approximately 6 in;

e. bank soils are silty sand with the soil characteristics as follows:

D_{85} = 0.0043 ft;
D_{50} = 0.0015 ft;
D_{15} = 0.0005 ft; and
K (permeability) = $3.3 \, x \, 10^{-6}$ ft/s;

f. available rock riprap has a specific gravity of 2.60 and is described as angular;

g. 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; and

h. bank height along the cut banks is approximately 9.0 ft.

Figure 38-6U provides the completed Example 2. Figures 38-6V and 38-6W summarize the data used in Example 2.

a. Compile Field Data.

(1) See the given information for this example.
(2) See the site history described above.

b. Design Discharge.

(1) Given as 46,620 ft^3/s.

(2) From backwater analysis of this reach, it is determined that the discharge confined to the main channel, Q_{mc}, is 34,610 ft^3/s.

c. Design Cross Section.

(1) 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 38-6V illustrates the existing channel section.

(2) To minimize loss of bank vegetation and to limit the encroachment of the channel on adjacent lands, a 2H:1V bank slope is to be used.

(3) The current bank height along the cut banks is 9.0 ft.

d. Compute Design Water Surface.

(1) Determine roughness coefficient n = 0.042. This represents the average reach n used in the backwater analysis.

(2) Compute flow depth:

(a) Flow depth determined from backwater analysis. The maximum main channel depth is determined to be d_{max} = 15.25 ft.

(b) Hydraulic radius for main channel. R = 10.5 ft from the backwater analysis. *R assumed* of 10 ft is approximately equal to *R actual*. Therefore, n as computed is OK.

e. Determine Other Design Parameters. From the backwater analysis, the main-channel values are as follows:

A = 2750 ft^2
V_a = 12.6 ft/s
$d_a = d$ = 12 ft

f. Bank angle correction factor.

θ = 2H:1V
φ = 41 deg, from Figure 38-6B and Figure 38-6W
K_1 = 0.73 from Figure 38-6A

g. Determine riprap size.

(1) Using Figure 38-6A, D_{50} = 0.9 ft

(2) Riprap specific gravity = 2.60 (given)
Stability factor = 1.6
gradually varying flow, sharp bend, bend radius to width = 4
C = 1.6

(3) no piers or abutments to evaluate for this example, therefore $C_{p/a}$ = 1

(4) Corrected riprap size.

$D'_{50} = D_{50}(1.6)(1.0) = 1.44$ ft

h. This step is not applicable.

i. Surface Waves. Surface waves determined not to be a problem at this site.

j. Select design riprap size, gradation, and layer thickness for preliminary design of waterway area.

D_{50} size: Recommend AASHTO 0.25-ton-class riprap
$D_{50} = 1.8$ ft

Gradation: See Figure 38-6U.

Layer thickness, T:
$T = 2D_{50} = 2(1.8) = 3.6$ ft, or $T = D_{100} = 2.3$ ft
Use $T = 3.6$ ft

k. 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, 1.5W, downstream of the bend exit.

l. Vertical Extent of Protection. Riprap the entire channel bank from top of bank to below the depth of anticipated scour. Scour depth is evaluated as illustrated in Section 38-5.07, as follows:

$d_s = 6.5D_{50}^{-0.11}$ (Equation 38-5.3)
$d_s = 6.5\,(0.5)^{-0.11} = 7.0$ ft

Adding this to the observed maximum depth yields a potential maximum scour depth of 15 + 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.

m. Filter Layer Design.

(1) Filter-material size (Figure 38-6K).

$$\frac{D_{15}\ coarser\ layer}{D_{15}\ \ finer\ layer} < 5 < \frac{D_{15}\ coarser\ layer}{D_{15}\ \ finer\ layer} < 40 \qquad \text{(Equation 38-6.17)}$$

(2) Riprap-to-soil interface.

$$\frac{D_{15}\ riprap}{D_{85}\ soil} = \frac{0.5}{.004} = 125 > 5 \qquad \text{(Equation 38-6.18)}$$

and,

$$\frac{D_{15}\ riprap}{D_{15}\ soil} = \frac{0.5}{0.005} = 1000 > 40 \qquad \text{(Equation 38-6.19)}$$

Therefore, a filter layer is needed. Try a 0.5-in. uniformly-graded fine-gravel filter with gradation characteristics as shown in Figure 38-6J..

(3) Filter-to-soil interface.

$$\frac{D_{15}\ \ filter}{D_{85}\ soil} = \frac{0.015}{0.004} = 3.8 < 5 \qquad \text{(Equation 38-6.20)}$$

and,

$$\frac{D_{15}\ riprap}{D_{15}\ soil} = \frac{0.015}{0.0005} = 31 > 5\ and\ < 40 \qquad \text{(Equation 38-6.21)}$$

Therefore, filter-to-soil interface is OK.

(4) Riprap-to-filter interface.

$$\frac{D_{15}\ riprap}{D_{85}\ \ filter} = \frac{0.5}{0.10} = 5 \le 5 \qquad \text{(Equation 38-6.22)}$$

and,

$$\frac{D_{15}\ riprap}{D_{15}\ \ filter} = \frac{0.5}{0.015} = 33 > 5\ and\ < 40 \qquad \text{(Equation 38-6.23)}$$

Therefore, the 0.5-in. filter material is adequate.

(5) Filter Layer Thickness. Because the soil-gradation curve and filter layer, riprap, and bank soil are approximately parallel, use a layer thickness of 8 in.

n. Edge Details. For the flank and toe details, see Figure 38-6W.

Anticipated scour depth below the existing channel bottom at the bank, d'_s, is the depth of scour computed in Step 12 minus the current bed elevation at the bank (see Figure 38-6V): 22.3 ft – 12.3 ft = 10 ft.

Rock quantity required below the existing bed is determined as follows:

$$R_q = 1.5Td'_s \sin^{-1}\theta \qquad \text{(Equation 38-6.24)}$$

Where: R_q =required riprap quantity per foot of bank, ft^2
θ = bank angle with the horizontal, deg
T = riprap-layer thickness, ft

R_q = 3 (2.24)(1.1)(1.5) = 11.1 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 38-6W illustrates the resulting toe-trench detail.

38-6.02 Wire-Enclosed Rock

As described in Section 38-4.03, a wire-enclosed rock, or gabion, revetment consists of rectangular wire mesh baskets filled with rock. The most common types of wire-enclosed revetment are mattresses and stacked blocks. The wire cages which make up the mattresses and gabions are available from commercial manufacturers.

A rock-and-wire-mattress revetment consists of flat wire baskets or units filled with rock that are laid end to end and side to side on a prepared channel bed or bank. The individual mattress units are wired together to form a continuous revetment mattress.

A stacked-block gabion revetment consists of rectangular wire baskets which are filled with stone and stacked in a stepped-back fashion to form the revetment surface. It is commonly used at the toe of an embankment slope as a toewall, which helps to support other upper-bank revetments and prevents undermining.

38-6.02(01) Mattress Gabion

Components of a rock-and-wire-mattress 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.

1. General. A rock-and-wire-mattress revetment can be constructed from commercially-available wire units as illustrated in Figures 38-6X and 38-6Y, or from available wire-fencing material as illustrated in Figure 38-6Z. The use of commercially-available basket units is the most common practice and the least expensive.

   A rock-and-wire-mattress revetment can be used to protect either the channel bank as illustrated in Figure 38-6X, or the entire channel perimeter (Figure 38-6Y). If used for bank protection, this revetment consists of a toe section and upper-bank paving (see Figure 38-6X). As illustrated in Figure 38-6X, a variety of toe designs can be used. Design emphasis should be placed on toe design and filter design. These designs are discussed later. The vertical and longitudinal extent of the mattress should be based on guidelines provided in Section 38-5.07.

2. Bank and Foundation Preparation. The channel bank should be graded to a uniform slope. The graded surface, either on the slope or on the streambed 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 in. Blunt or sharp objects such as rocks or tree roots protruding from the graded surface should be removed.

3. Mattress-Unit Size and Configuration. Individual mattress units should be of a size that is easily handled on site. Commercially-available gabion units are available in standard sizes as indicated in Figure 38-6AA. Manufacturers' literature indicates that alternative sizes can be manufactured if 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. Diaphragms should be installed at a nominal 3-ft spacing within each of the gabion units to provide the recommended compartmentalization (see Figure 38-6BB).

On a slope steeper than 1H:3V, and in an environment subject to high stresses (area prone to high flow velocity, debris flow, ice flow, etc.), diaphragms should be spaced at minimum intervals of 2 ft to prevent movement of the stone inside the basket.

The thickness of the mattress is determined by the erodibility of the bank soil, the maximum velocity of the water, and the bank slope. The minimum thickness required for given conditions is tabulated in Figure 38-6CC. These values are based on observations of a number of mattress installations which assume a filling material in the size range of 3 in to 6 in.

The mattress thickness should be at least as thick as two overlapping layers of stone. The thickness of a mattress used as a bank-toe apron should exceed 12 in. The range is 12 in to 20 in. The thickness of a mattress revetment can vary according to need by utilizing gabions of different depths as illustrated in Figure 38-6X(d).

4. Stone Size. The maximum stone size should not exceed the thickness of the individual mattress units. The stone should be well-graded within the sizes available. Seventy percent 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 Figure 38-6AA.

   The common median-stone size used in a mattress design ranges from 3 in. to 6 in. for a mattress less than 12 in. thick. For a thicker mattress, rock with a median size up to 12 in. is used.

5. Stone Quality. The stone should satisfy the quality requirements for dumped-rock riprap.

6. 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 0.007 ft in diameter. The wire for edges and corners is approximately 0.009 ft in diameter. Manufacturers' instructions for field assembly of basket units should be followed.

   Galvanized wire baskets may be safely used in fresh water or where the pH of the liquid in contact with it is not greater than 10.

   For a highly-corrosive condition, such as in a salt-water environment, industrial area, polluted stream, or soil such as muck, peat, or cinders, a polyvinyl chloride (PVC) coating should be placed over the galvanized wire. It should be capable of resisting deleterious effects of natural weather exposure and immersion in salt water and should not show a material difference in its initial characteristics over time.

7. Edge Treatment. The toe, head, and flanks of a rock-and-wire mattress revetment installation require treatment to prevent damage from undermining. Figure 38-6X illustrates the possible toe-treatment 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. Where little toe scour is expected, the apron can be replaced by a single-course gabion toewall which helps to support the revetment and prevents undermining. Where an excessive amount of toe scour is anticipated, both an apron and a toe wall can be used.

   To provide extra strength at the revetment flanks, mattress units having additional thickness be used at the upstream and downstream edges of the revetment (see Figure 38-6EE). A thin layer of topsoil should be spread over the flank units to form a soil layer to be seeded once the revetment installation is complete. The head of a rock-and-wire-mattress revetment can be terminated at grade as illustrated in Figure 38-6X.

8. Filter Design. Individual mattress units will act as a crude filter and a pavement unit if filled with overlapping layers of hand-size stones. However, the need for a filter should be investigated. If necessary, a layer of permeable membrane cloth (geotextile) woven from synthetic fibers, or a 4-in. to 6-in. layer of gravel should be placed between the silty bank and the rock-and-wire-mattress revetment to further inhibit washout of fines.

9. Construction. Construction methods for a rock-and-wire mattress vary with the design and purpose for which the protection is provided. Details are illustrated in Figures 38-6X, 38-6Y, and 38-6Z. A rock-and-wire-mattress revetment may be fabricated where it is to be placed or at an off-site location. Fabrication at an offsite location requires that the individual mattress units be transported to the site. Moving and placing the baskets should not damage them by breaking or loosening strands of wire or ties, or by removing the galvanizing or PVC coating. Because of the potential for damage to the wire enclosures, off-site fabrication is not recommended.

   Installation of mattress units above the water line is 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 38-6FF provides an illustration. Where the mattress units must be placed below the water line in relatively shallow water, mattress units can be assembled at a convenient location and then be placed on the bank using a crane as illustrated in Figure 38-6GG. For a deep-water installation, an efficient method of large-scale placement is to fabricate the mattress sections on a barge or pontoon and then launch them into the water at the shore line (see Figure 38-6HH).

38-6.02(02) Stacked-Block Gabion

Components of a stacked-block gabion revetment include the 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 a mattress design.

1. General. A stacked-block gabion revetment is used instead of a gabion mattress where the slope to be protected is steeper than 1H:1V, or where the purpose of the revetment is for flow training. Design methods include flow-training wall, as shown in Figure 38-6 I I(a), or low retaining wall, as shown in Figure 38-6 I I(b).

   A stacked-block gabion revetment must be based on a firm foundation. The foundation or base elevation of the structure should be below the anticipated scour depth. In an alluvial stream where channel-bed fluctuations are common, an apron should be used as illustrated in Figure 38-6 I I(a) and (b). An apron should be used where the estimated scour depth is uncertain.

2. Size and Configuration. The common commercial sizes are listed in Figure 38-6AA. The most common size used is that of width and depth of 3.0 ft. A size of less than 1.0 ft thickness is not practical.

   Configurations include flow-training wall or structural retaining walls. The primary function of a flow-training wall is to establish a normal channel boundary in a river where erosion has created a wide channel, or to realign the river where it is encroaching on an existing or proposed structure. A stepped-back wall is constructed at the desired bank location. Counterforts are installed to tie the wall to the channel bank at regular intervals as illustrated. The counterforts are installed to form a structural tie between the training wall and the natural stream bank and to prevent overflow from scouring a channel behind the wall. Counterforts should be spaced to eliminate the development of eddy or other flow currents between the training wall and the bank which can cause further erosion of the bank. The dead-water zones created by the counterforts so spaced will encourage sediment deposition behind the wall which will enhance the stabilizing characteristics of the wall.

   A retaining wall can be designed in a stepped-back configuration as illustrated in Figure 38-6 I I(b). Structural details and configurations can vary from site to site.

   A gabion wall is a gravity structure, and its design follows standard engineering practice for a retaining structure. Design procedures are available in standard soil mechanics texts or in gabion manufacturers’ literature.

3. <u>Edge Treatment</u>. The upstream and downstream flanks of the revetment should include counterforts. See Figure 38-6 I I(a). The counterforts should be placed 12 ft to 18 ft from the upstream and downstream limits of the structure and should extend a minimum of 12 ft into the bank.

   The toe of the revetment should be protected by placing the base of the gabion wall at a depth below the anticipated scour depth. Where it is difficult to predict the depth of expected scour, or where channel-bed fluctuations are common, a mattress apron should 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.

4. <u>Backfill or Filter Requirements</u>. Gabion-structure 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 a gabion structure permits natural drainage of the supported embankment. However, because material leaching through the gabion wall can become trapped and can cause plugging, a granular backfill material should be used. The backfill should consist of a 2 in. to 12 in. layer of graded crushed stone backed by a layer of fine granular backfill.

5. Basket Fabrication. Commercially-fabricated basket units are formed from galvanized steel wire mesh of triple twist hexagonal weave. Figure 38-6JJ illustrates the details of basket fabrication.

6. <u>Construction</u>. Construction of a gabion installation varies with the design and purpose for which the protection is being provided. Installation methods are shown in Figures 38-6 I I and 38-6JJ.

   As with a mattress gabion, 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 sequence used for installing a stacked-block gabion revetment or wall.

   a. Prepare the revetment foundation. This includes excavation for the foundation and revetment wall.

   b. Place the filter and gabion mattress, if required, on the prepared grade. Sequentially stack the gabion baskets to form the revetment system.

   c. Each basket is unfolded and assembled by lacing the edges together and the diaphragms to the sides.

d. Fill the gabions to a depth of 1.0 ft with stone of 4 in. to 12 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.

e. Wire the adjoining gabions together by their vertical edges. Stack empty gabions onto the filled gabions and wire them at front and back.

f. After the gabion is filled, fold the top shut and wire it to the ends, sides, and diaphragms.

g. Crushed stone and granular backfill should be placed at intervals to help support the wall structure. Backfill be should placed in three-course intervals.

38-6.03 Precast Concrete Blocks

A precast-concrete-block revetment consists of preformed sections which interlock with each other, are attached to each other, or butt together to form a continuous blanket or mat. The concrete blocks which make up the mats differ in shape and method of articulation, but they share certain common features. The features include flexibility, rapid installation, and provision for the establishment of vegetation within the revetment.

38-6.03(01) Block Configurations

Precast-concrete blocks are available in a number of shapes and configurations. Figures 38-6KK, 38-6LL, 38-6MM, 38-6NN and 38-6 OO illustrate commercially-available concrete-block configurations. Other manufacturers' configurations are available. A precast-concrete revetment is bound to rectangular sheets of filter fabric, interlocks individual blocks, or is butted together at the site. The most common method is to join individual blocks with wire cable or synthetic fiber rope.

38-6.03(02) Design Guidelines

Components of a precast-concrete-block revetment design include layout of a general scheme or concept, bank preparation, mattress and block size, slope, edge treatment, filter design, and surface treatment.

As illustrated in Figures 38-6KK, 38-6LL, 38-6MM, 38-6NN and 38-6 OO, precast-concrete blocks are placed on the channel bank as continuous mattresses. Design emphasis should be placed on toe design, edge treatment, and filter design.

1. <u>Bank Preparation</u>. Channel banks should be graded to a uniform slope. Large boulders, roots, or debris should be removed from the bank prior to final grading. Holes, soft areas, and large cavities should be filled. The graded surface, either on the slope or on the stream bed at the toe of the slope on which the revetment is to be constructed, should be true to line and grade. The bank surface should be lightly compacted to provide a solid foundation for the mattress.

2. <u>Mattress And Block Size</u>. The overall mattress size is dictated by the longitudinal and vertical extent required of the revetment system. An articulated block mattress is assembled in sections prior to placement on the bank. Individual mattresses should be constructed to a size that is easily handled on site with available construction equipment. The size of individual blocks is variable from manufacturer to manufacturer. Individual manufacturers have a number of standard sizes of a particular block available. Manufacturers' literature should be consulted in selecting an appropriate block size for a given hydraulic condition.

3. <u>Slope</u>. An articulated precast-block revetment can be used on a bank slope up to 1.5H:1V. However, an earth anchor should be used at the top of the revetment to secure the system against slippage (see Figures 38-6MM and 38-6NN). A precast-block revetment that is assembled by butting individual blocks end to end with no physical connection should not be used on a slope steeper than 3H:1V.

4. <u>Edge Treatment</u>. The toe, head, and flanks require treatment to prevent undermining. Toe treatment includes an apron design as illustrated in Figures 38-6KK and 38-6NN, and a toe-trench design as illustrated in Figures 38-6LL and 38-6MM. As a minimum, a toe apron should extend 1.5 times the anticipated scour depth in the vicinity of the bank toe. If a toe trench is used, the mattress should extend to a depth greater than the anticipated scour depth in the vicinity of the bank toe.

   The edges can be terminated at-grade (Figures 38-6KK, 38-6LL, and 38-6NN) or in a termination trench. A termination trench is recommended in an environment subject to significant erosion (silty or sandy soil, or high velocity), or where failure of the revetment results in significant economic loss. A termination trench provides more protection against failure from undermining and outflanking than an at-grade termination. However, where upper-bank erosion or lateral outflanking is not expected to be a problem, a grade termination may provide an economic advantage.

   For an articulated block, earth anchors should be placed at regular intervals along the top of the revetment (see Figures 38-6LL and 38-6MM). Anchors are spaced based on soil type, mat size, and the size of the anchors. See manufacturers' literature for the recommended spacing.

5. Filter. Prior to installing the mats, a geotextile filter fabric should be installed on the bank to prevent bank material from leaching through the openings in the mattress structure. Although a fabric filter is recommended, graded filter material can be used if it is properly designed and installed to prevent movement of the graded material through the protective mattress.

6. Surface Treatment. The spaces between and within individual blocks located above the low-water line should be filled with earth and seeded so that natural vegetation can be established on the bank (see Figures 38-6LL and 38-MM). This treatment enhances both the structural stability of the embankment and its aesthetic qualities.

38-6.03(03) Construction

Schematics of the types of precast-block revetments discussed above are provided in Figures 38-6KK, 38-6LL, 38-6MM, 38-6NN, and 38-6 OO. More-detailed sketches and information are available from individual manufacturers. Manufacturers also have available information on construction procedures. A manufacturer can provide on-site advice and assistance in the installation of its system.

After the site preparation work is completed, construction follows the following sequence.

1. Excavate toe, flank, and upper bank protection trenches as required.

2. Place filter fabric or graded filter material on the prepared subgrade.

3. Individual mats are attached to a spreader bar and lifted for placement onto the embankment slope. Mats are placed side by side on the bank until the entire prepared surface is covered.

4. Adjacent mats are secured to one another by fastening side-connecting cables and end loops or by pouring side-connecting keys.

5. Optional anchors are placed at the top and flanks of the protection as required.

6. Backfill is spread over the mats and into the open cells or spaces between cells and into the anchor trenches. Anchor trenches are then compacted. The backfill should be seeded and fertilized according to local seasonal conditions.

A non-articulated block revetment, in which the blocks are butted together instead of being physically attached, is constructed in a similar manner. However, the individual blocks must be placed on the bank by hand, one at a time. This results in a much more labor-intensive installation procedure.

38-6.04 Grouted Riprap

38-6.04(01) Design Guidelines

Grouted revetment riprap consists of rock-slope-protection having voids filled with concrete grout to form a monolithic armor. See Section 38-4.05 for additional descriptive information and performance characteristics.

Components of grouted-rock-riprap design include the layout of a general scheme or concept, bank preparation, bank slope, rock quality, grout quality, edge treatment, filter design, and pressure relief.

Grouted riprap is a rigid monolithic bank-protection method. Once complete, it forms a continuous surface. A grouted-riprap section is shown in Figure 38-6PP.

Grouted riprap should extend from below the anticipated channel-bed scour depth to the design highwater level, plus additional height for freeboard.

1. Bank and Foundation Preparation. The bank should be prepared by clearing all trees or debris from the bank and grading the bank surface to the desired slope. The graded surface should not deviate from the specified slope line by more than 6 in. However, a local depression larger than this can be accommodated because initial placement of filter material or rock for the revetment will fill the depression.

   Because grouted riprap is rigid but not extremely strong, support by the embankment must be maintained. To form a firm foundation, the bank surface should be tamped or lightly compacted. Soil permeability similar to that of the natural, undisturbed bank material should 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.

2. Bank Slope. The bank slope for should not be steeper than 1.5H:1V.

3. Rock Quality. Rock used in grouted-rock slope-protection is the same as that used in ordinary rock-slope-protection. However, the values for specific gravity and hardness may be lowered if necessary as the rocks are protected by the surrounding grout. The

rock used in a grouted-riprap installation should be free of fines so that penetration of grout may be achieved.

4. Grout Quality and Characteristics. Grout should consist of concrete with a maximum aggregate size of ¾ in. and a slump of 3 in. to 4 in. A sand mix may be used where roughness of the grout surface is unnecessary, provided sufficient cement is added to provided strength and workability.

   The finished grout should leave face stones exposed for one-fourth to one-third their depth. The surface of the grout should expose a matrix of coarse aggregate. See the INDOT *Standard Specifications* for more information on grout.

5. Edge Treatment. The head, toe, and flanks require 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 38-6PP(a). After excavating to the desired depth, the riprap-slope protection should be extended to the bottom of the trench and grouted. The remainder of the excavated area in the toe trench should be filled with ungrouted riprap. The grout-free riprap provides extra protection against undermining at the bank toe. Edge-treatment designs are illustrated in Figure 38-6PP (a), (b) and (c).

6. Filter Design. A filter is required under the grouted-riprap revetment to provide a zone of high permeability to carry off seepage water and prevent damage to the overlying structure from uplift pressure. A 6-in. 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.

7. Pressure Relief. Weep holes should be provided in the revetment to relieve hydrostatic pressure buildup behind the grout surface; see Figure 38-6PP(a). Weep holes should extend through the grout surface to the interface with the gravel underdrain layer. Weep holes should consist of 3-in. diameter pipes having a maximum horizontal spacing of 6 ft and a maximum vertical spacing of 10 ft. The buried end of the weep hole should be covered with wire screening or a fabric filter of a gage that will prevent passage of the gravel underlayer.

38-6.04(02) Construction

Construction details for a grouted-riprap revetment are illustrated in Figure 38-6PP. The following construction procedures should be followed.

1. Construction procedures include bank clearing and grading, development of foundations, placement of rock-slope protection, grouting of the interstices, backfilling toe and flank trenches, and vegetation of disturbed areas.

2. The rock should be wet immediately prior to commencing the grouting operation.

3. The grout may be transported to the place of final deposit by means of chutes, tubes, buckets, pneumatic equipment, or other mechanical method which will control segregation and uniformity of the grout.

4. Spading and rodding are necessary where penetration is achieved through gravity flow into the interstices.

5. Loads should not be allowed upon the revetment until the proper strength has been developed.

38-6.05 Grouted-Fabric Slope Paving

A grouted fabric-formed revetment is a relatively new development for use on an earth surface subject to erosion. It has been used as an alternative to traditional revetment such as a concrete liner, or riprap on a reservoir, canal, or dike.

A grouted fabric-formed revetment is made by pumping a fluid structural grout, or fine-aggregate concrete, into an in-situ envelope consisting of a double-layer synthetic fabric. During filling, excess mixing water is squeezed out through the permeable fabric, to substantially reduce the water/cement ratio with consequent improvement in the quality of the hardened concrete. An advantage of this type of revetment is that it may be assembled underwater or in a dry location.

38-6.05(01) Types

The types of fabric-formed revetments are as follows.

1. Type 1. Two layers of nylon fabric are woven together at 5 in. to 10 in. centers as indicated in Figure 38-6QQ. These points of attachment serve as filter points to relieve hydrostatic uplift caused by percolation of groundwater through the underlying soil. The finished revetment has a deeply-cobbled or quilted appearance. Mat thickness averages from 2 in. to 6 in.

2. 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. See Figure 38-6RR. 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 uniform cross section and has a lightly-pebbled appearance. Mat thickness averages from 2 in to 10 in.

3. Type 3. Two layers of nylon fabric are interwoven into rectangular block patterns. The points of interweaving serve as hinges to permit articulation of the hardened concrete blocks. The revetment is reinforced with steel cables or nylon rope threaded between the two layers of fabric prior to grout injection and remains embedded in the hardened cast-in-place blocks. Block thickness is controlled with spacer cords in the center of each block.

38-6.05(02) Design Guidelines

The woven fabric for a grouted fabric-formed revetment is available from a number of manufacturers. The designer should consult the manufacturers' literature for designing and selecting the appropriate type of material and thickness for a given hydraulic condition.

38-6.06 Sand-Cement Bags

A sand-cement bag consists of a dry mix of sand and cement placed in a burlap or other suitable bag. It requires firm support from the protected bank. A filter fabric is placed underneath this type of riprap. Adequate protection of the terminals and toe is essential. The riprap has little flexibility and low tensile strength. It is susceptible to damage on a flatter slope where the area of contact between the bags is less.

38-6.06(01) Design Guidelines

Concrete riprap consists of approximately 0.7 ft^3 of class C concrete in a burlap bag or in a cement sack. Each bag should be tied if in a cement sack, or the top should be folded around the bag if in a burlap sack. This type of riprap provides a heavy protection regardless of the requirements of the site. The riprap has little flexibility and low tensile strength, and it is susceptible to damage from floating ice. It requires firm support from the protected bank and requires a filter blanket underneath the riprap. Adequate protection of the terminals and toe is essential. The toe trench must end in firm support and extend below the depth of anticipated scour. Details of terminal protection and cutoff stubs are shown in Figure 38-6SS.

The bags make close contact with each other. Some bond is secured between the bags due to the cement mortar leaking through the porous bags. A flat slope reduces the area of contact between the sacks and thus the bond is less. The slope of the protected embankment is 1.5H:1V. If the slope is as flat as 2H:1V, all sacks after the bottom row should be laid as headers, with the long dimension of each sack in line with the slope, rather than as stretchers, with the long dimension at a right angle to the slope direction.

Concrete riprap in bags can be placed as a dry mix. The riprap is wetted as the work progresses. Some of the bond between sacks can be lost by this method, but it allows the sacks to be filled at a convenient location and brought to the construction site. A well-graded filter blanket is essential to drain the water that is added during construction.

38-6.06(02) Construction

Cloth sacks, about two-thirds filled with concrete, and securely tied, or burlap grain sacks, containing about 0.7 ft^3 of concrete and folded at the top, are immediately placed into position after filling. The fold on each burlap sack is placed underneath the sack for a header, or against the previously-placed sack for a stretcher. Where the protected slope is 1.5H:1V or steeper, a bed consisting of two rows of sacks placed as stretchers is followed by a row of sacks placed as headers. Succeeding rows of sacks are placed as stretchers with joints between sacks staggered (see Figure 38-6SS). Each sack is hand placed and pushed into contact with the adjacent sack. On a slope flatter than 1.5H:1V, all rows after the bed row are placed as headers.

Cutoffs and weep holes should be placed as shown on the plans or as directed by the designer. The finished work should present a neat appearance with parallel rows of sacks. No sack should protrude more than 3 in. from the finished surface.

The riprap should be placed only if the temperature is above 35 ^{o}F and rising. It should be protected from freezing and cured as for concrete.

If placement of concrete riprap in bags is delayed sufficiently to affect the bond between succeeding courses, a small trench about half the depth of a sack should be excavated back of the last row of sacks in place and the trench filled with fresh concrete before the next layer of sacks is placed. At the start of each day's work or if a delay of over 2 h occurs during the placing of successive layers of sacks, the previously-placed sacks should be moistened and dusted with cement to develop bond.

38-6.07 Soil-Cement

Soil-cement is an acceptable method of slope protection for a dam, dike, levee, channel, or highway embankment. Soil-cement can also be used to construct an impervious core in a retention-dam type structure and provide a protective facing. Soil-cement is constructed in a stairstep manner by placing and compacting the soil-cement in horizontal layers stairstepped up the embankment (See Figure 38-6TT). An embankment slope from 2.5H:1V and 4H:1V, and a horizontal-layer width from 7.0 ft to 9.0 ft provide minimum protective thicknesses of about 1.5 ft to 2.5 ft measured normal to the slope.

A number of soil types 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 on placement and compaction methods.

Use of soil-cement does not require further design considerations for the embankment. Proper embankment design procedures should be followed based on individual project conditions, to prevent subsidence or other type of embankment distress.

38-6.07(01) Design Guidelines

1. Top, Toe, and End Features. All extremities of the facing should be tied into non-erodible sections or abutments. Adequate freeboard and carrying the soil-cement to the paved roadway, plus a lower-section detail as shown in Figure 38-6TT, will minimize erosion from behind the crest and under the toe of the facing. The ends of the facing should terminate smoothly in a flat slope or against a rocky abutment. A small amount of rock riprap may be placed over and adjacent to the edges of the soil-cement at its contact with the abutment.

   Where a structure such as a culvert extends through the facing, the areas immediately adjacent to such a structure are constructed by placing and compacting the soil-cement by hand, with small power tools, or by using a lean-mix concrete.

2. Special Conditions. Slope stability is provided for an embankment by means of the strength and impermeability of the soil-cement facing. Further design considerations should not be necessary for a soil-cement-faced embankment. It is necessary to utilize proper design and analysis procedures to ensure the structural and hydraulic integrity of the embankment. Conditions most likely to require analysis include subsidence of the embankment or rapid drawdown of the reservoir or river.

3. 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. If the unit weight and flexural strength of the soil-cement are not known, they should be taken as 120 lb/ft^3 and 150 to 200 lb/in^2, respectively. The layer effect can be ignored.

The post-construction appearance of a pattern of narrow surface cracks of about 10 ft to 20 ft apart is evidence of normal hardening of the soil-cement. Substantial embankment subsidence can 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.

4. Rapid Drawdown. Rapid drawdown exceeding 15 ft or more within 3 to 4 days theoretically produces hydrostatic pressure from moisture trapped in the embankment against the back of the facing. The design concepts that can be used to prevent damage due to rapid drawdown-induced pressure are as follows:

   a. 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;

   b. arbitrarily assuming the weight of the facing sufficient to resist uplift pressures that may develop; and

   c. providing free drainage behind, through, or under the soil-cement facing to prevent adverse hydrostatic pressure.

38-6.07(02) Construction

The method of construction at a 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, however, allows faster production and provides maximum control of mixing operations. In the mixed-in-place method, mixing should be deep enough so that there will be no unmixed seams between the layers. However, excessive striking of the soil-cement below the layer being mixed should be avoided. A compacted layer thickness of 6 in. should be used, with the recommended maximum of 9 in. for efficient, uniform compaction.

The central-plant method should be more economical for all but the smallest project. The contractor should be permitted the option of using either method where the quantity of soil-cement involved is only about several thousand cubic feet. The PCA has sample specifications regarding these construction methods.

38-7.0 REFERENCES

1. U.S. Corps of Engineers, April 1985, *Design of Coastal Revetments, Seawalls, and Bulkheads*, Engineering Manual EM-1110-2-1614.

2. U.S. Department of Transportation, Federal Highway Administration, *Design of Riprap Revetment*, Hydraulic Engineering Circular No. 11, 1989.

3. U.S. Department of Transportation, Federal Highway Administration, *Geosynthetic Design and Construction Guidelines*, FHWA-HI-95-038.

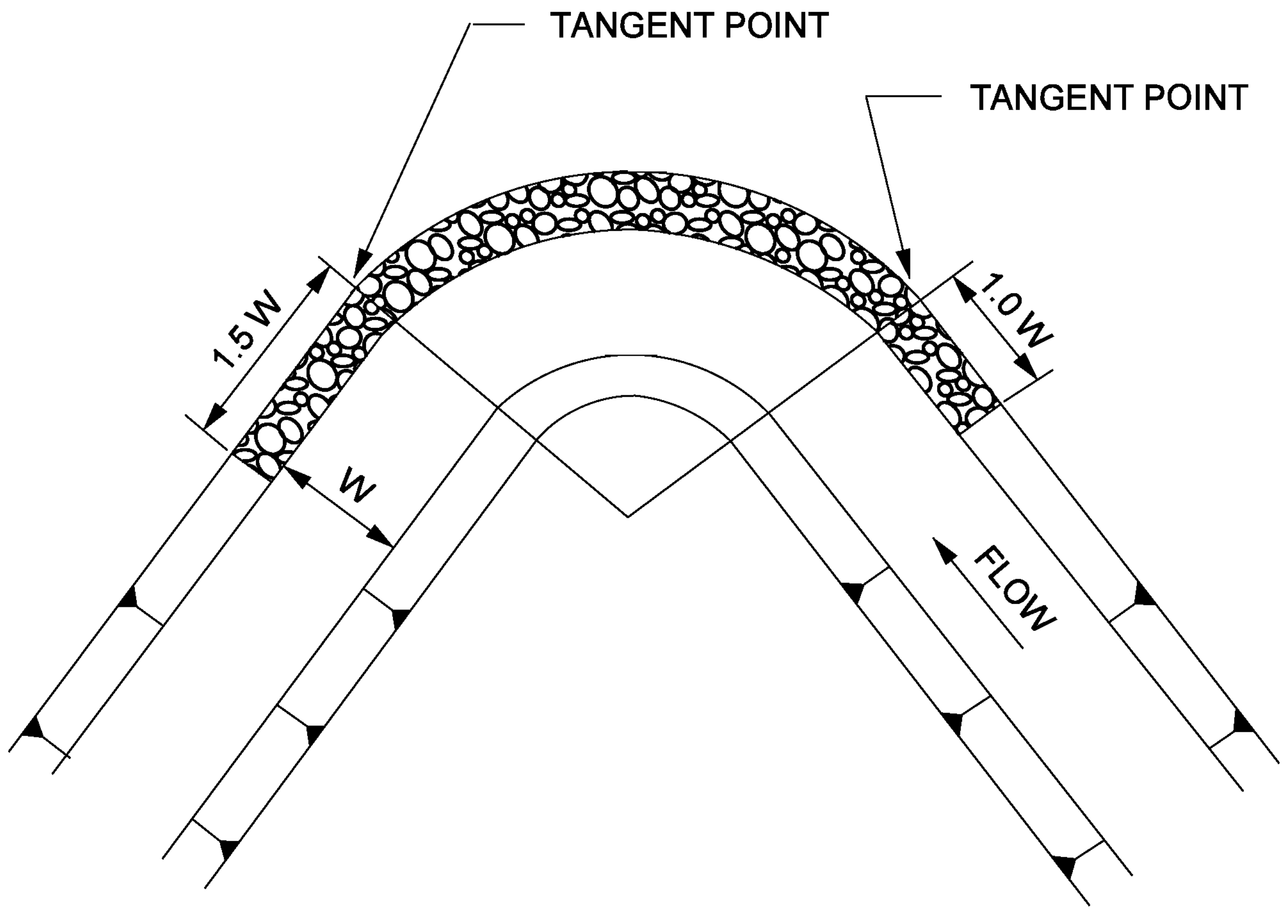
4. U.S. Department of Transportation, Federal Highway Administration, *Stream Stability at Highway Structures*, Hydraulic Engineering Circular No. 20, February 1991.

5. Holtz, Robert D., Christopher, Barry R., and Berg, Ryan R., *Geosynthetic Design & Construction Guidelines*, Participant Notebook, July 29, 1994 Draft, National Highway Institute, US Department of Transportation, Federal Highway Administration.

Symbol	Definition	Unit
AOS	Apparent opening size in filter cloth	in.
A	Coefficient used to determine apparent opening size	--
C	Coefficient, relates free-vortex motion to velocity streamline for equal radius of curvature	--
C_u	Uniformity coefficient	--
D_{50}	Median-bed-material size	ft or in.
D_{15}	15% finer particle size	ft or in.
D_{85}	85% finer particle size	ft or in.
d_{avg}	Average flow depth in main-flow channel	ft
d_s	Estimated probable maximum depth of scour	ft
g	Gravitational acceleration, 32.2 ft/s^2	ft/s^2
H	Wave height	ft
k	Permeability	ft/s or in./s
K_1	Correction term reflecting bank angle	--
n	Manning's roughness coefficient	--
O_{95}	Opening size in geotextile material for which 95% of the openings are smaller	in.
Q_{mc}	Discharge in zone of main-channel flow	ft^3/s
R	Hydraulic radius	ft
R	Wave runup	ft
R_o	Mean radius of channel centerline at bend	ft
S_f, S	Friction slope or energy-grade-line slope	ft/ft
SF	Stability factor	--
S_s, s	Specific gravity of riprap material	--
T	Top width of channel between its banks	ft
V	Velocity	ft/s
V_a	Mean channel velocity	ft/s
W_{50}	Weight of median particle	lb
Z	Superelevation of water surface	ft
γ	Unit weight of riprap material	lb/ft^3
θ	Bank angle with the horizontal	deg
Φ	Riprap material's angle of repose	deg

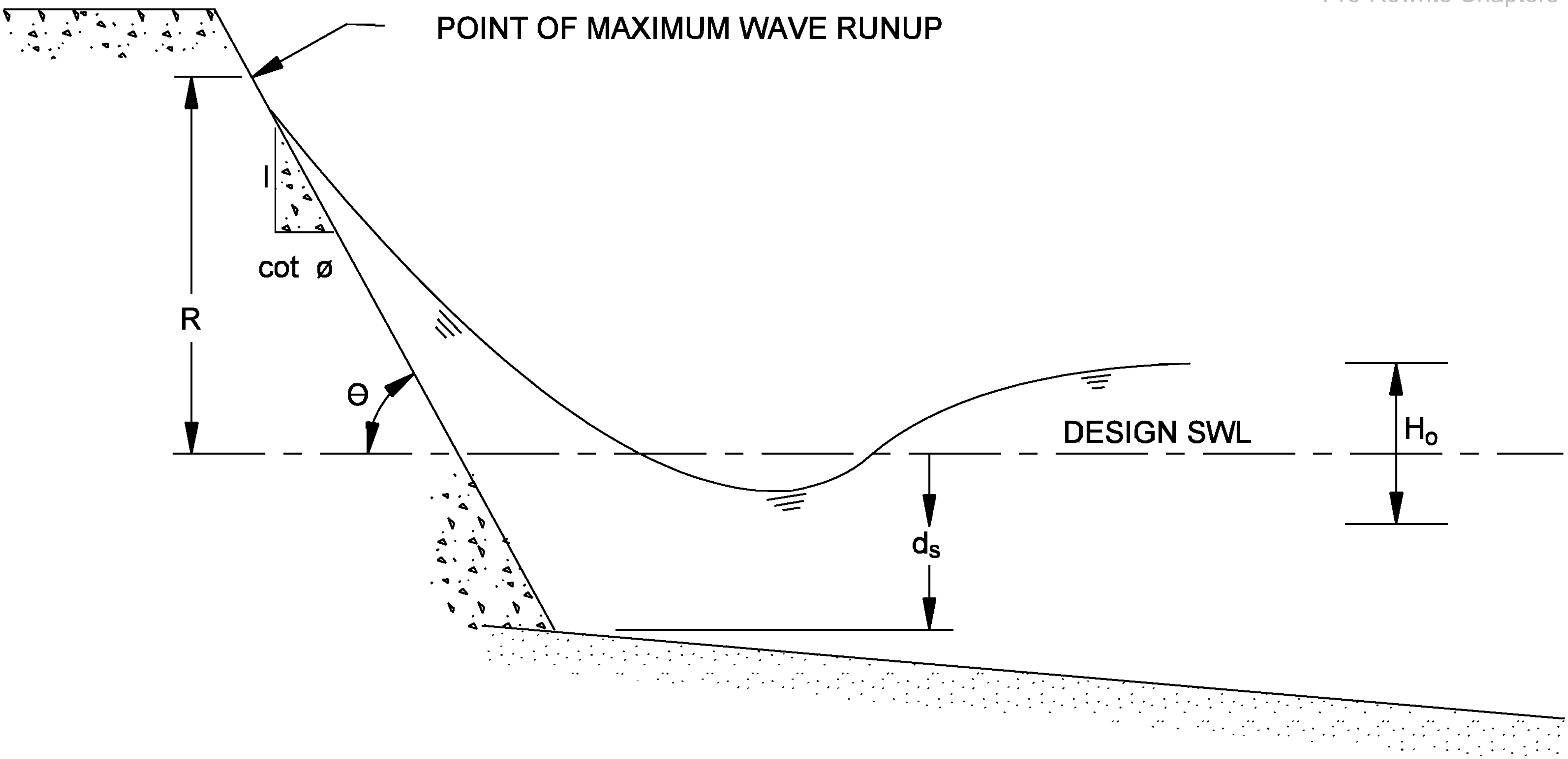
SYMBOLS AND DEFINITIONS

Figure 38-1A



LONGITUDINAL EXTENT OF REVETMENT PROTECTION

Figure 38-5A



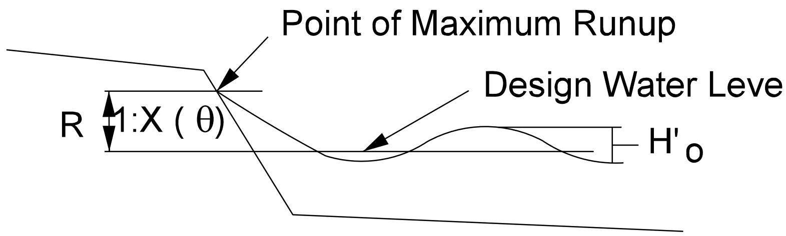
WAVE HEIGHT DEFINITION SKETCH

Figure 38-5B

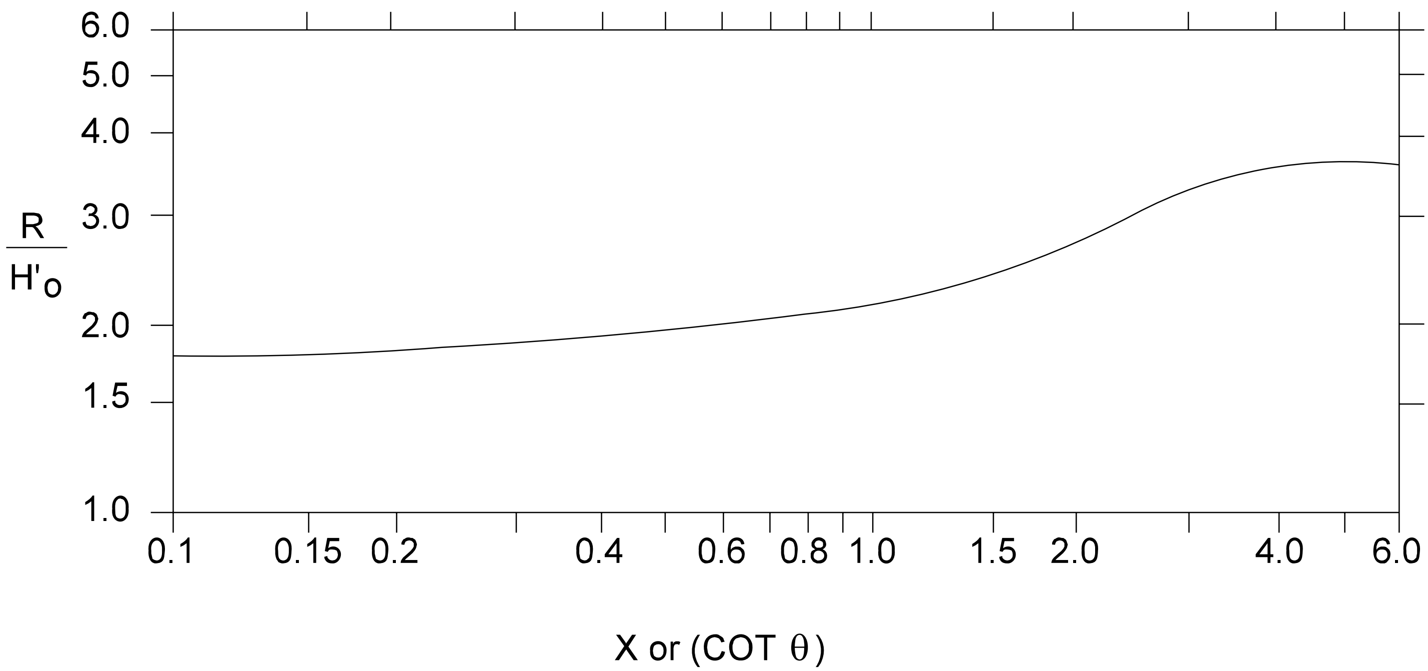
Slope-Surface Characteristic	Placement Method	Correction Factor
Concrete blocks, voids < 20%	fitted	0.90
Concrete blocks, 20% ≤ voids < 40%	fitted	0.70
Concrete blocks, 40% ≤ voids ≤ 60%	fitted	0.50
Concrete pavement	---	1.00
Grass	---	0.85 – 0.90
Grouted rock	---	0.90
Rock riprap, angular	random	0.60
Rock riprap, hand-placed or keyed	keyed	0.80
Rock riprap, round	random	0.70
Wire-enclosed rocks or gabions	---	0.80

CORRECTION FACTORS FOR WAVE RUNUP

Figure 38-5C



R = Wave Run Up Height (ft)
H'_o = Wave Height (ft)
θ = Bank Angle with the Horizontal

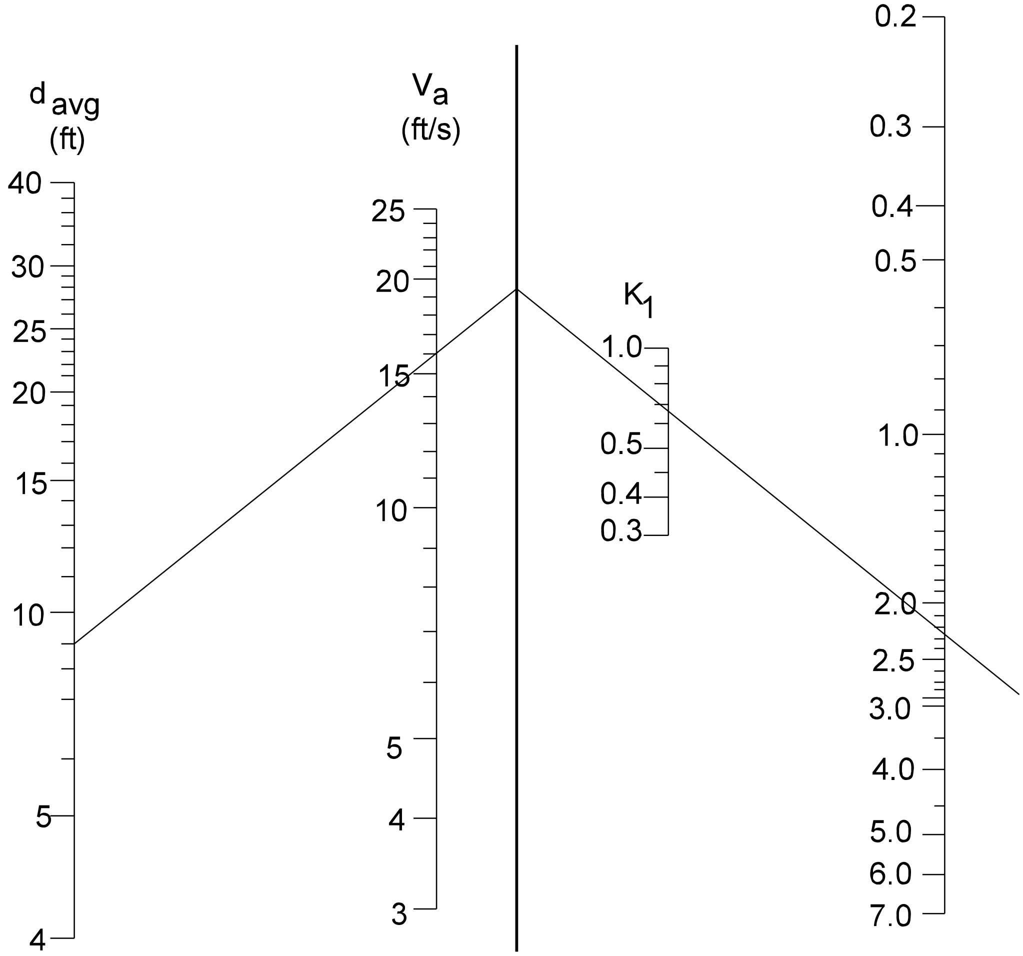


WAVE RUNUP ON SMOOTH, IMPERMEABLE SLOPES

Figure 38-5D

$$D_{50} = 0.001 V_a^3/(d_{avg}^{1/2} K_1^{3/2}$$

D_{50} = Median Riprap Size (ft)
V_a = Average velocity in main channel (ft/s)
d_{avg} = Average depth in main channel (ft)
K_1 = Bank angle correction term



Example

Given:

V_a = 16 ft/s
d_{avg} = 9 ft
K_1= 0.72

Find:

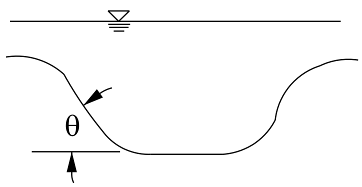
D_{50}

Solution:

D_{50} = 2.25 ft

RIPRAP SIZE RELATIONSHIP

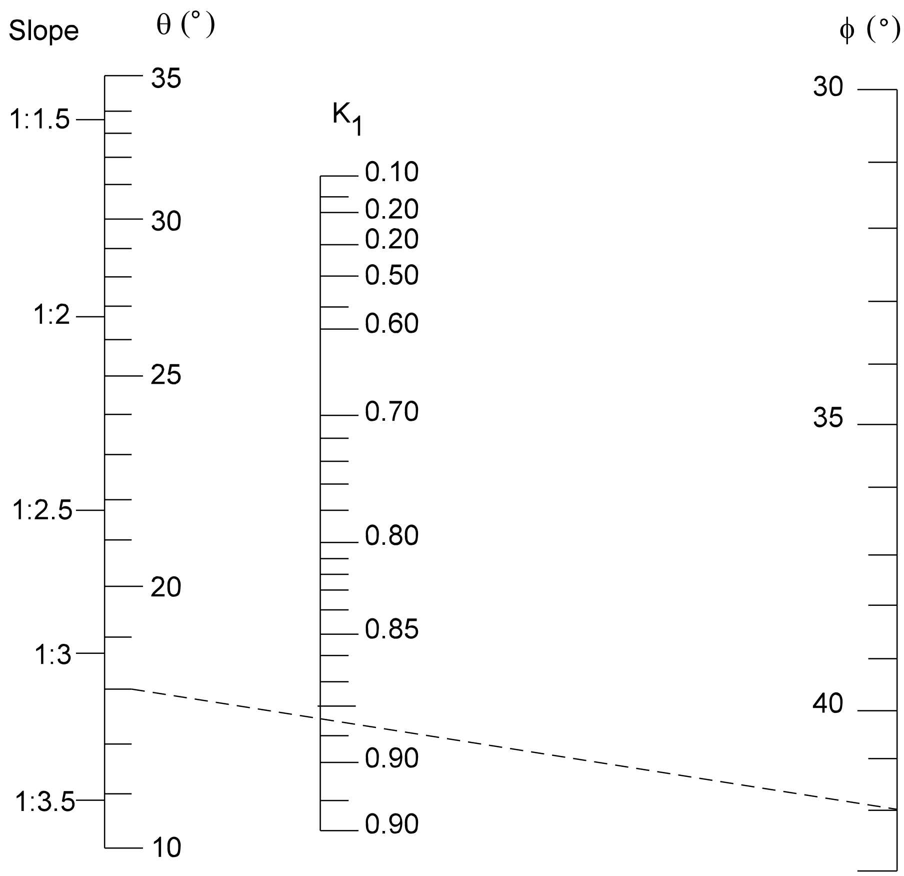
FIGURE 38-6A



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

θ = Bank angle with horizontal

ϕ = Material angle of repose (See chart 4)

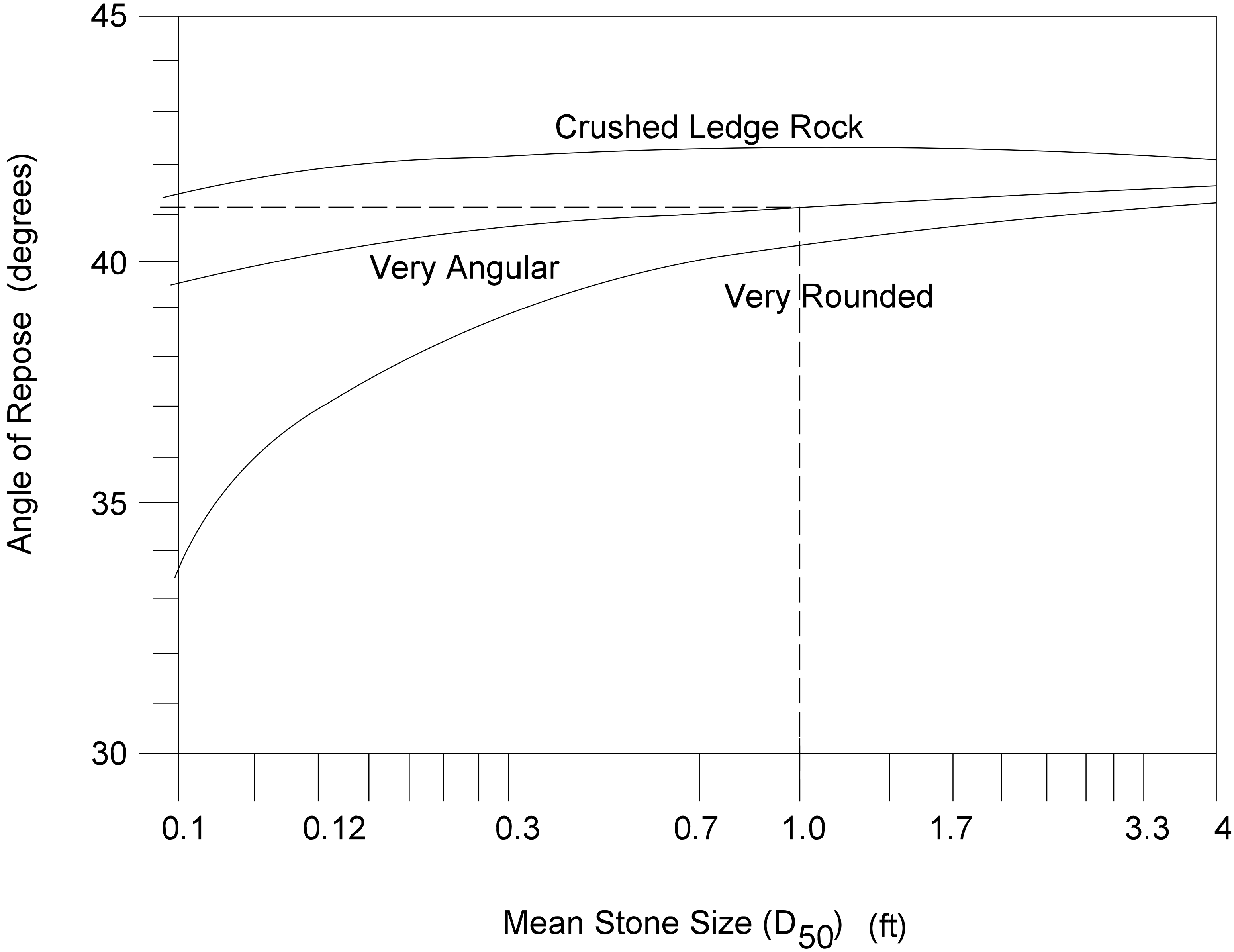


Example

Given:	Find:	Solution:
θ = 16°	K_1	ϕ = 42°
Very Angular		K_1 = 0.885
D_{50} = 1.5 ft		

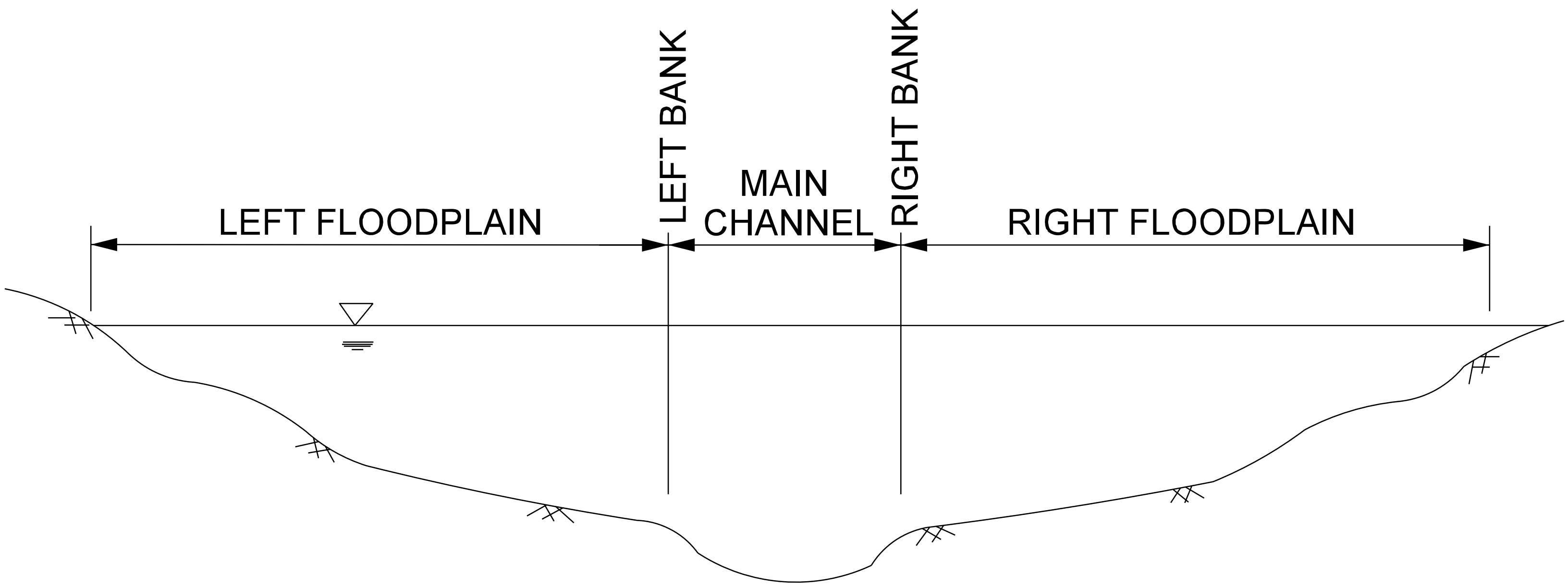
BANK ANGLE CORRECTION FACTOR (K_1) NOMOGRAPH

Figure 38-6B



ANGLE OF REPOSE OF RIPRAP IN TERMS
OF MEAN SIZE AND SHAPE OF STONE

Figure 38-6C

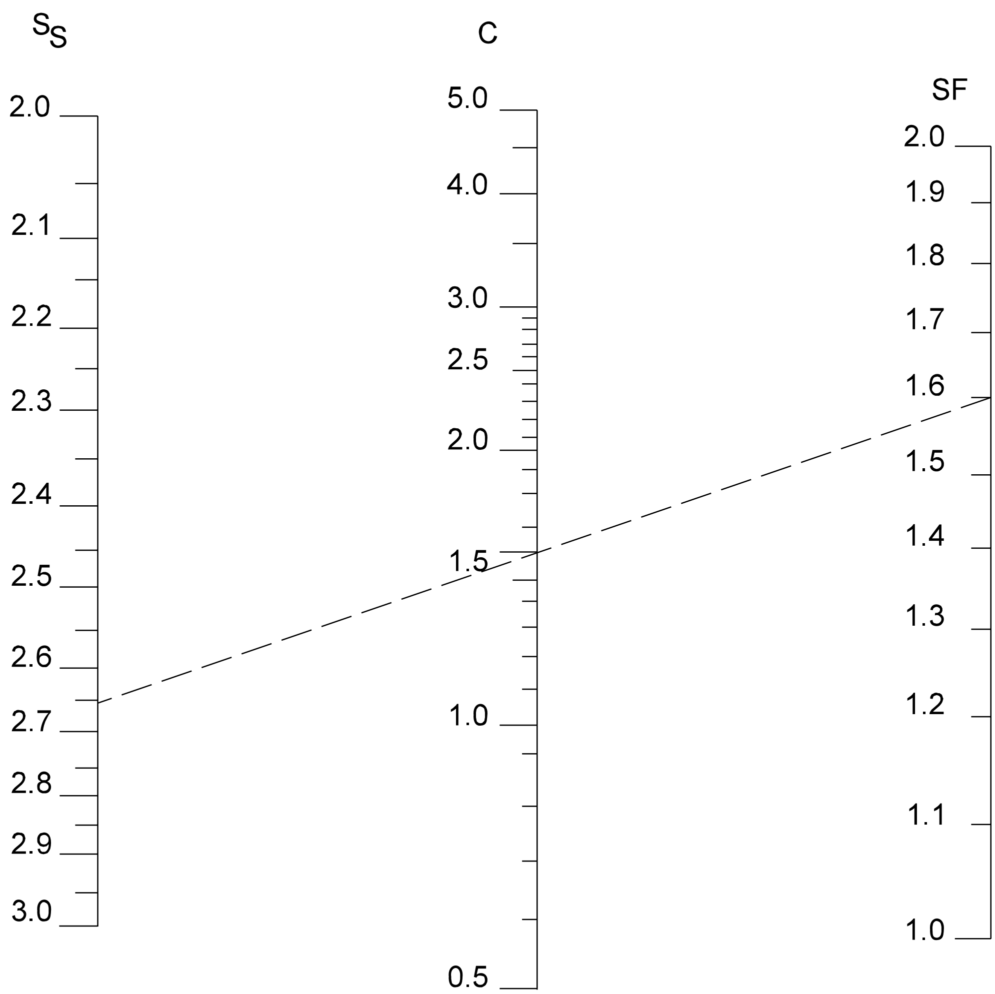


DEFINITION SKETCH
(Channel Flow Distribution)

Figure 38-6D

$C = 1.61 SF^{1.5} / (S_S - 1)^{1.5}$

CORR = D_{50} CORRECTION FACTOR
SF = STABILITY FACTOR
S_S = SPECIFIC GRAVITY OF ROCK



Example:

Given:	Find:	Solution:
S_S = 2.65	C	C = 1.59
SF = 1.60		

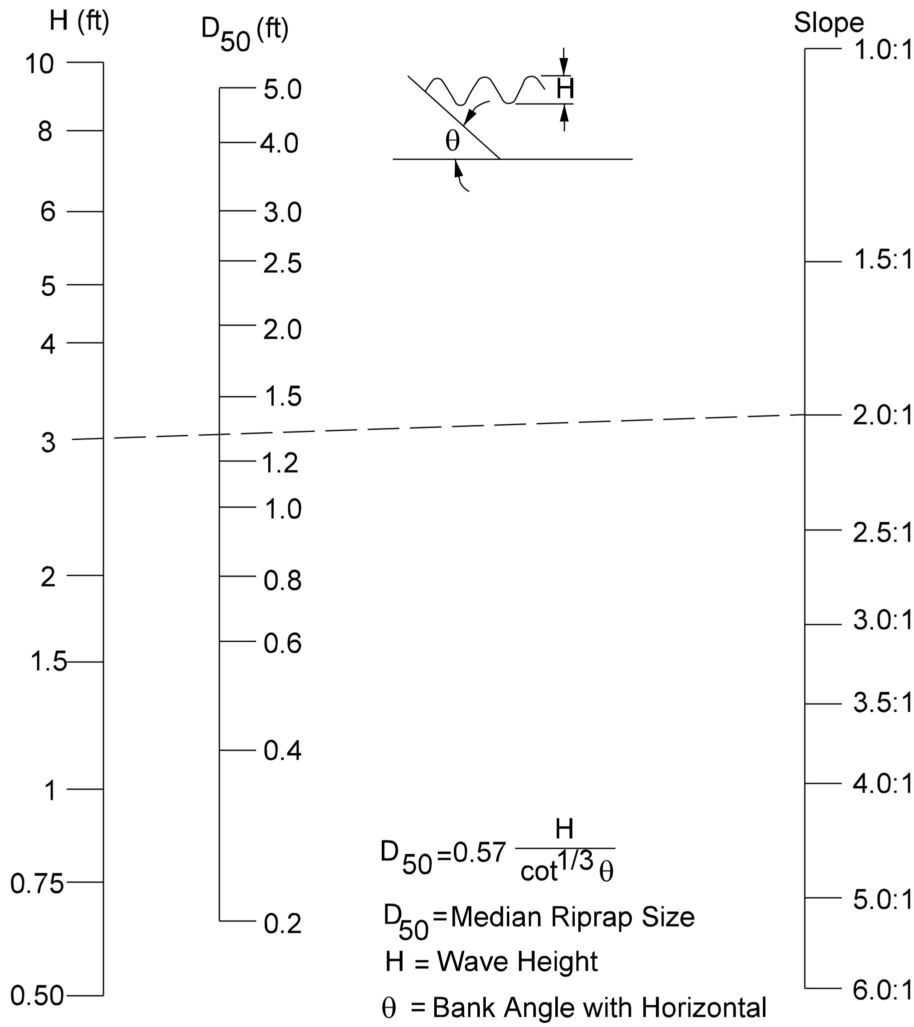
CORRECTION FACTOR FOR RIPRAP SIZE

Figure 38-6E

Conditions	Stability Factor Range
Uniform flow. Straight or mildly-curving reach, curve radius/channel width ≥ 30. Impact from wave action and floating debris is minimal. Little or no uncertainty in design parameters.	1.0 – 1.2
Gradually-varying flow. Moderate bend curvature, 10 < curve radius/channel width < 30. Impact from waves or floating debris moderate.	1.21 – 1.6
Approaching rapidly-varying flow. Sharp bend curvature, curve radius/channel width ≤ 10. Significant impact potential from floating debris or ice. Significant boat-generated waves, 1 ft to 2 ft. High flow turbulence. Turbulently mixing flow at bridge abutment. Significant uncertainty in design parameters.	1.61 – 2.0

GUIDELINES FOR SELECTION OF STABILITY FACTORS

Figure 38-6F



Example

Given:
Slope = 1V:2H
H = 3.0 ft

Find:
D_{50}

Solution:
D_{50} = 1.3

HUDSON RELATIONSHIP FOR RIPRAP SIZE REQUIRED TO RESIST WAVE EROSION

Figure 38-6G

Stone-Size Range (ft)	Stone-Weight Range (lb)	Percent of Gradation Smaller Than
$1.5D_{50}$ to $1.7D_{50}$	$3.0W_{50}$ to $5.0W_{50}$	100
$1.2D_{50}$ to $1.4D_{50}$	$2.0W_{50}$ to $2.75W_{50}$	85
$1.0D_{50}$ to $1.15D_{50}$	$1.0W_{50}$ to $1.5W_{50}$	50
$0.4D_{50}$ to $0.6D_{50}$	$0.1W_{50}$ to $0.2W_{50}$	15

ROCK RIPRAP GRADATION LIMITS

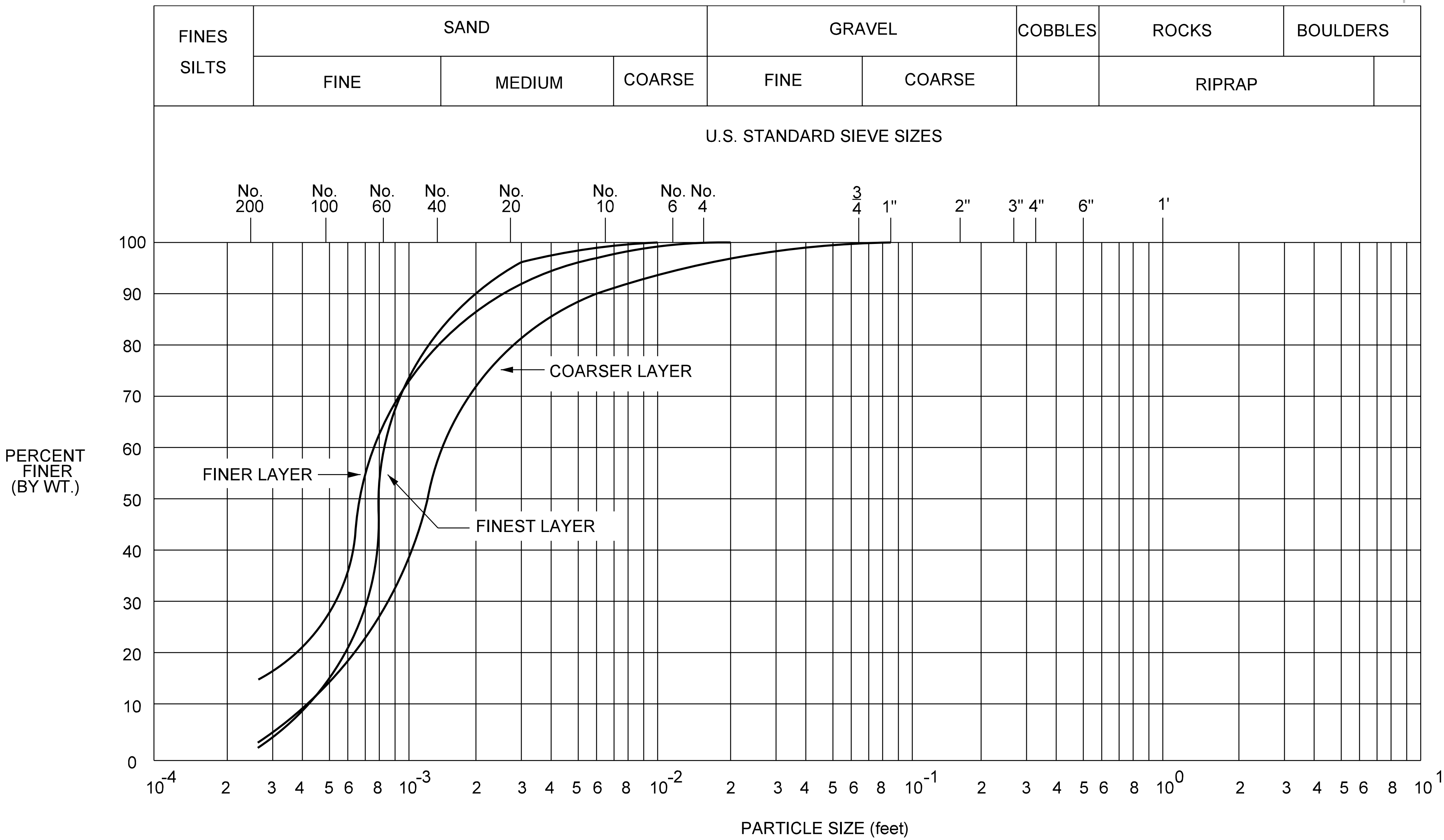
Figure 38-6H

Size, in.	Percent Smaller		
	Revetment	Class 1	Class 2
30			100
24		100	85-100
18	100	85-100	60-80
12	90-100	35-50	20-40
6	20-40	10-30	0-20
3	0-10	0-10	0-10
Minimum Depth of Riprap	18 in.	24 in.	30 in.

Note: The material should be coarse aggregate, Class F or higher. See the INDOT Standard Specifications *for more information.*

RIPRAP GRADATION REQUIREMENTS

Figure 38-6 I



MATERIAL GRADATION

Figure 38-6J

ROUTE: _________ DES. NO.: _______________ PROJECT NO.: _______________________________
DESCRIPTION: ____
Prepared By: _______________________ Date: ___________ Checked By: ________________________ Date: __________

GRANULAR FILTER

LAYER	DESCRIPTION	D_{15} (in.)	D_{85} (in.)	RATIO	$D1 = \frac{D_{15}Coarse}{D_{85}Fine}$	$D1 < 5 < D2$?	$D2 = \frac{D_{15}Coarse}{D_{15}Fine}$	$D2 < 40$?
						Yes No		Yes No
						Yes No		Yes No
						Yes No		Yes No
						Yes No		Yes No
						Yes ☐ No ☐		Yes ☐ No ☐
						Yes ☐ No ☐		Yes ☐ No ☐
						Yes ☐ No ☐		Yes ☐ No ☐

SUMMARY

LAYER DESCRIPTION	D_{15} (in.)	D_{85} (in.)	THICKNESS (in.)

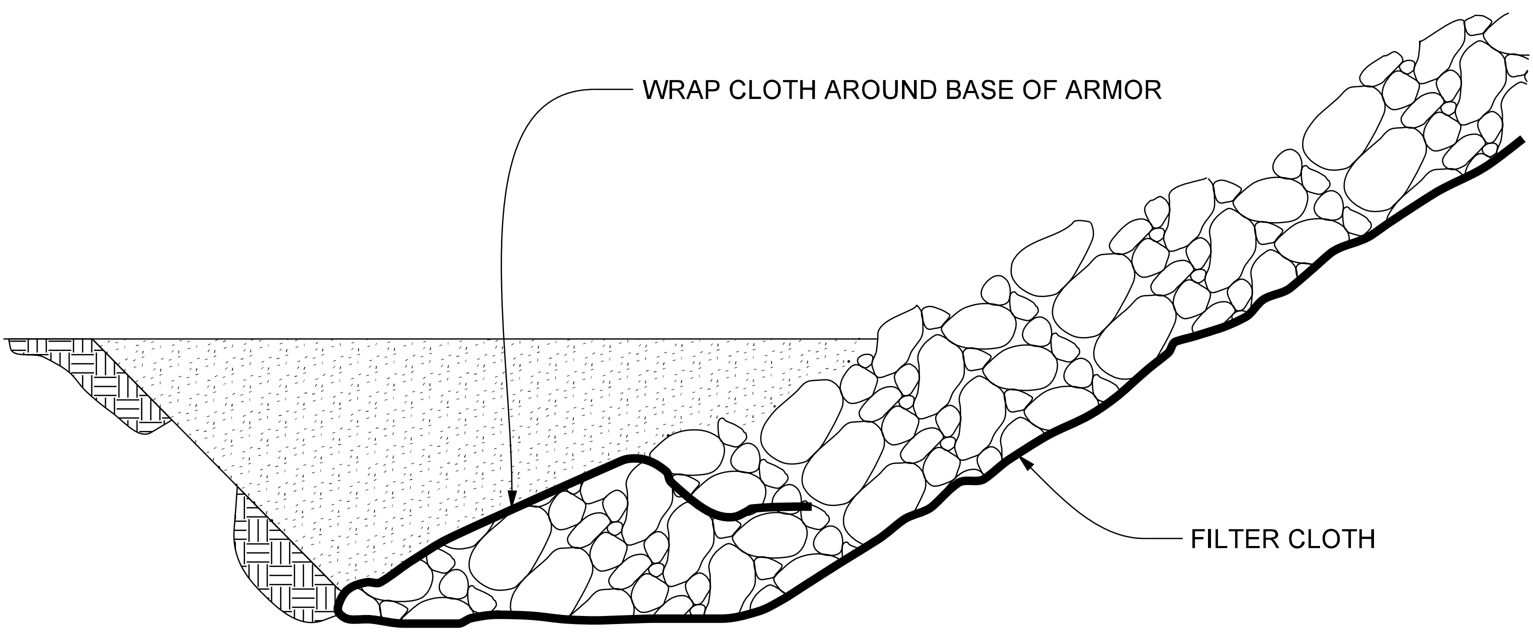
FABRIC FILTER

Click When Done

Clear Data

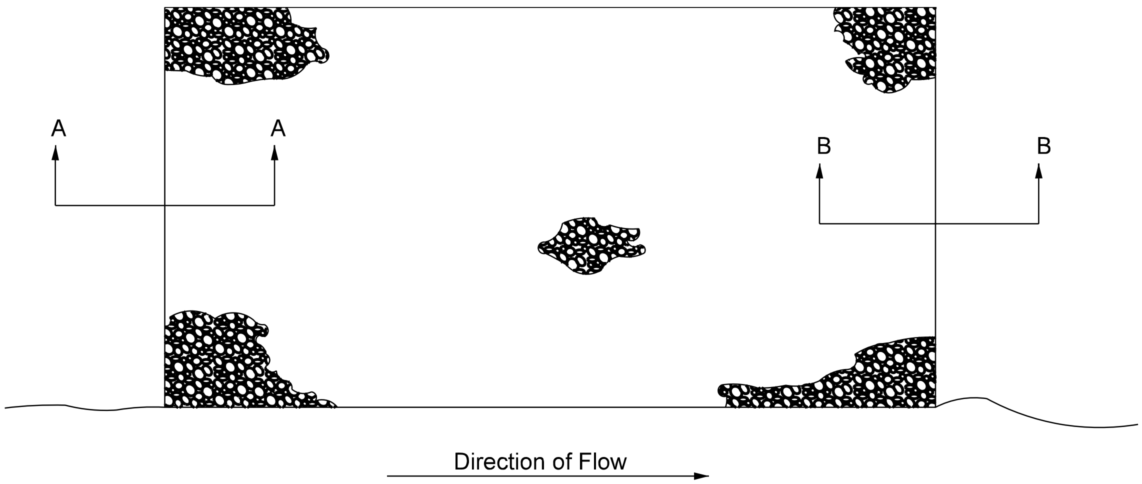
PHYSICAL PROPERTIES CLASS: ______________
HYDRAULIC PROPERTIES:
PIPING RESISTANCE < 50% PASSING #200 AOS < 24 mils
PIPING RESISTANCE < 50% PASSING #200 AOS < 12 mils
PERMEABILITY: SOIL PERMEABILITY < FABRIC PERMEABILITY
SELECTED FILTER FABRIC SPECIFICATIONS: ____

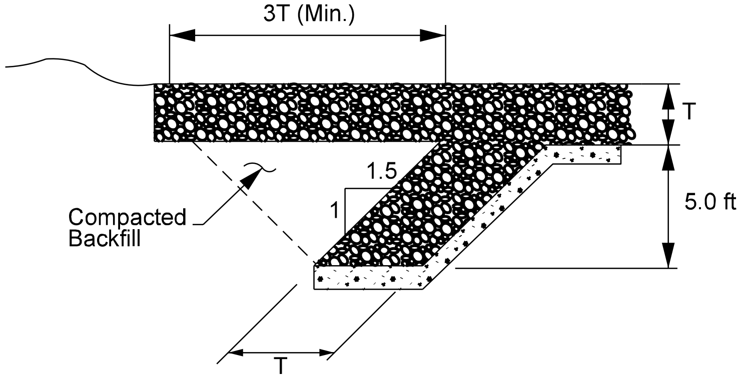
FILTER DESIGN



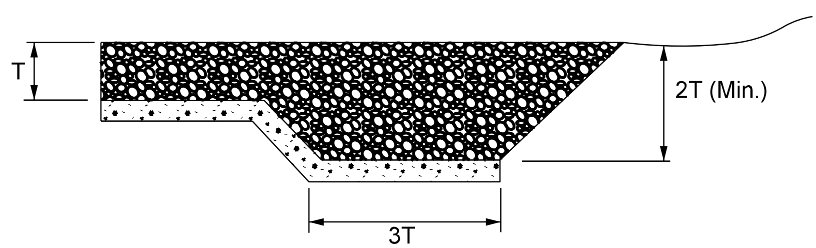
GEOTEXTILE FILTER

Figure 38-6L





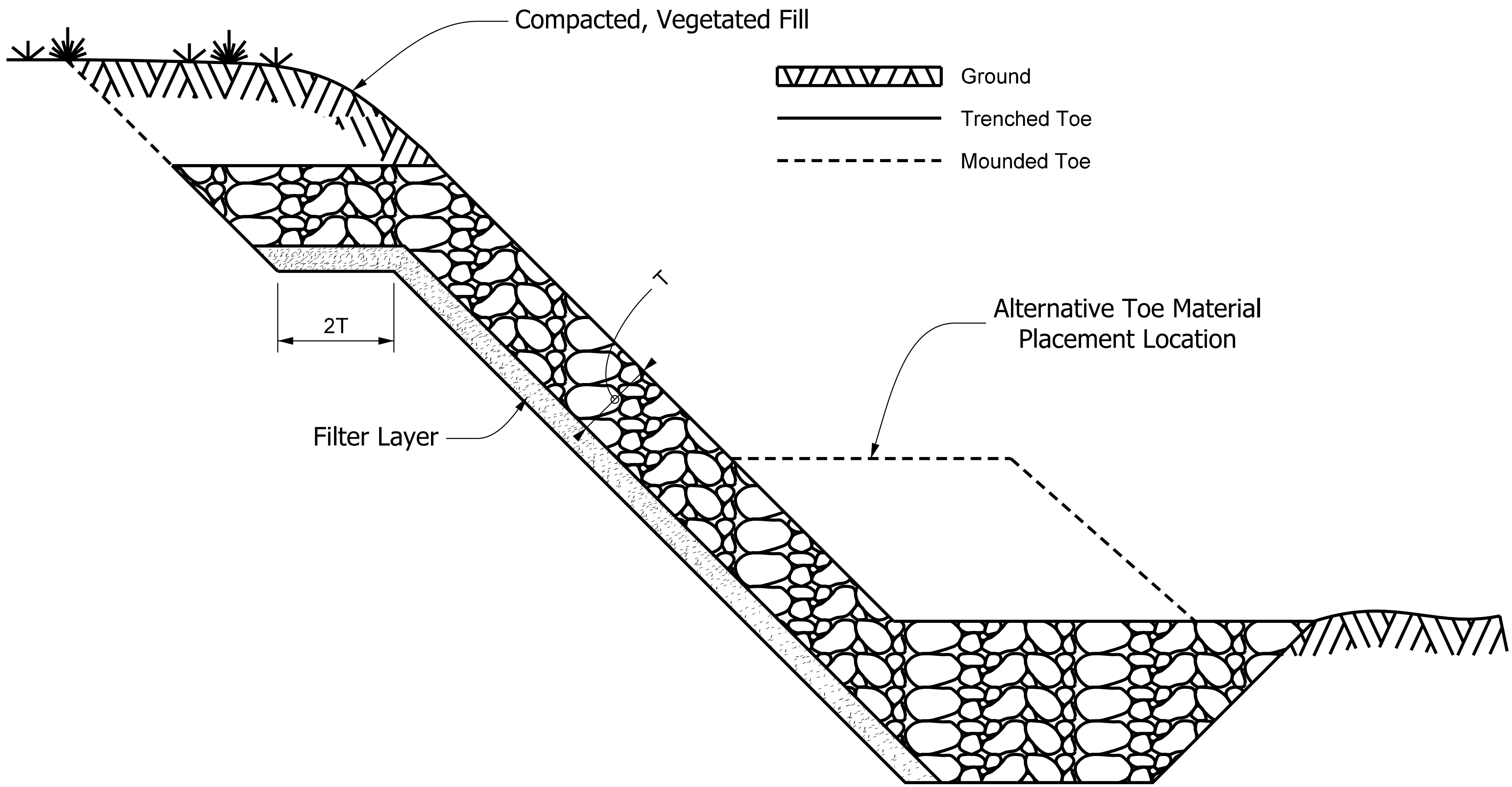
SECTION: A-A



SECTION: B-B

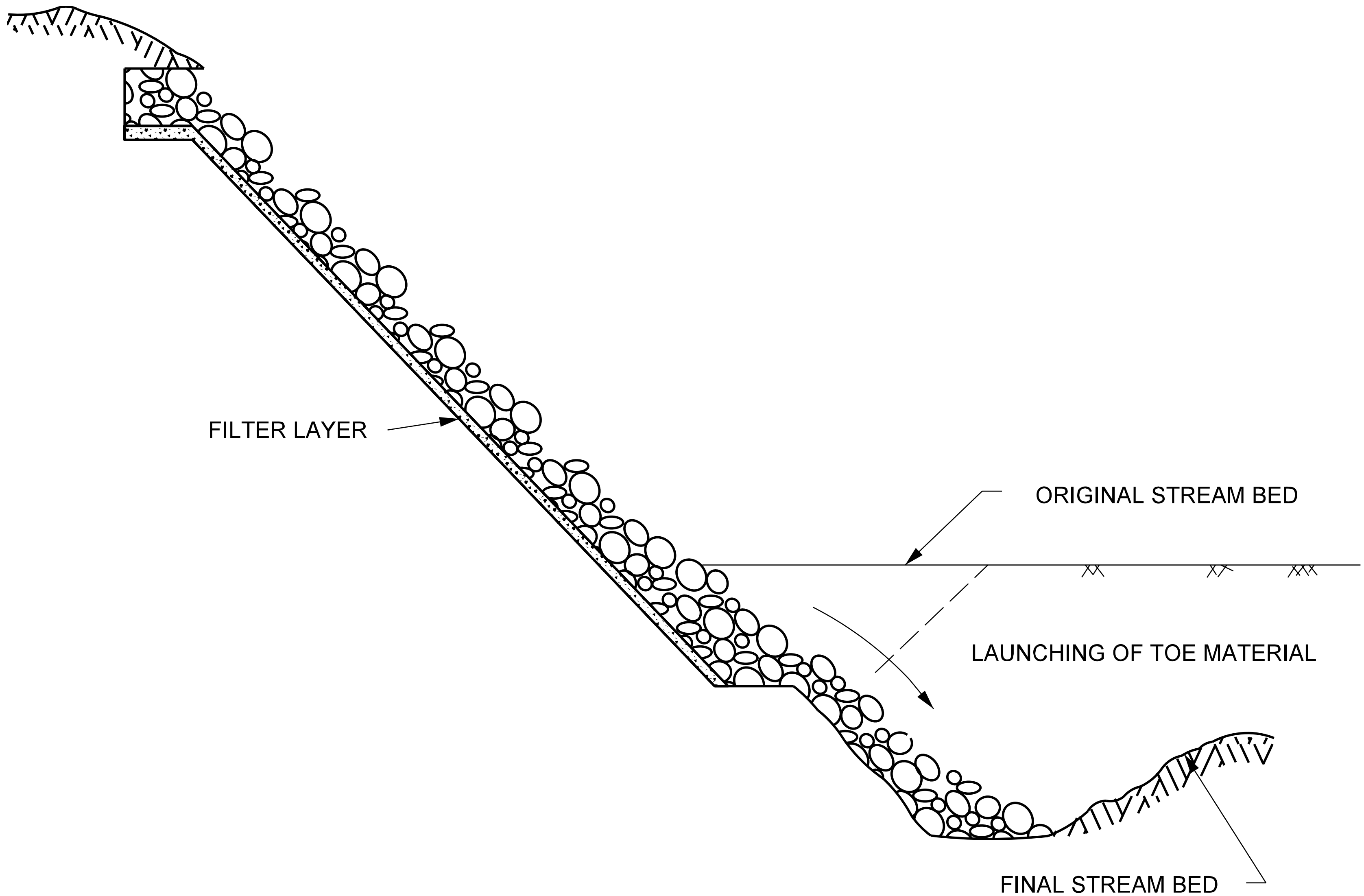
TYPICAL RIPRAP INSTALLATION: PLAN AND FLANK DETAILS

Figure 38-6M



TYPICAL RIPRAP INSTALLATION: SIDE VIEW
(Bank Protection Only)

Figure 38-6N



LAUNCHING OF RIPRAP TOE MATERIAL

Figure 38-6O

ROUTE: _________ DES. NO.: _______________ PROJECT NO.: ________________________________
DESCRIPTION: ____
Prepared By: ________________________ Date: ___________ Checked By: _________________________ Date: __________

Definition Sketch

Definition Sketch for Riprap Size Particle Erosion

Q_{TOTAL} __________
Q_{MC} ____________
Q_{LB} ____________
Q_{RB} ____________

Soil Characteristics:
D_{15} ____________
D_{50} ____________
D_{85} ____________

DEPTH OR W.S. (ft) (1)	A (ft^2) (2)	V_g (ft/s) (3)	d_a (ft) (4)	θ (5)	Φ (6)	K_1 (7)	D_{50} (in.) (8)	SF (9)	S_S (10)	C (11)	$C_{P/A}$ (12)	D_{50} (in.) (13)	NOTES

Design Sketch

RIPRAP CHARACTERISTICS:
Size: _______ Thickness: ________
D_{50} _______ $2D_{50}$ _________
Class ______ D_{100} _________
AASHTO Use _________
Gradation:

Size: (in.)	Percent Finer
_____	100
_____	50
_____	5-10

FABRIC CHARACTERISTICS:
Granular:

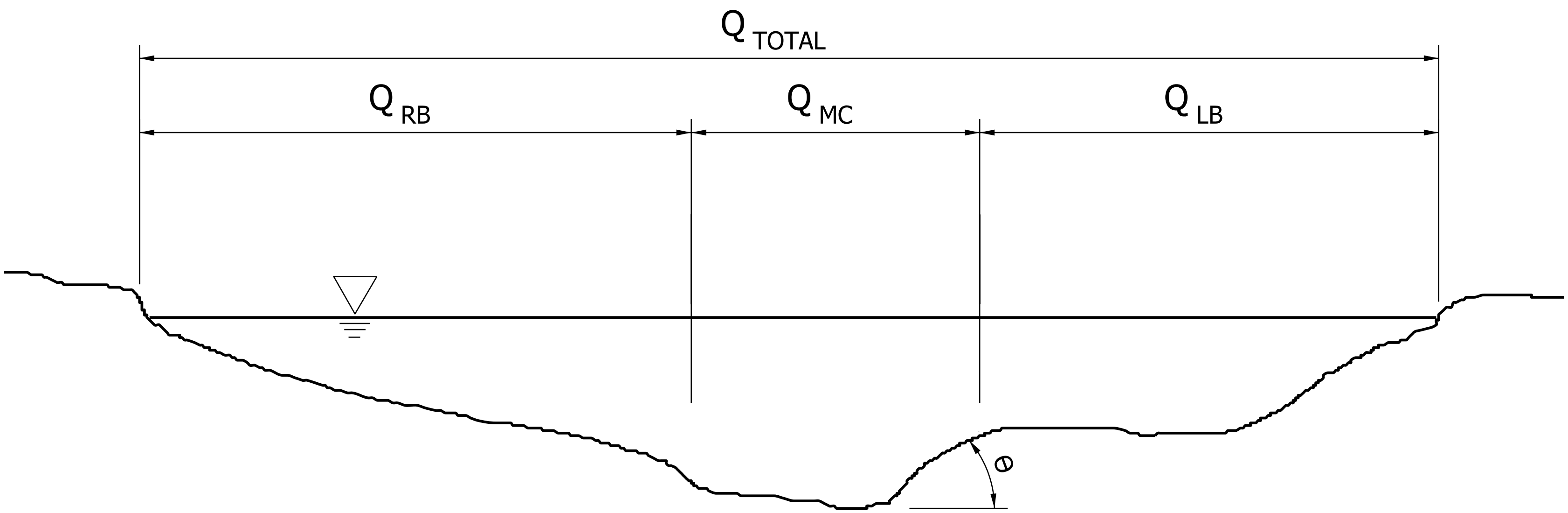
Size: (in.)	Percent Finer
_____	100
_____	50
_____	5-10

Fabric: AOS < _____ mils
Perm. > _____ mils

(1) Water surface elevation
(2) Main channel flow area
(3) Main channel average velocity
(4) Main channel average depth
(5) Bank angle
(6) Riprap angle of repose (Fig. 38-6C)
(7) Bank angle correction (Fig. 38-6B)
(8) Riprap size (Fig. 38-6A)
(9) Stability factor
(10) Riprap specific gravity
(11) Riprap size correction factor (Fig. 38-6E)
(12) Pier/abutment correction (3.38 if applicable)
(13) Correction D_{50} = (8) x (11) x (12)

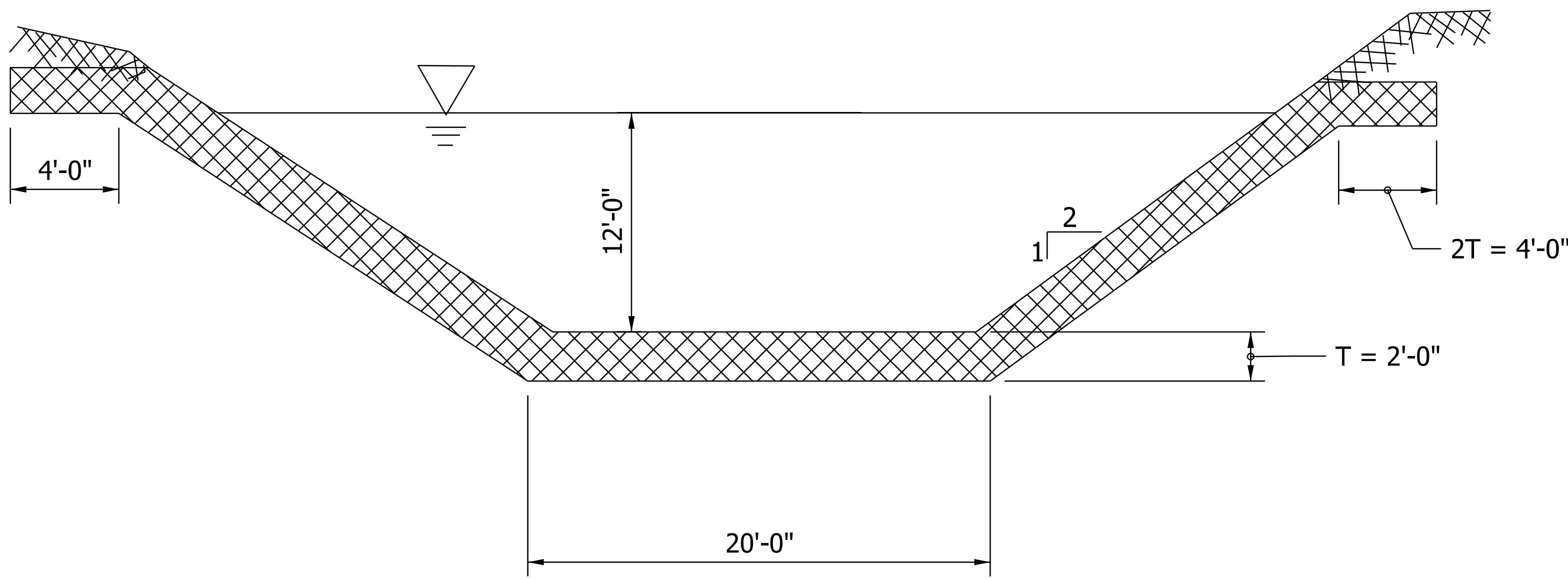
RIPRAP SIZE PARTICLE EROSION

Click When Done Clear Data



DEFINITION SKETCH FOR
RIPRAP SIZE PARTICLE EROSION

Figure 38-6P (Page 2 of 3)



DESIGN SKETCH FOR
RIPRAP SIZE PARTICLE EROSION

Figure 38-6P (Page 3 of 3)

ROUTE: _________ DES. NO.: _______________ PROJECT NO.: _______________________________ Sheet ____ o f ____
DESCRIPTION: ____
Prepared By: _______________________ Date: ___________ Checked By: ________________________ Date: __________

Definition Sketch

Definition Sketch for Data Sheet

WIND SPEED (mph)	FETCH (mi)	R_v (ft)	H_o (ft)	$\frac{R_v}{H_o}$	Θ	CORR. FACTOR	D_{50} (in.)

RIPRAP SIZE:
D_{50} _______
Class ______

REVETMENT THICKNESS:
$2D_{50}$ ________
D_{100} ________
Use ________

AASHTO GRADATION:

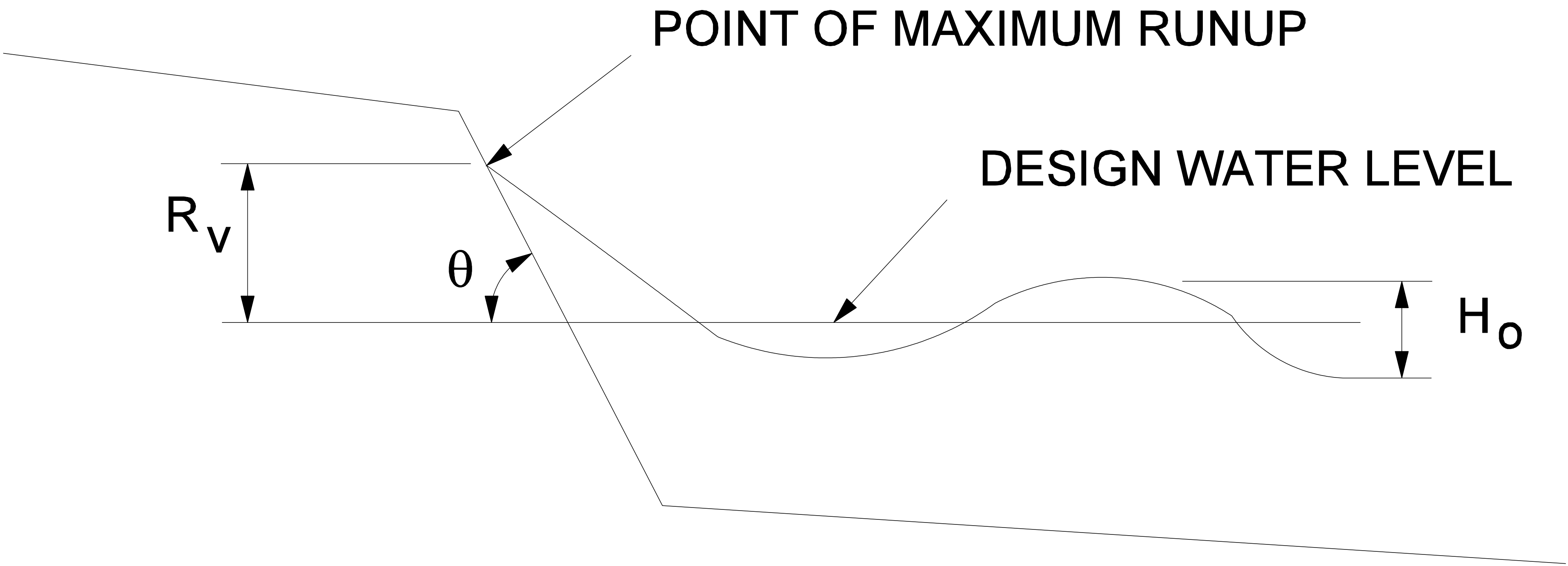
Size: (in.)	Percent Finer
_____	100
_____	50
_____	5-10

RIPRAP DATA WORKSHEET

Click When Done

Clear Data

Back to Data Sheet



DEFINITION SKETCH FOR DATA SHEET

ROUTE: ________ DES. NO.: __________ PROJECT NO.: ____________________
DESCRIPTION: Example 1
Prepared By: SAB Date: 2/4 Checked By: CJH Date: 3/4

Definition Sketch

See Figure 38-6R(1) for definition sketch.

Q_{TOTAL} 5000
Q_{MC} 5000
Q_{LB} ________
Q_{RB} ________

Soil Characteristics:
D_{15} 0.0055 ft
D_{50} 0.04 ft
D_{85} 0.105 ft

DEPTH OR W.S. (ft) (1)	A (ft^2) (2)	V_g (ft/s) (3)	d_a (ft) (4)	θ (5)	Φ (6)	K_1 (7)	D_{50} (in.) (8)	SF (9)	S_S (10)	C (11)	$C_{P/A}$ (12)	D_{50} (in.) (13)	NOTES
11.8	514.5	9.7	11.8	2:1	41°	0.73	5.16	1.2	2.65	1	N/A	0.43	BANK
							3.36	1.2	2.65	1	N/A	0.28	BED

Design Sketch

RIPRAP CHARACTERISTICS:
Size: ______ Thickness: ________
D_{50} 0.95 $2D_{50}$ 1.9 ft
Class Facing D_{100} 1.3 ft
AASHTO Use 2.0 ft
Gradation:

Size: (ft)	Percent Finer
1.30	100
0.95	50
0.40	5-10

FABRIC CHARACTERISTICS:
Granular:

Size: (ft)	Percent Finer
0.20	85
0.17	50
0.10	15

Fabric: AOS < _____ mils
Perm. > _____ mils

(1) Water surface elevation
(2) Main channel flow area
(3) Main channel average velocity
(4) Main channel average depth
(5) Bank angle
(6) Riprap angle of repose (Fig. 38-6C)
(7) Bank angle correction (Fig. 38-6B)
(8) Riprap size (Fig. 38-6A)
(9) Stability factor
(10) Riprap specific gravity
(11) Riprap size correction factor (Fig. 38-6E)
(12) Pier/abutment correction (3.38 if applicable)
(13) Correction D_{50} = (8) x (11) x (12)

RIPRAP SIZE FORM (EXAMPLE 1)
Figure 38-6R

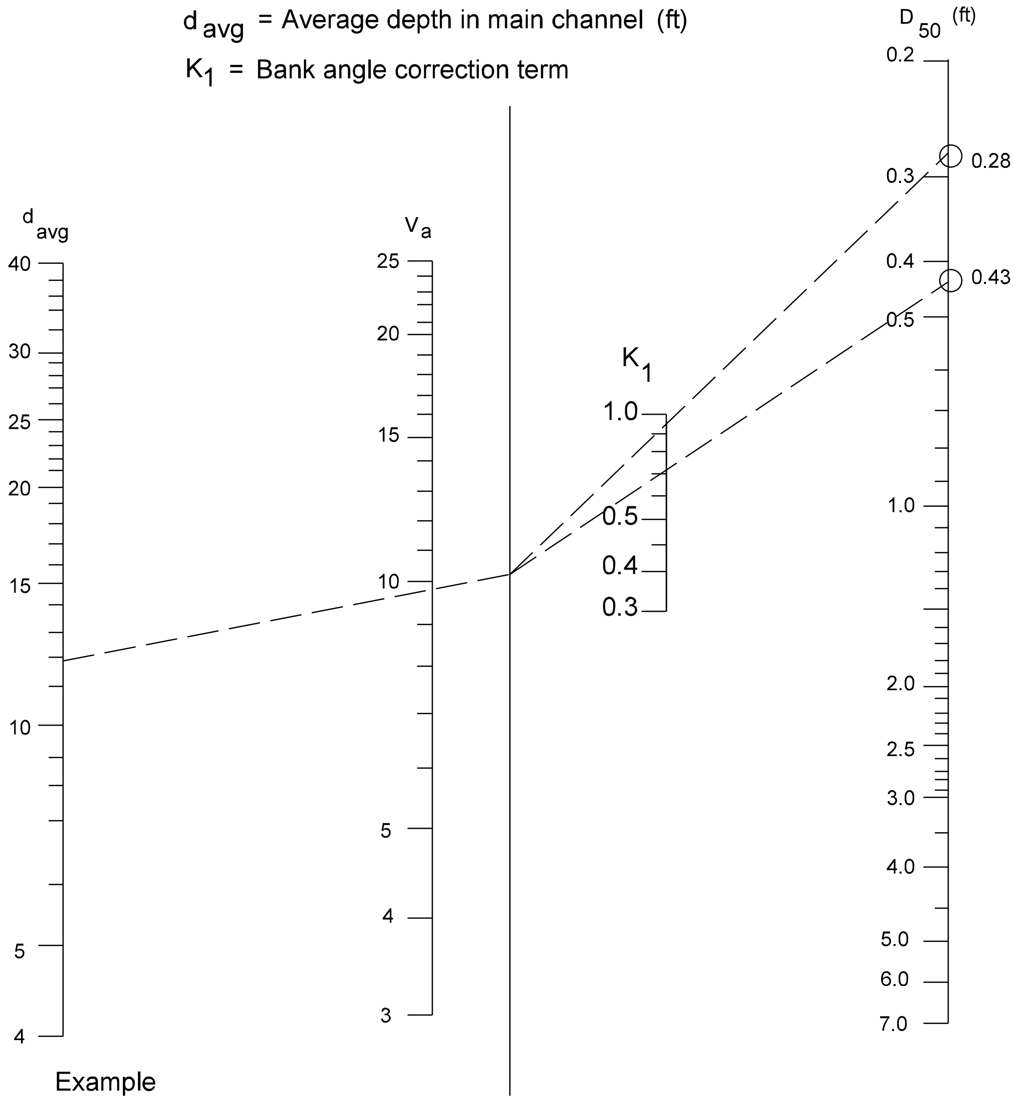
$$D_{50} = 0.001 \ V_a^3/(d_{avg}^{1/2} K_1^{3/2})$$

D_{50} = Median riprap size (ft)

V_a = Average velocity in main channel (ft/s)

d_{avg} = Average depth in main channel (ft)

K_1 = Bank angle correction term



Example

Given:

V_a = 9.7 ft/s

d_{avg} = 11.8 ft

K_1 = 0.73

Find:

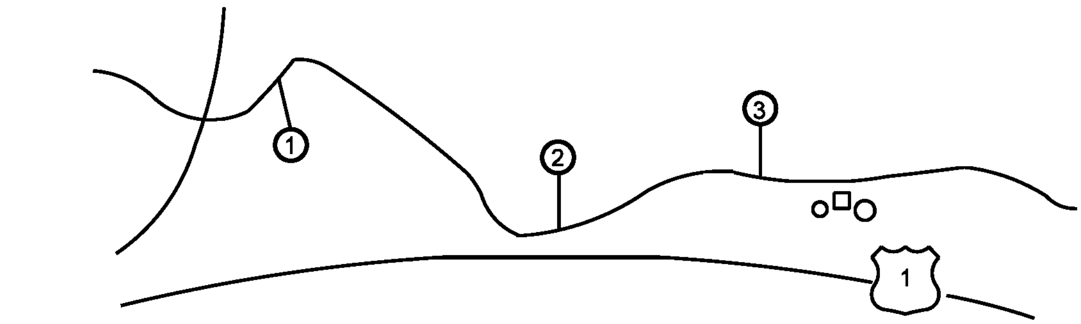
D_{50}

Solution:

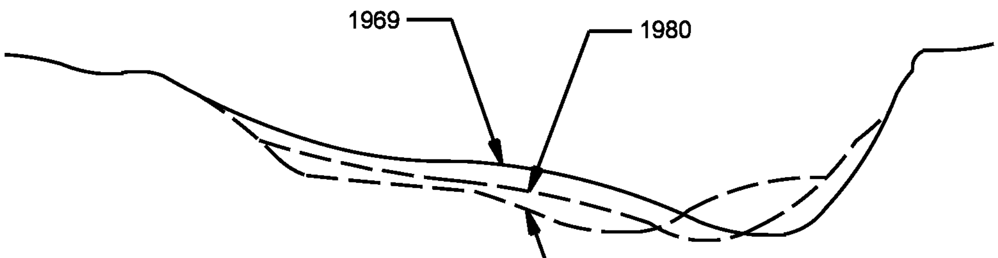
D_{50} = 0.43 ft

RIPRAP SIZE RELATIONSHIP
(Example 1, Step 7)

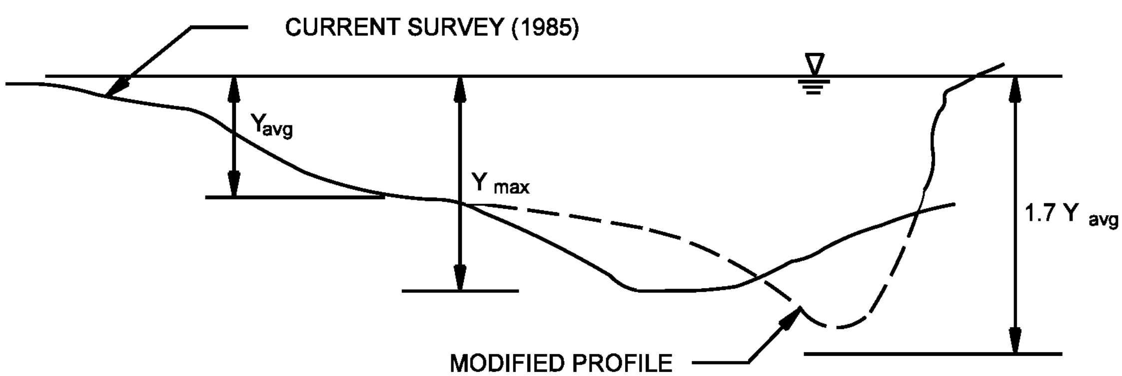
Figure 38-6S



A) SITE LOCATION



B) BOTTOM PROFILES AT LOCATION 1



C) BOTTOM PROFILE AND MODIFIED PROFILE AT LOCATION 2

CHANNEL GEOMETRY DEVELOPMENT (Example 2)

Figure 38-6T

PROJECT: Example 2 DESCRIPTION:	Prepared by/Date: / Checked by/Date: / Sheet of

Definition Sketch:

Q_{TOTAL} ______
Q_{MC} ______
Q_{LB} ______
Q_{RB} ______

Soil Characteristics:
D_{15} ______
D_{50} ______
D_{85} ______

DEPTH OR W.S. (ft) (1)	A (ft^2) (2)	V_g (ft/s) (3)	d_a (ft) (4)	θ (5)	Φ (6)	K_1 (7)	D_{50} (in.) (8)	SF (9)	S_S (10)	C (11)	$C_{P/A}$ (12)	D_{50} (in.) (13)	NOTES (14)
15	255	12.8	12	2:1	41^o	0.73	11	1.6	2.6	1.6	1	1.44	Sharp Bend

Design Sketch:

RIPRAP CHARACTERISTICS:
Size: 22 Thickness: 43
D_{50} ______ $2D_{50}$ 27
Class D_{100} 43
AASHTO Use __
Gradation:

Size: (in.)	Percent Finer
27	100
22	50
11.5	5-10

FABRIC CHARACTERISTICS:

Granular:

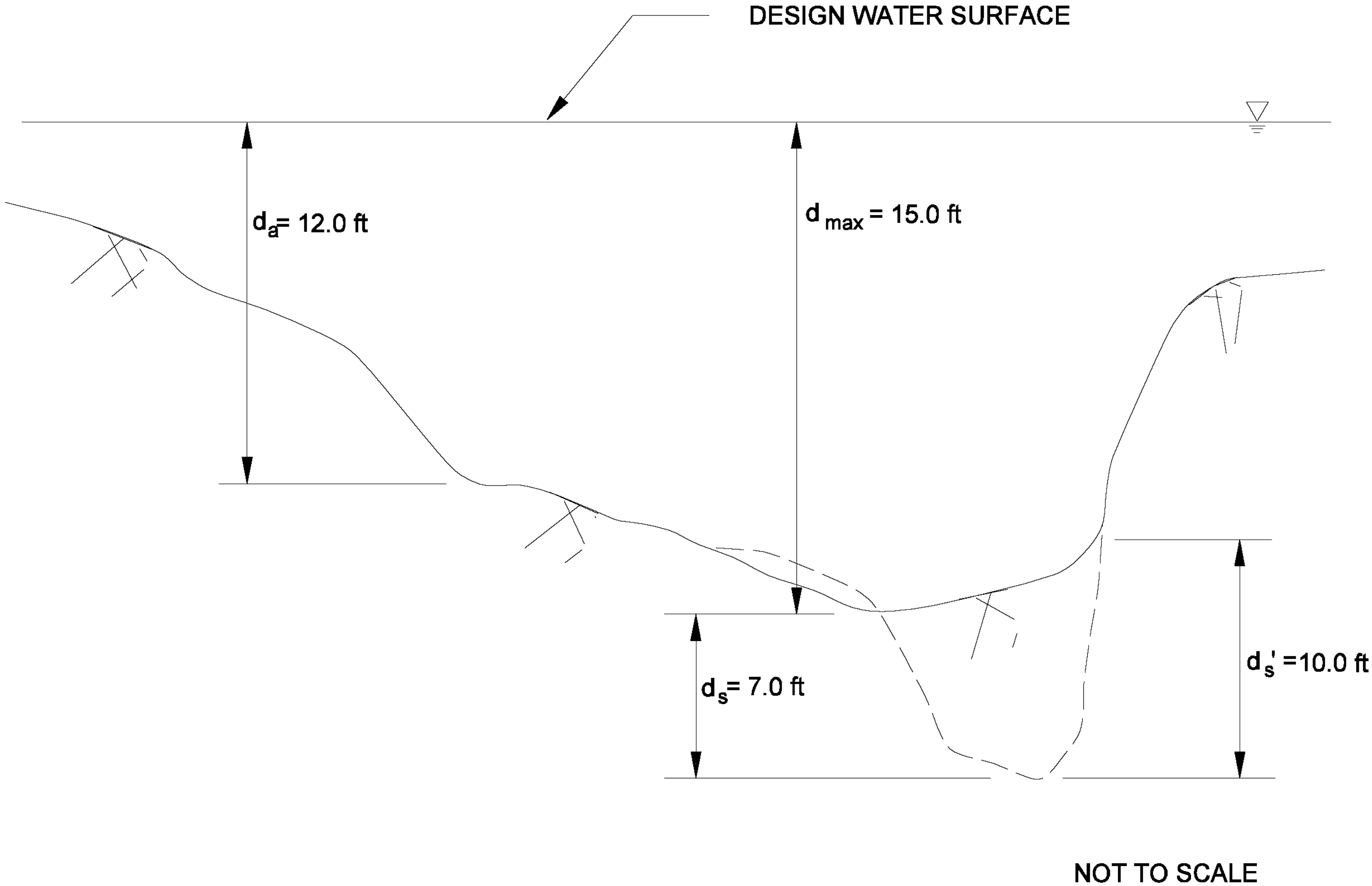
Size (in.)	Percent Finer
	85
	50
	15

Fabric:
AOS < ______
Perm. > ______

Average Opening Size

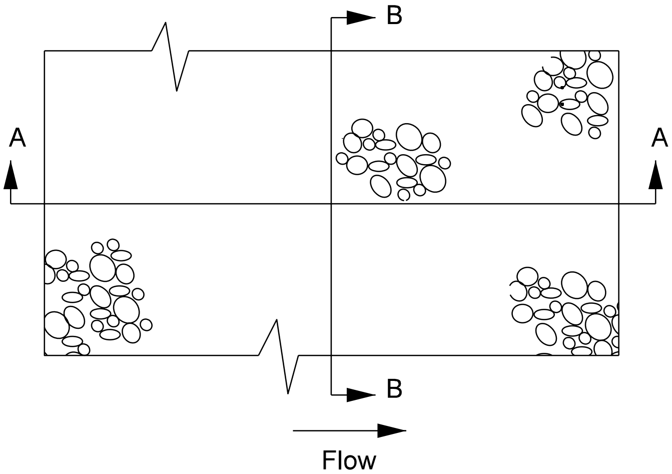
(1) Water surface elevation
(2) Main channel flow area
(3) Main channel average velocity
(4) Main channel average depth
(5) Bank angle
(6) Riprap angle of repose (Figure 38-6C)
(7) Bank angle correction (Figure 38-6B)
(8) Riprap size (Figure 38-6A)
(9) Stability factor
(10) Riprap specific gravity
(11) Riprap size correction factor (Figure 38-6E)
(12) Pier/abutment correction (3.38 if applicable)
(13) Correction D_{50} = (8) x (11) x (12)
(14) Notes or comments

RIPRAP SIZE FORM (Example 2)
Figure 38-6U

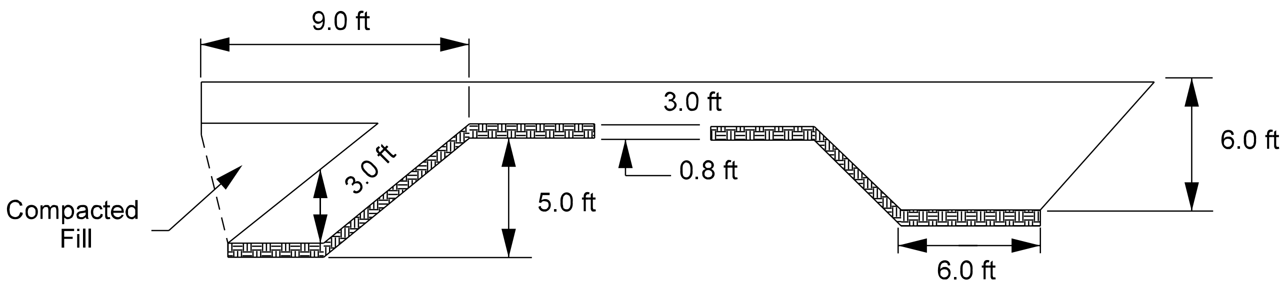


CHANNEL CROSS SECTION FOR EXAMPLE 2
ILLUSTRATING FLOW AND POTENTIAL SCOUR DEPTHS

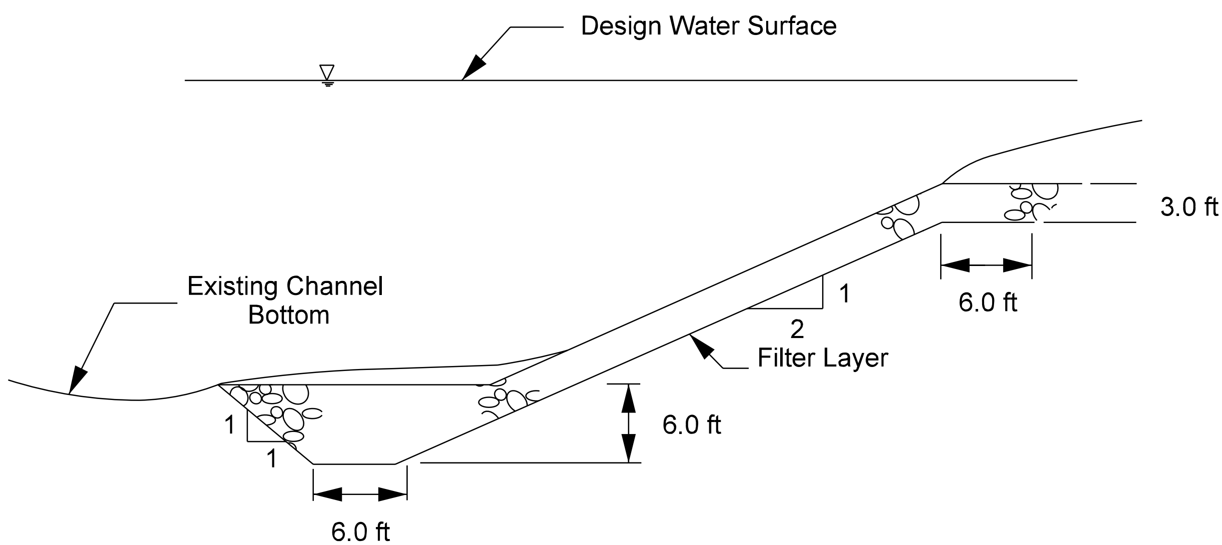
Figure 38-6V



(a) Plan



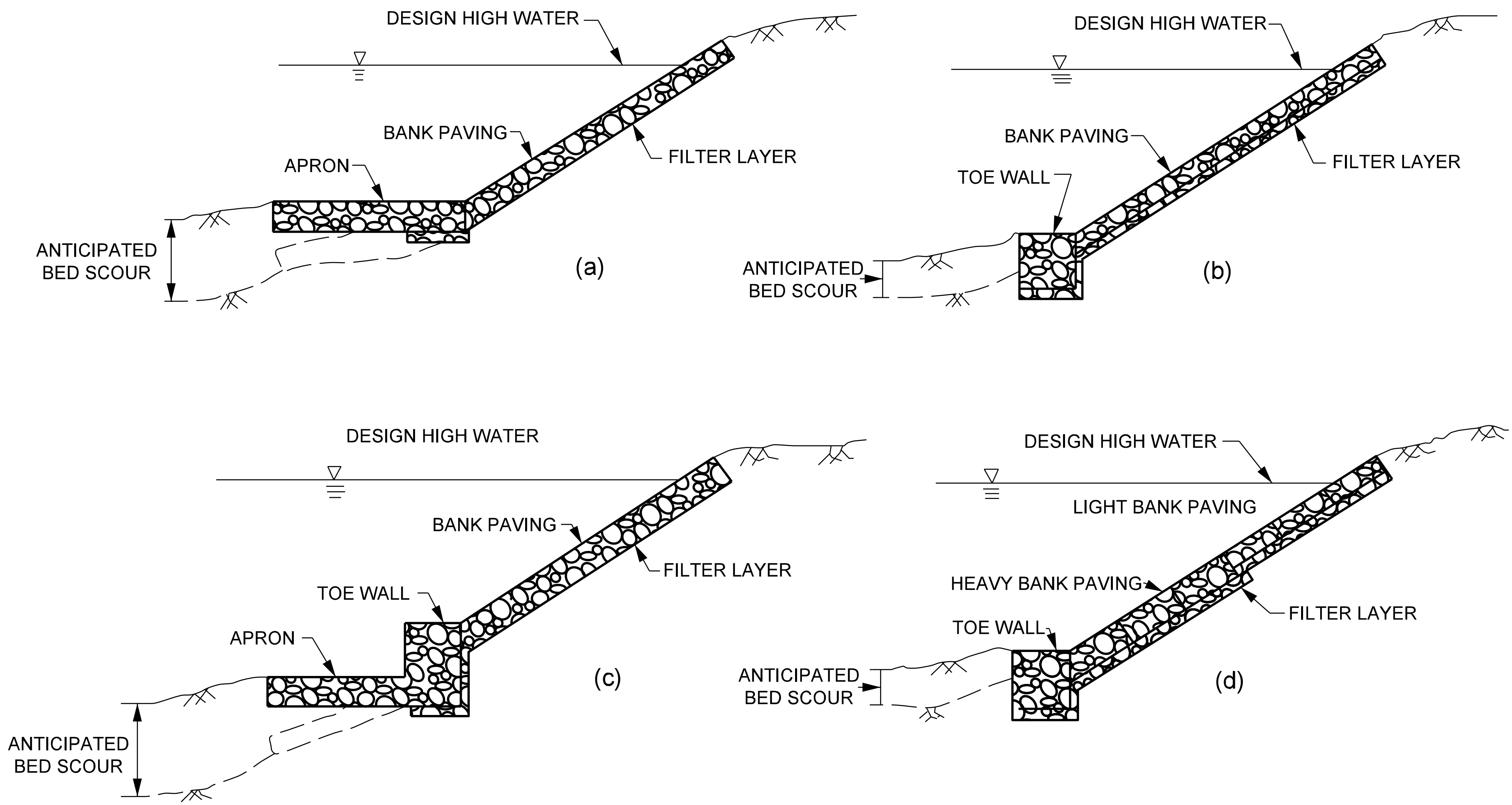
(b) Section A-A



(c) Section B-B

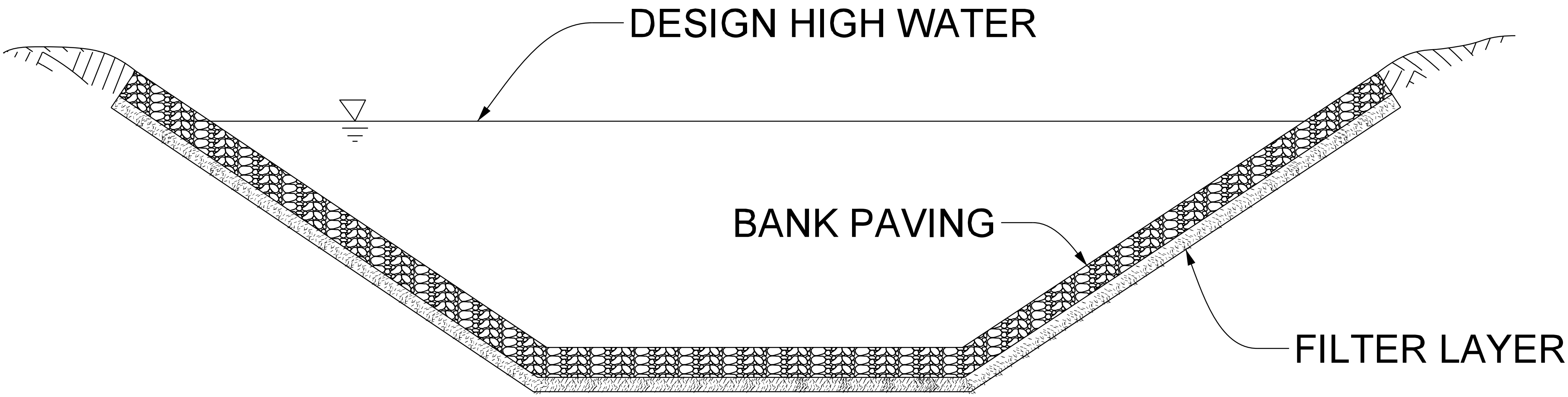
TOE AND FLANK DETAILS
(Example 2)

Figure 38-6W



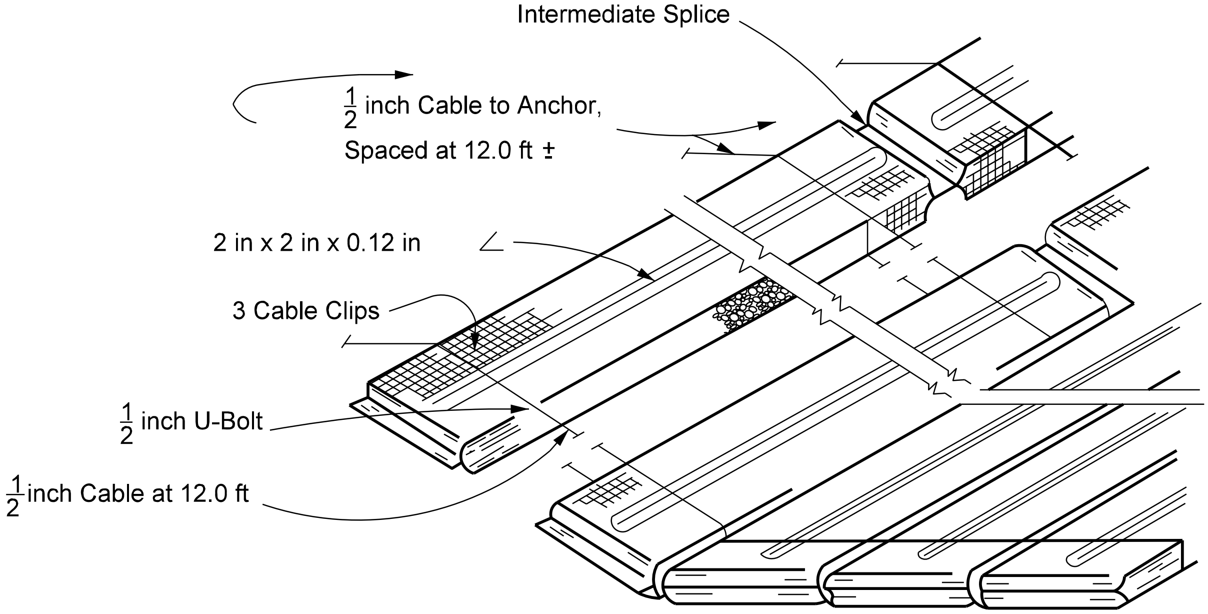
ROCK AND WIRE MATTRESS CONFIGURATION

Figure 38-6X

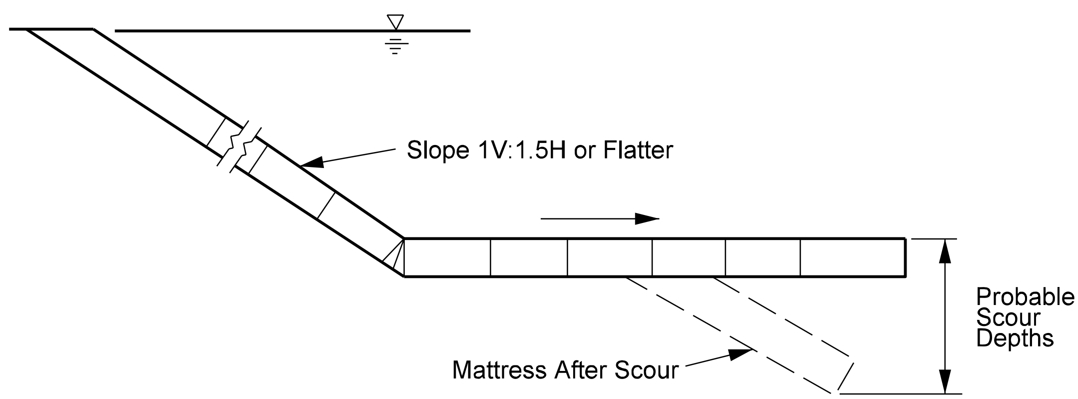


ROCK AND WIRE MATTRESS INSTALLATION COVERING
THE ENTIRE CHANNEL PERIMETER

Figure 38-6Y



Mattress Layout



Section

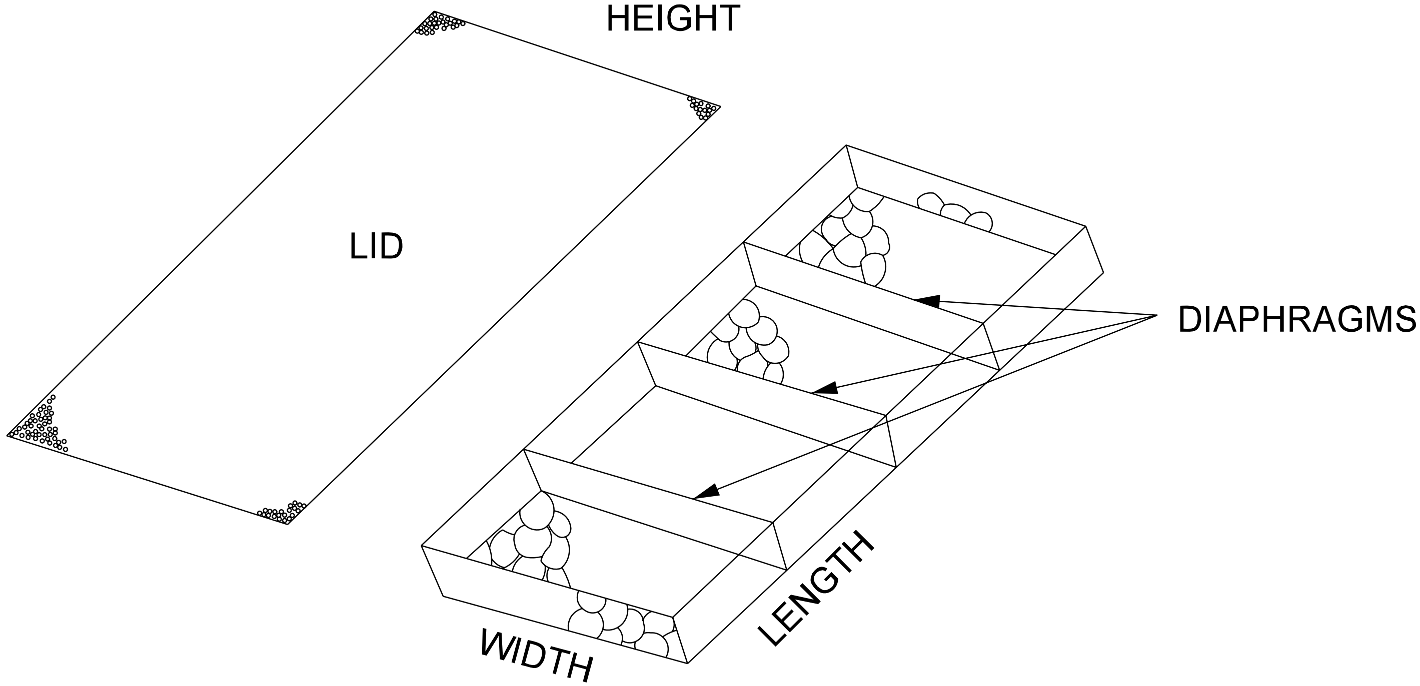
TYPICAL DETAIL OF ROCK AND WIRE MATTRESS
(Constructed from Available Wire Fencing Materials)

Figure 38-6Z

Thickness (ft)	Width (ft)	Length (ft)	Wire-Mesh Opening Size (in. x in.)
0.75	6	9	3 x 3
0.75	6	12	3 x 3
1.0	3	6	3 x 3
1.0	3	9	3 x 3
1.5	3	12	3 x 3
1.5	3	6	3 x 3
1.5	3	9	3 x 3
1.5	3	12	3 x 3
3.0	3	6	3 x 3
3.0	3	9	3 x 3
3.0	3	12	3 x 3

STANDARD GABION SIZES

Figure 38-6AA



MATTRESS CONFIGURATION

Figure 38-6BB

Bank Soil Type	Maximum Velocity (ft/s)	Bank Slope (H:V)	Minimum Required Mattress Thickness (in.)
Clay, Heavy Cohesive Soils	10	Flatter than 1:3	9
	13 – 16	Steeper than 1:2	12
	Any	Steeper than 1:2	≥ 18
Silt, fine sand	10	Flatter than 1:2	12
Shingle with Gravel	16	Flatter than 1:3	9
	20	Flatter than 1:2	12
	Any	Steeper than 1:2	≥ 18

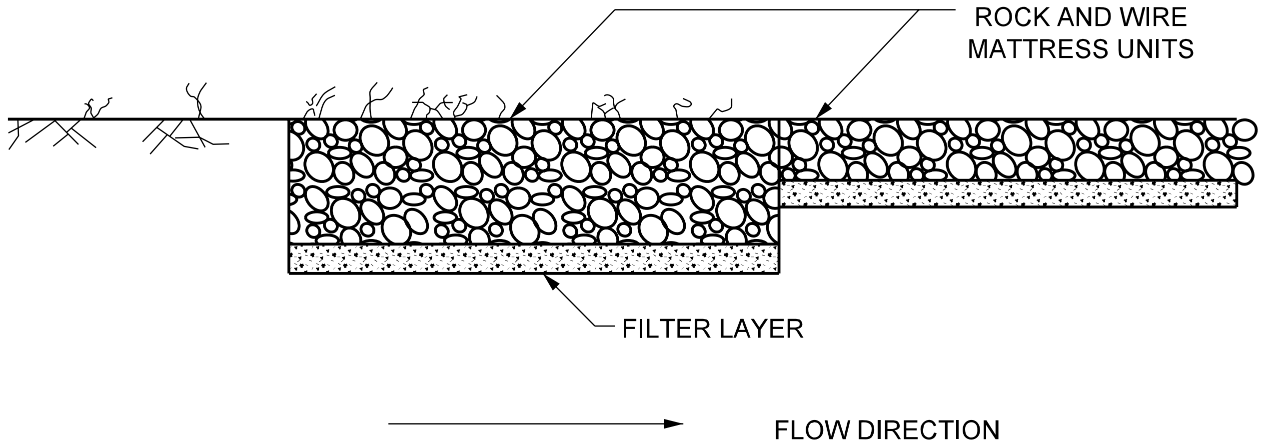
CRITERIA FOR GABION THICKNESS

Figure 38-6CC

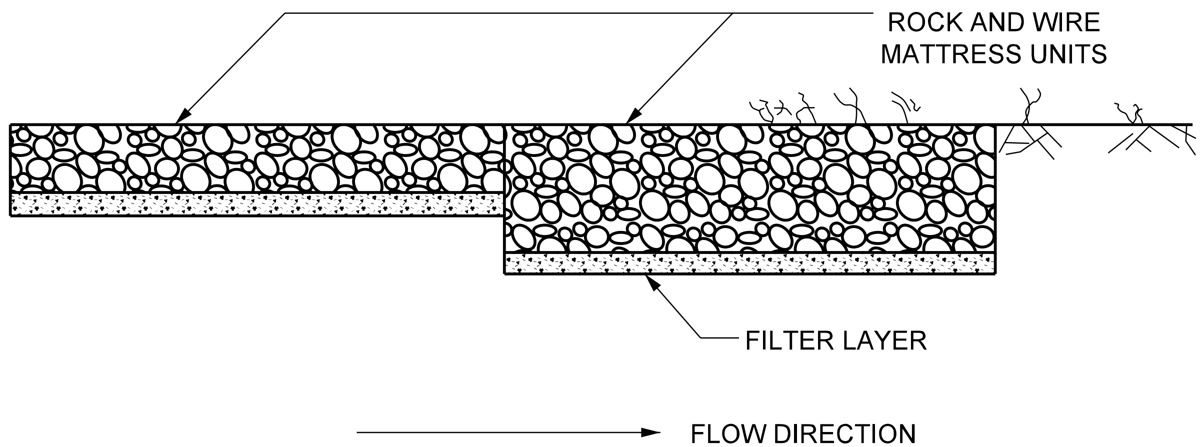
Nominal Diameter of Wire (in.)	Minimum Coating Weight, Class 3 or A Coating (oz/ft^2)
0.086	0.7
0.104	0.8
0.128	0.9

MINIMUM COATING WEIGHT

Figure 38-6DD



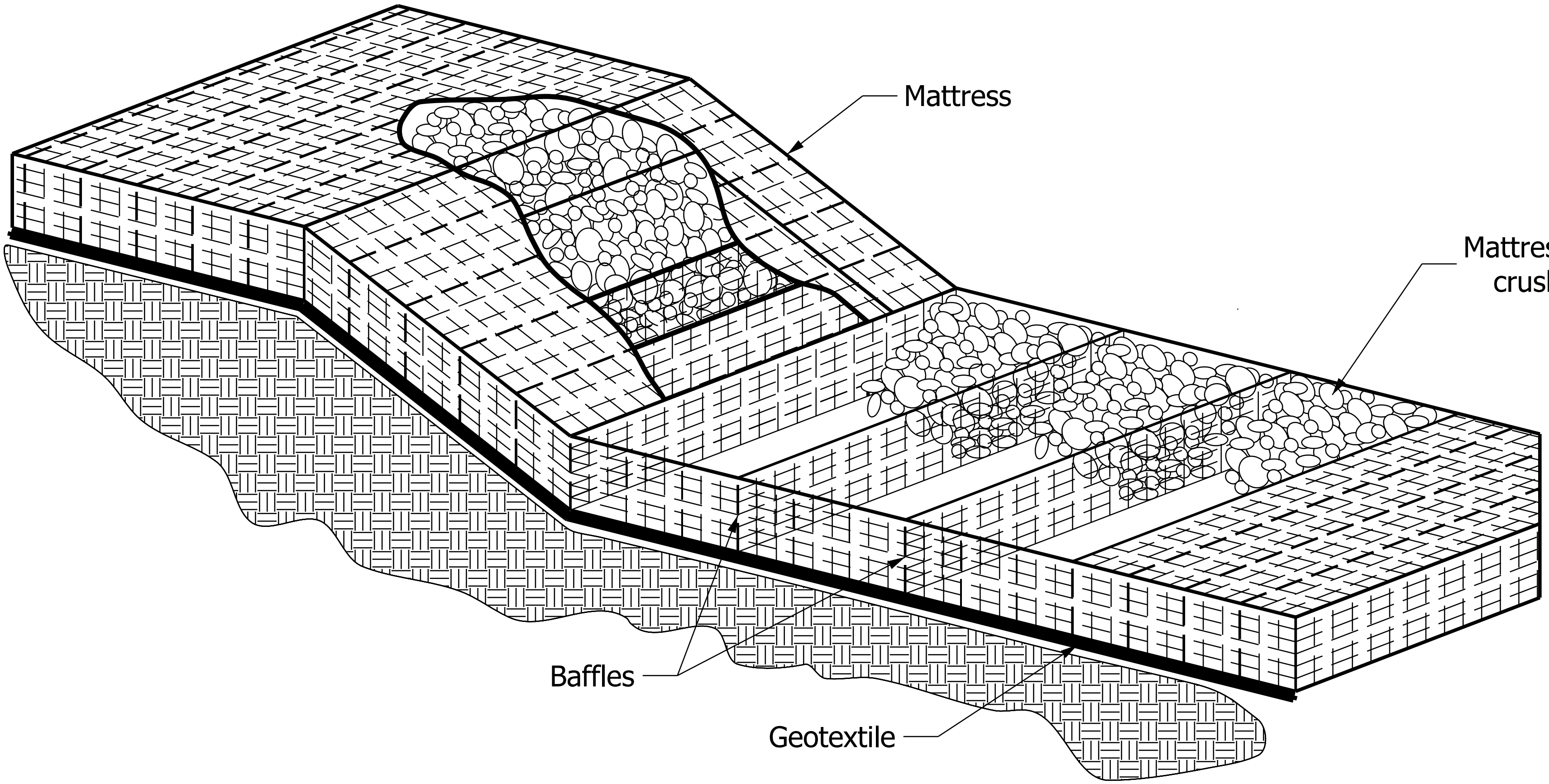
(a)



(b)

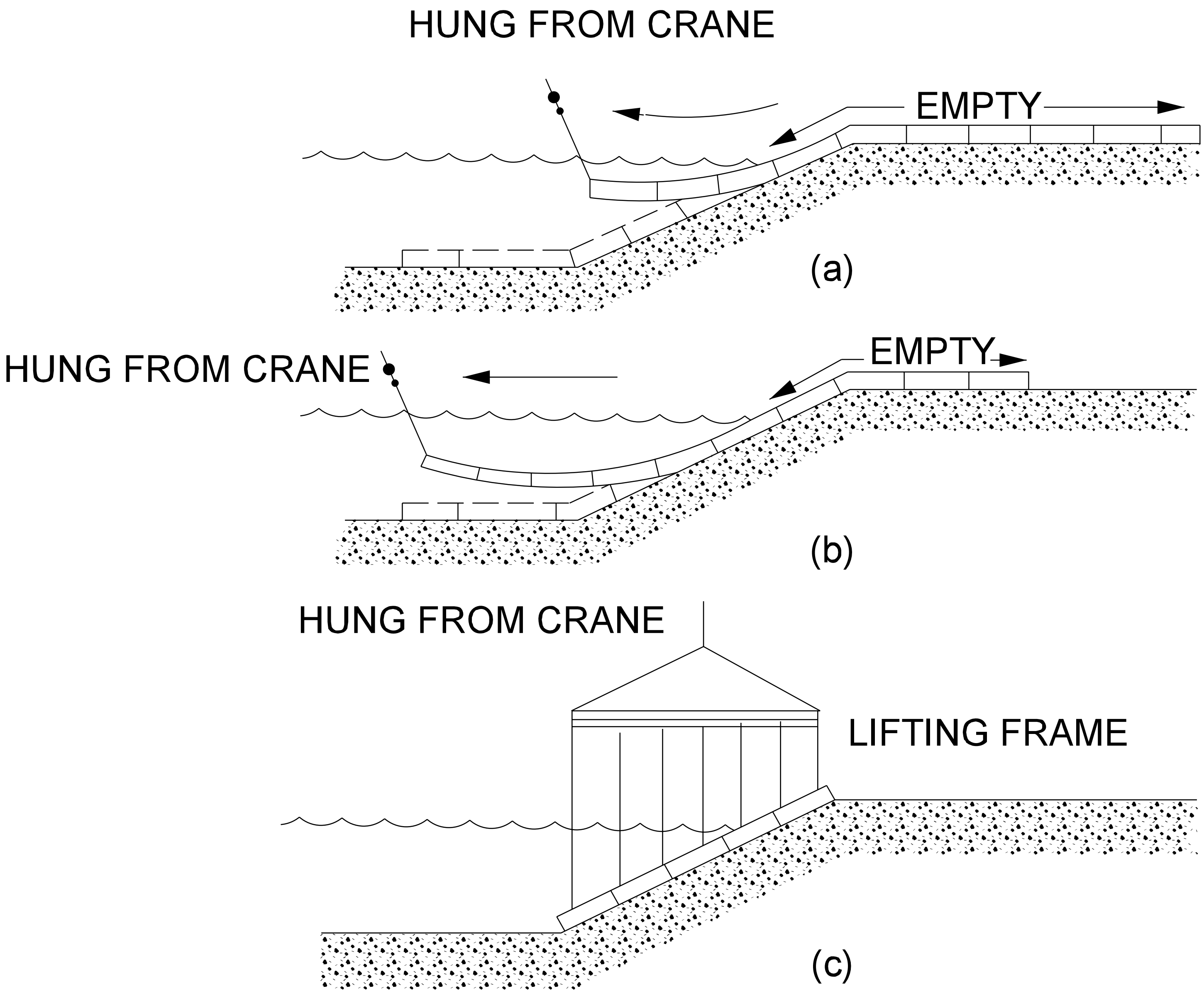
FLANK TREATMENT FOR ROCK AND WIRE MATTRESS DESIGNS
((a) Upstream Face; (b) Downstream Face)

Figure 38-6EE



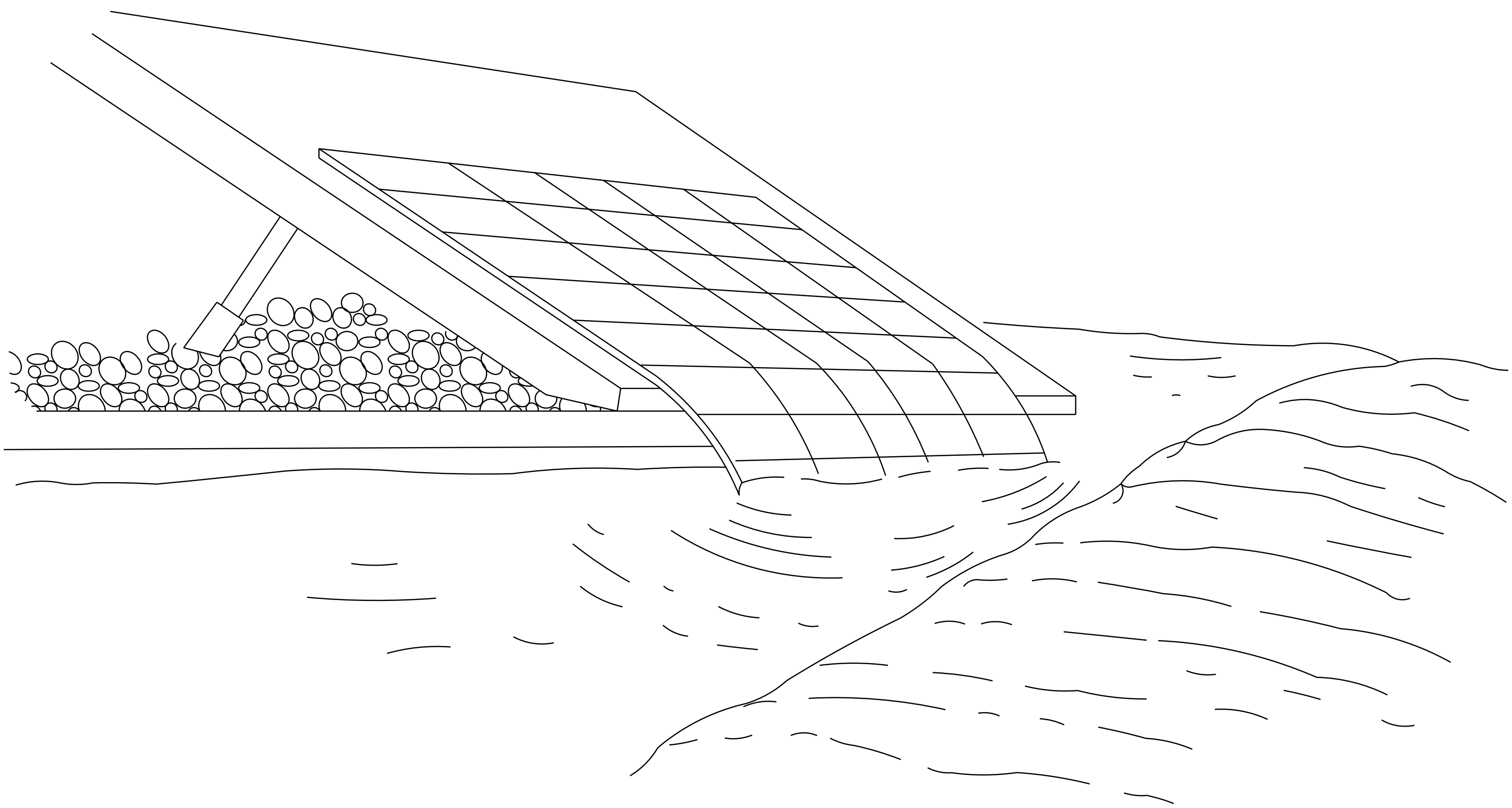
ROCK AND WIRE REVETMENT MATTRESS INSTALLATION

Figure 38-6FF



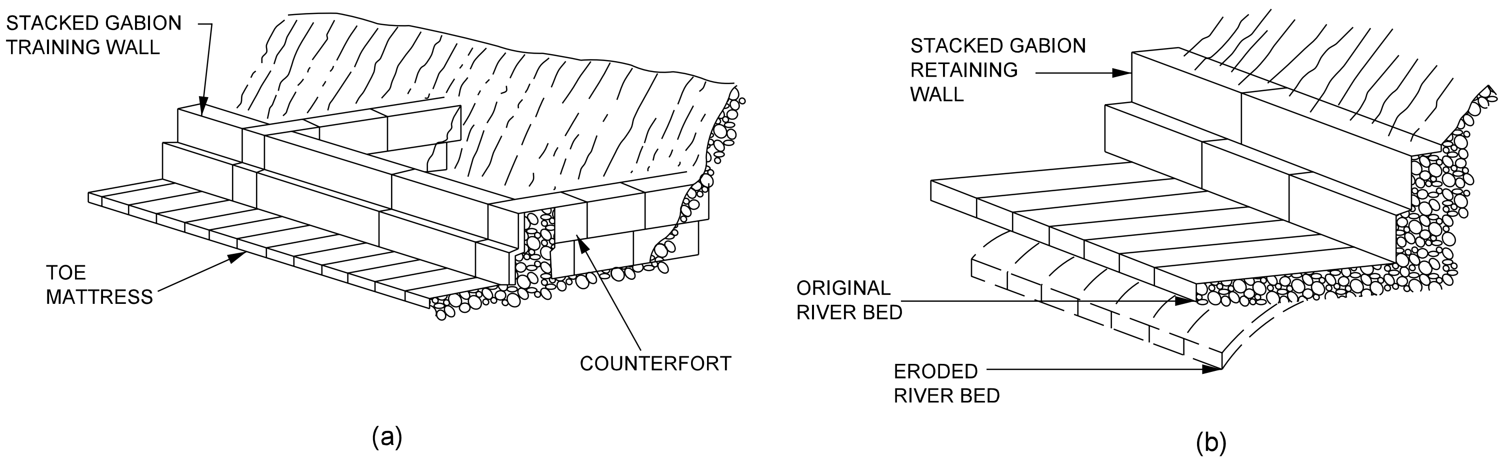
MATTRESS PLACEMENT UNDERWATER BY CRANE

Figure 38-6GG



PONTOON PLACEMENT OF WIRE MATTRESS

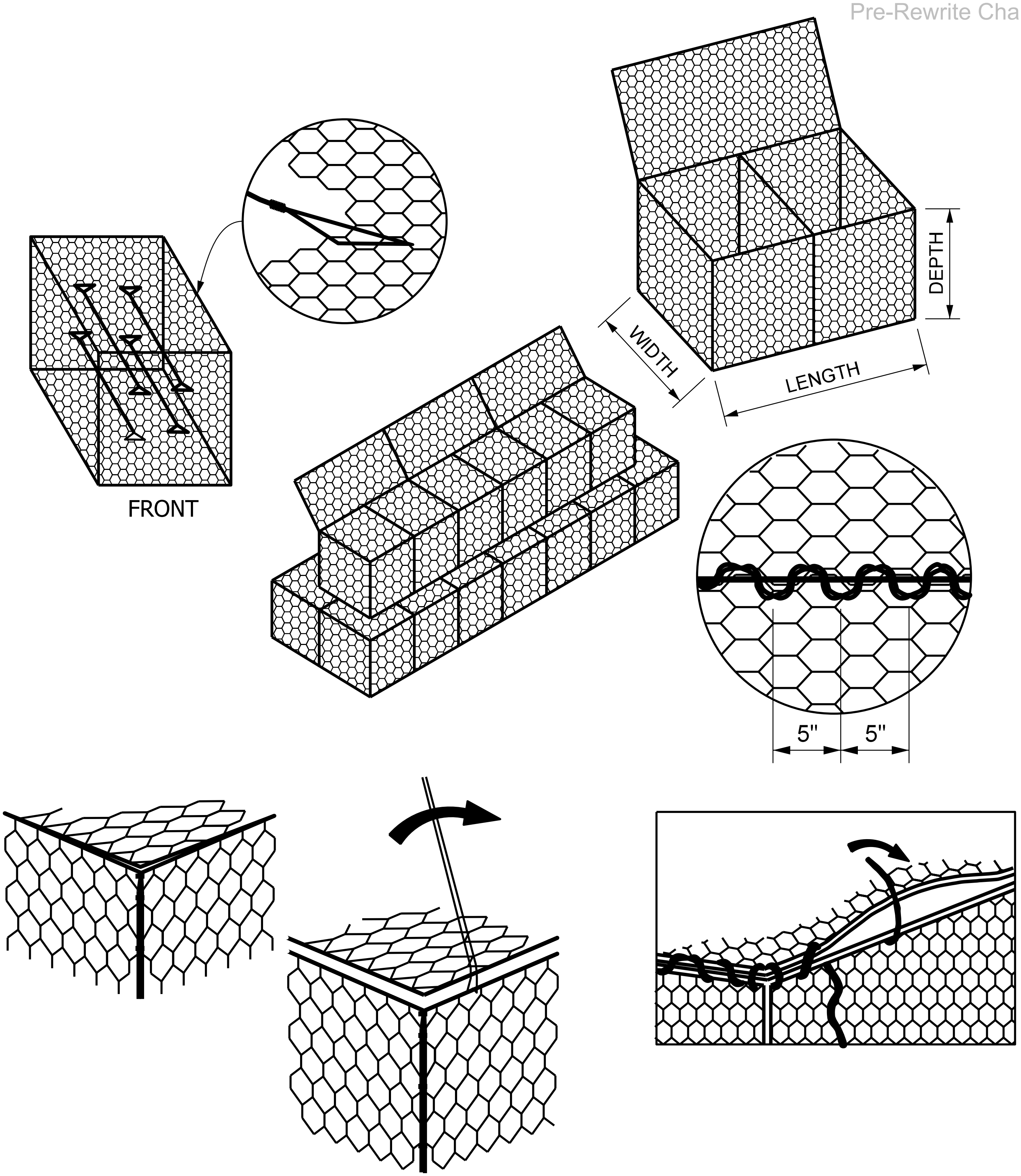
Figure 38-6HH



(a) training wall with counterforts; (b) stepped back low retaining wall with apron.

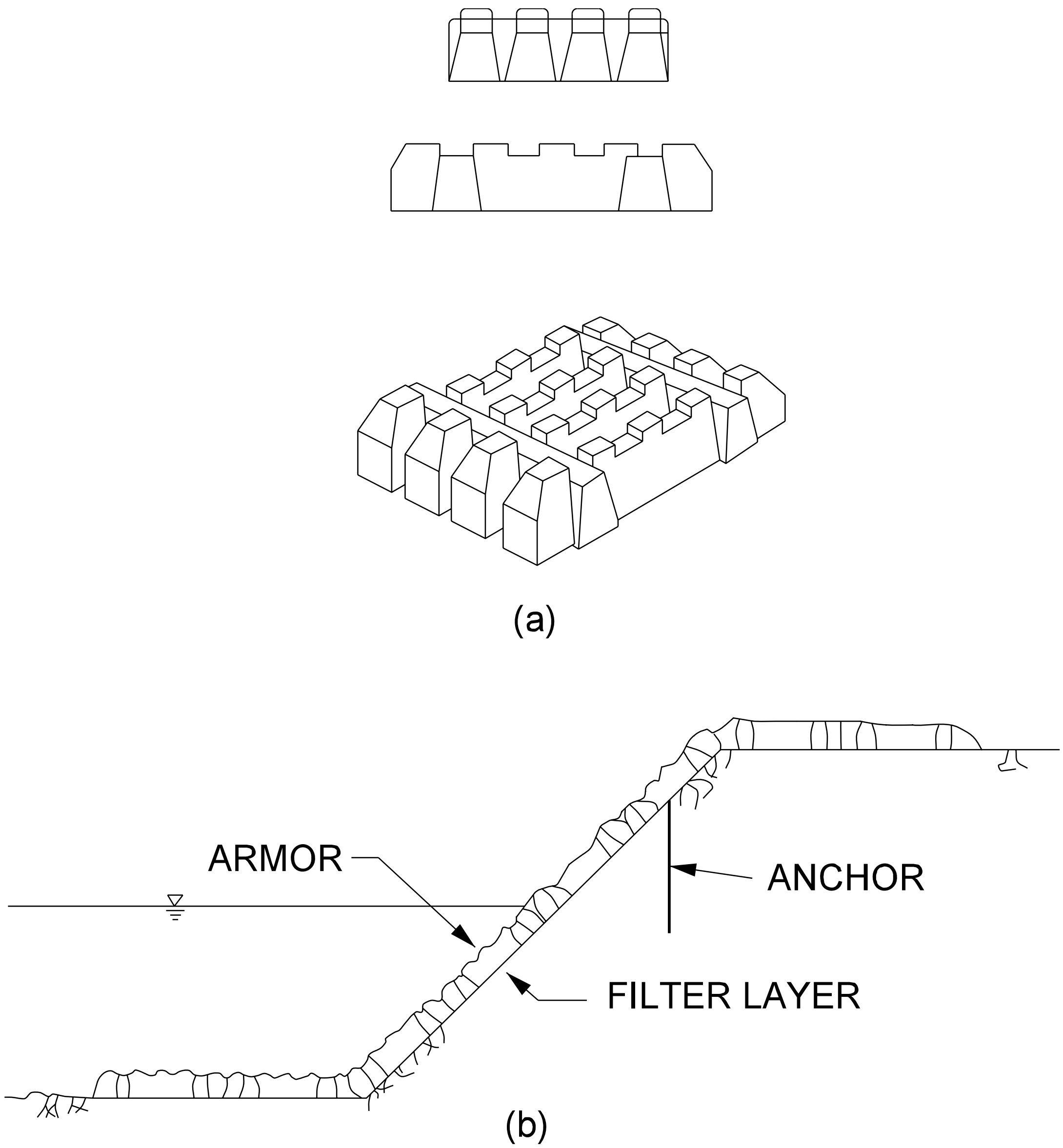
TYPICAL STACKED BLOCK GABION REVETMENT DETAILS

Figure 38-6II



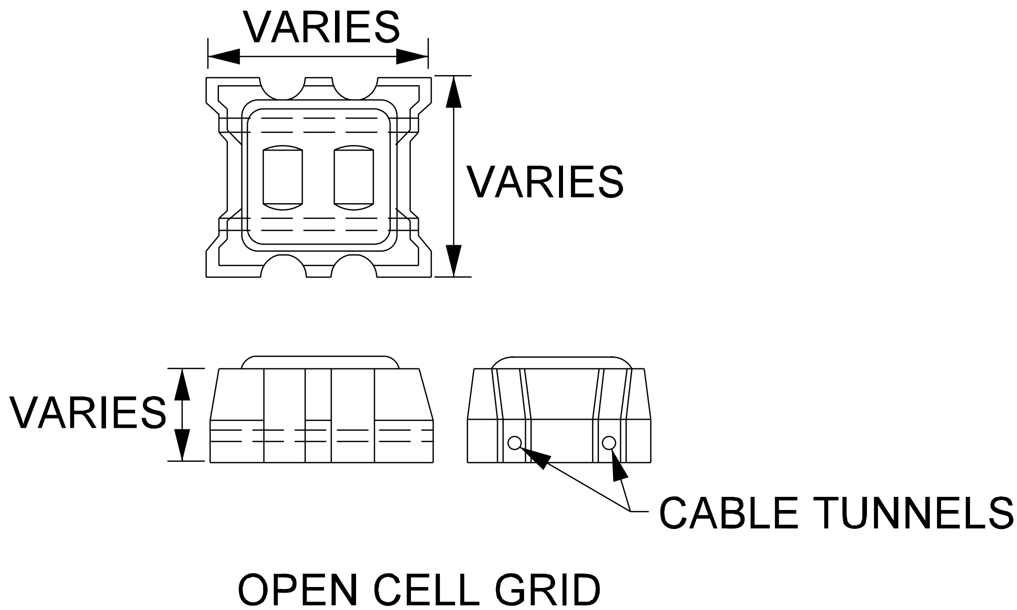
GABION BASKET FABRICATION

Figure 38-6JJ

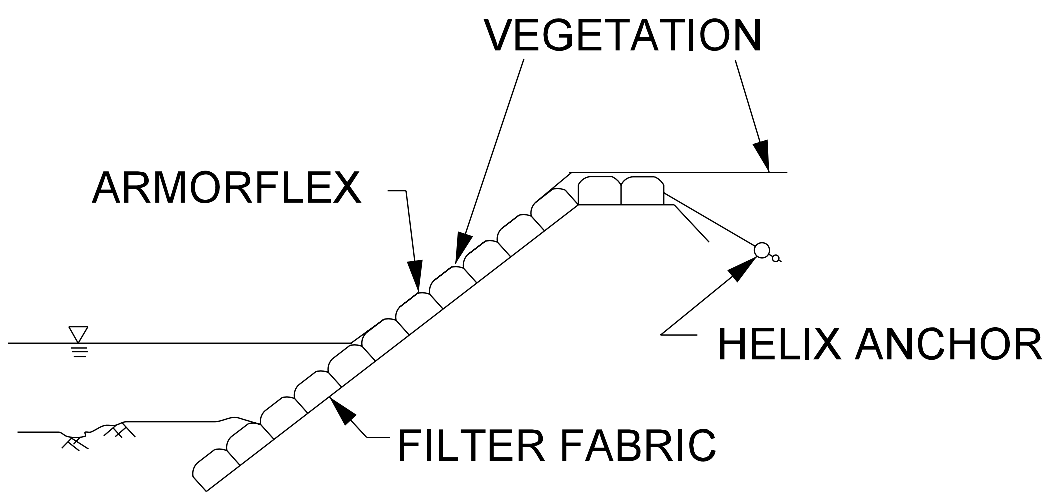


MONOSLAB REVETMENT
((a) block detail and (b) revetment detail)

Figure 38-6KK



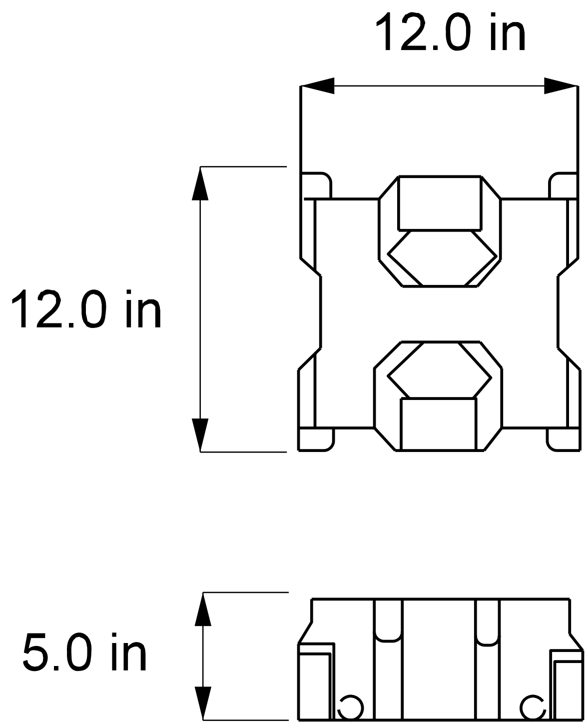
(a)



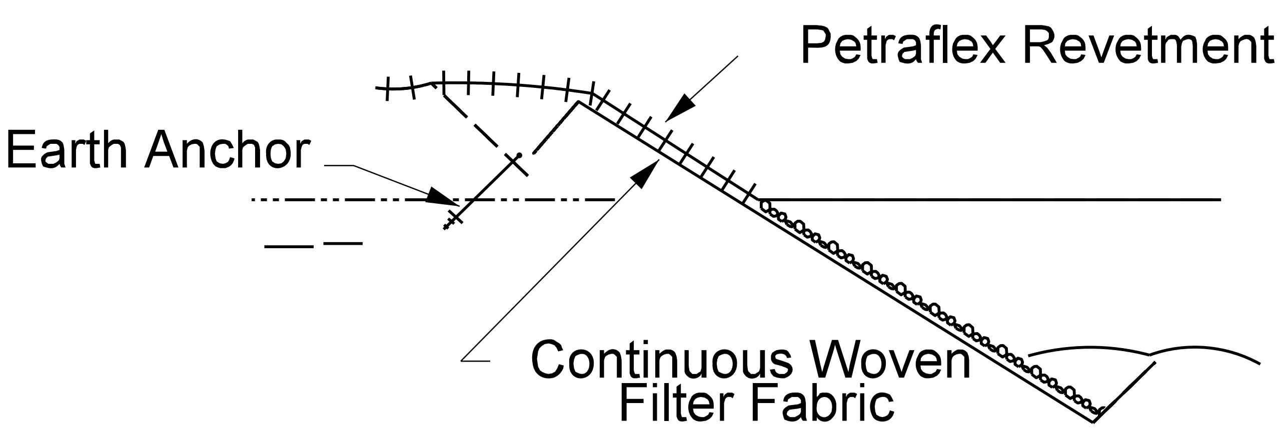
(b)

ARMORFLEX
((a) block detail and (b) revetment configuration)

Figure 38-6LL



(a)

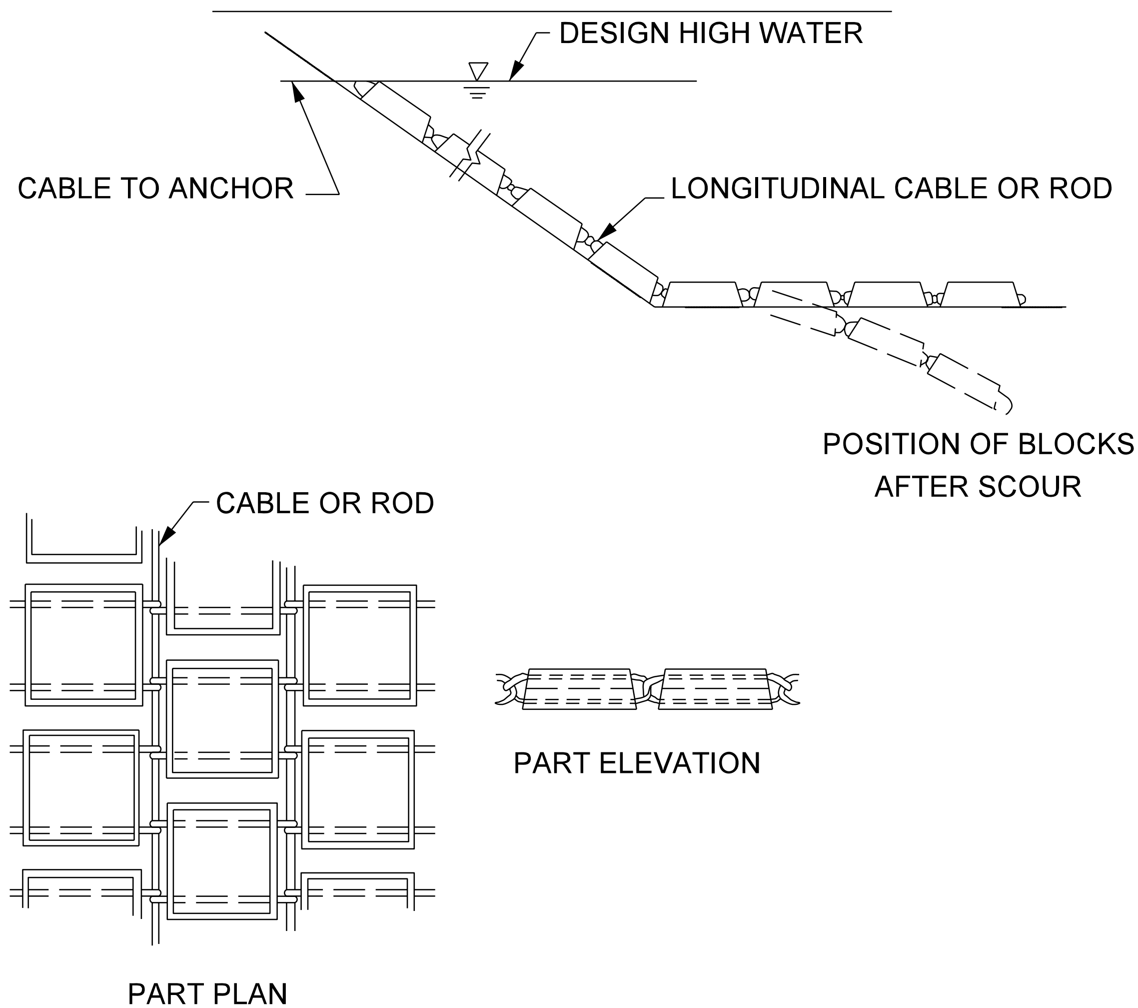


(b)

PETRAFLEX

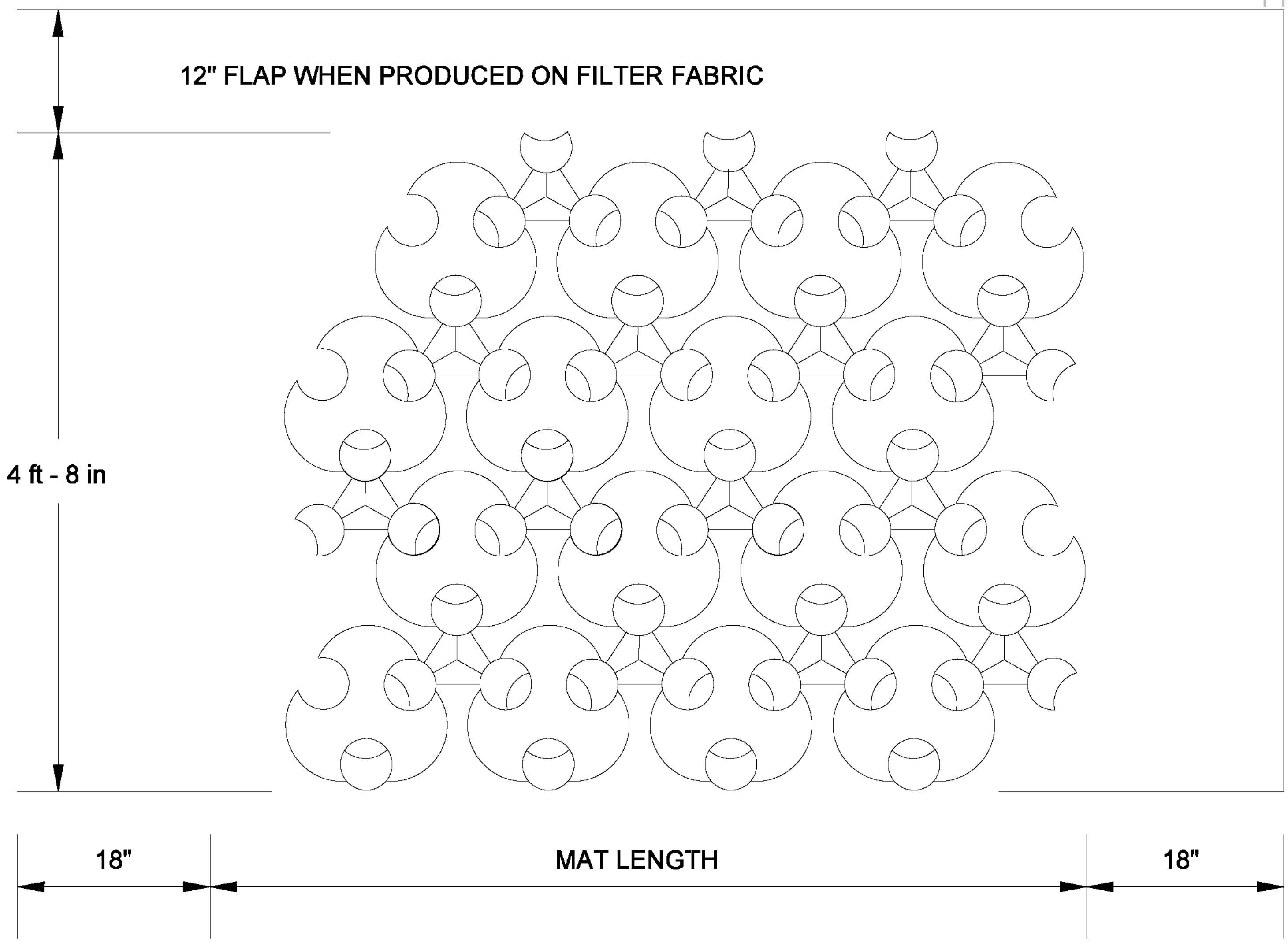
((a)block detail and (b) revetment configuration)

Figure 38-6MM



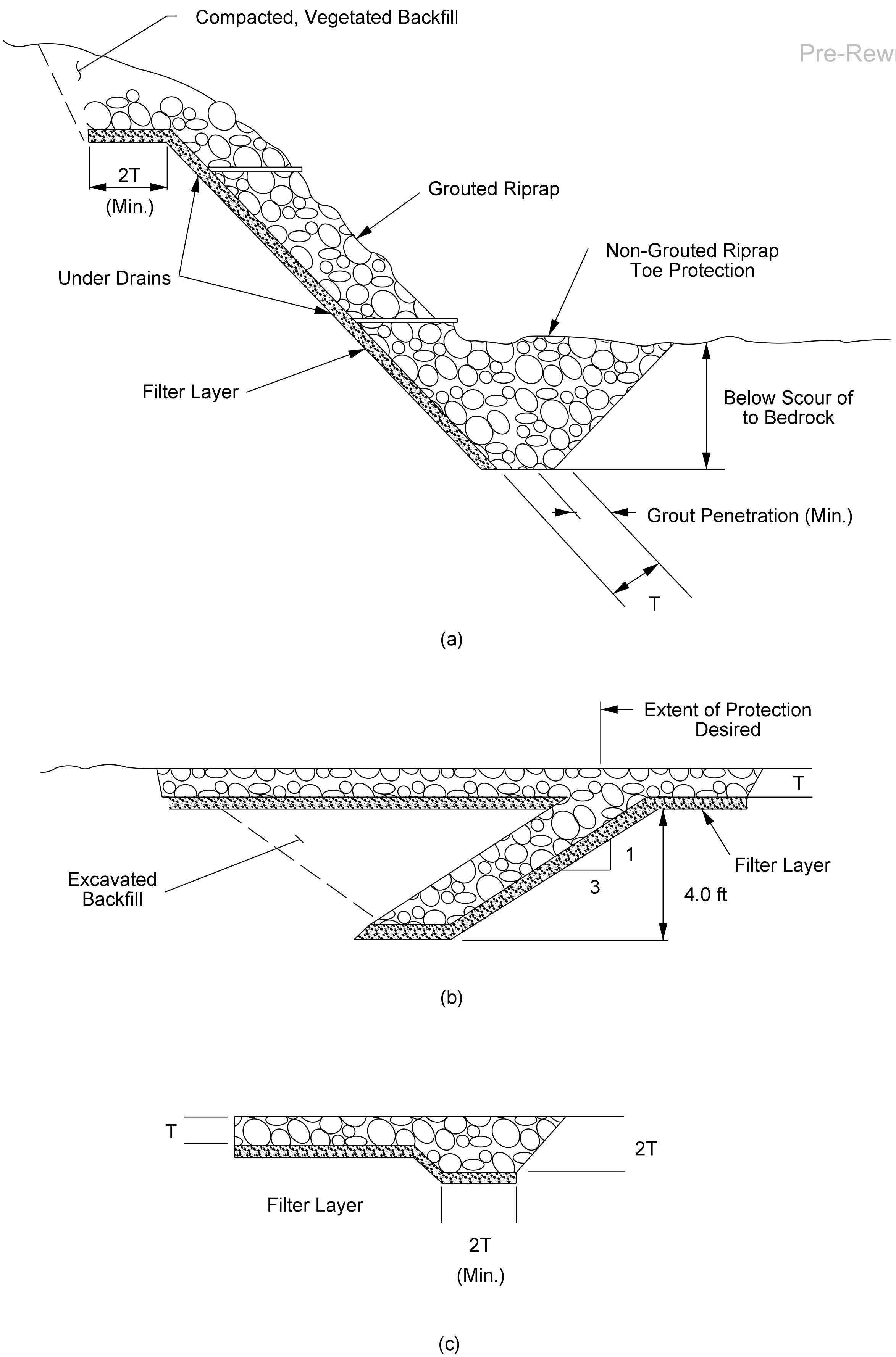
ARTICULATED CONCRETE REVETMENT

Figure 38-6NN



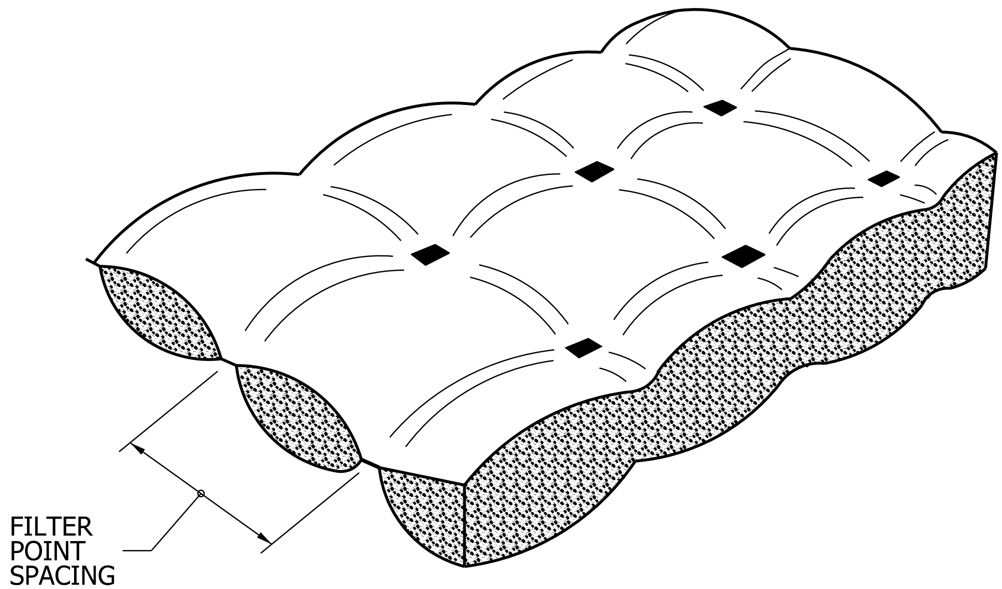
TRI-LOCK REVETMENT

Figure 38-6OO



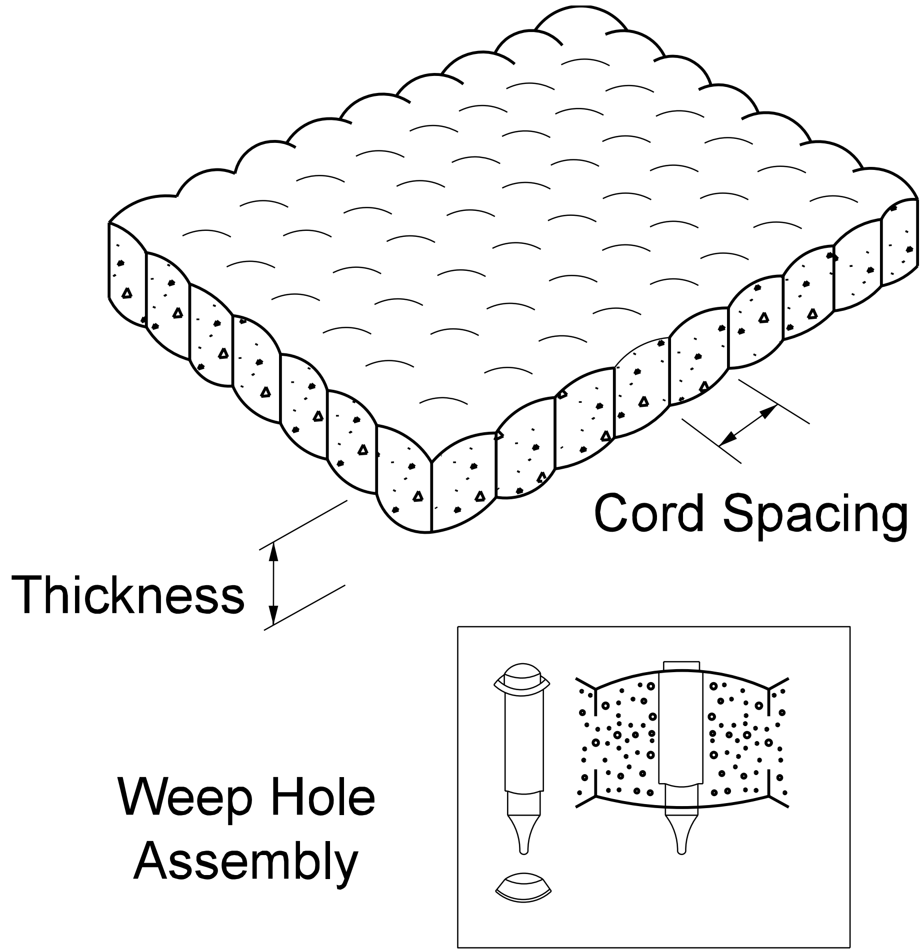
GROUTED RIPRAP SECTIONS
((a) Section A-A; (b) Section B-B; and (c) Section C-C)

Figure 38-6PP



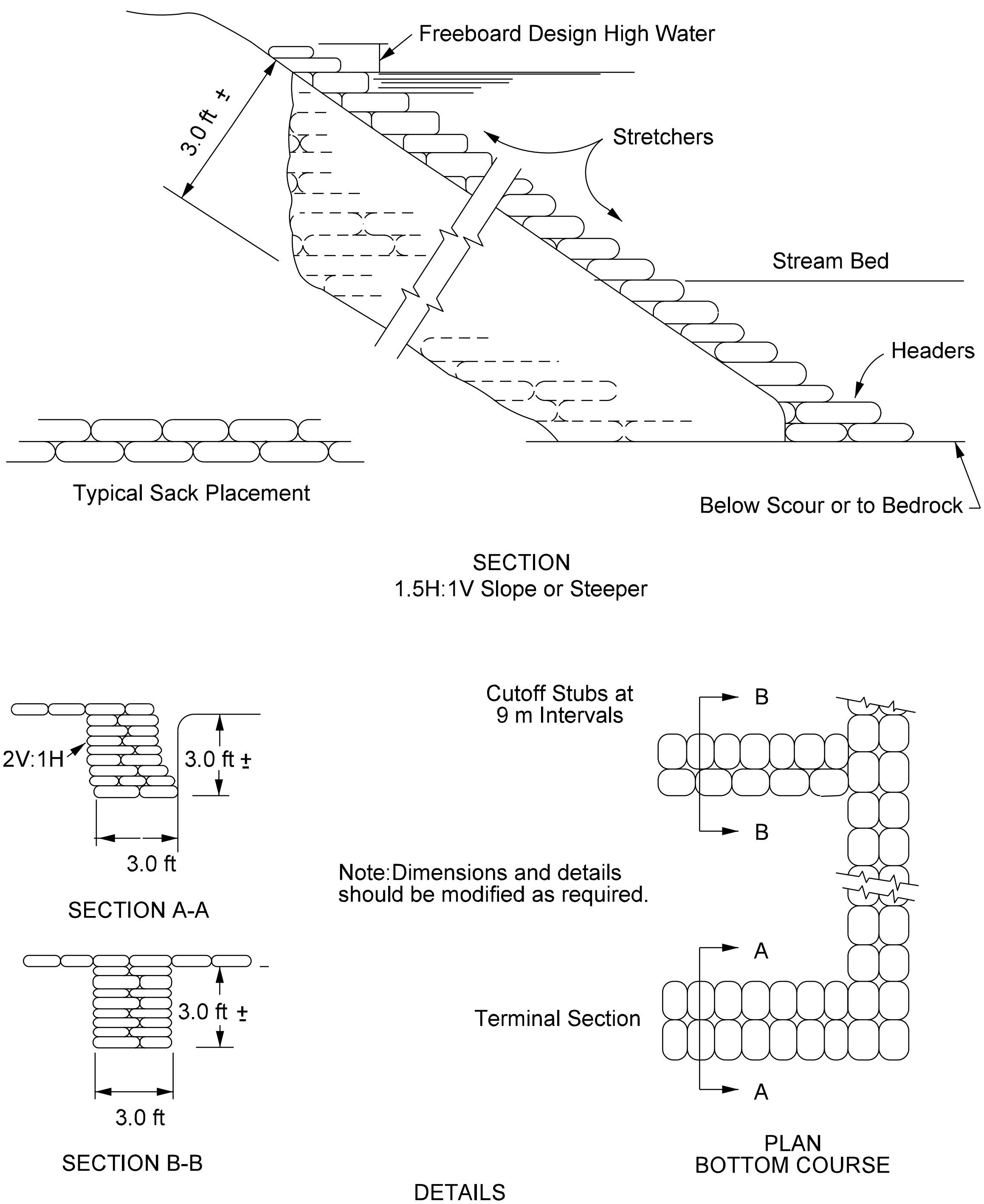
GROUTED FABRIC-FORMED REVETMENT
(Type 1)

Figure 38-6QQ



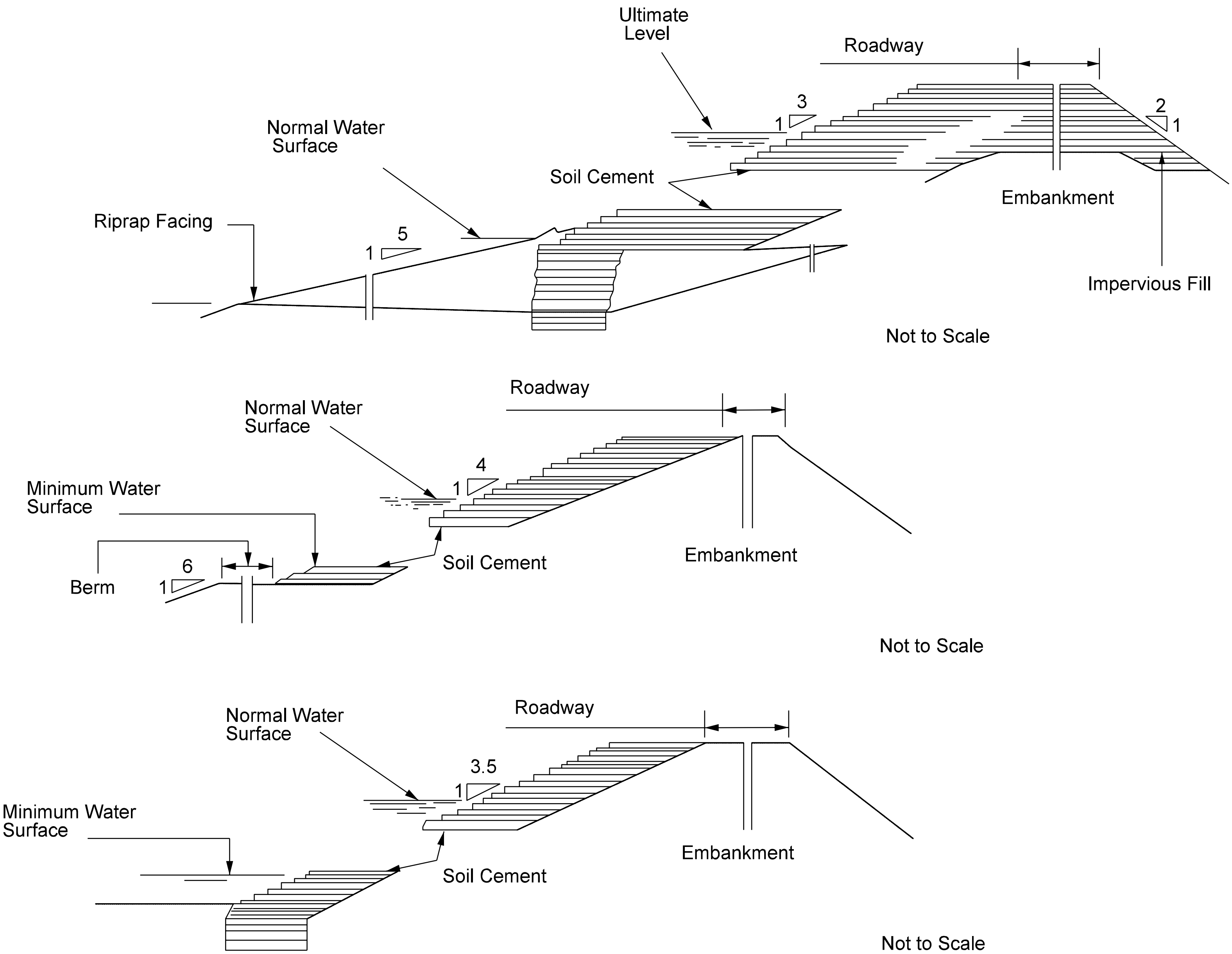
GROUTED FABRIC-FORMED REVETMENT
(Type 2)

Figure 38-6RR



TYPICAL SECTION AND DETAILS OF SACKED
CONCRETE SLOPE PROTECTION

Figure 38-6SS



DETAILS AND DIMENSIONS OF THREE SOIL-CEMENT FACINGS
DESIGN GUIDELINES

Figure 38-6TT

CHAPTER 204

Permanent Stormwater Quality Controls (Pre-Rewrite Version)

NOTE: This chapter did not exist prior to the Rewrite Version.

CHAPTER 205

Temporary Erosion and Sediment Control (Pre-Rewrite Version)

Design Memorandum	**Revision Date**	**Publication Date***	**Sections Affected**
12-22	Oct. 2012	Jan. 2013	Ch. 2015

*Revisions will appear in the next published edition of the *Indiana Design Manual*.

TABLE OF CONTENTS

LIST OF FIGURES

Figure Title

CHAPTER THIRTY-SEVEN

TEMPORARY EROSION AND SEDIMENT CONTROL

37-1.0 GENERAL INFORMATION [REV. JAN. 2011]

An erosion- and sediment-control plan is required to be submitted to IDEM for an INDOT project, or the applicable local Soil and Water Conservation District (SWCD) for an LPA project, to comply with 327 IAC 15-5 (Rule 5) as required by Section 9-2.05(06). INDOT *Standard Specifications* and *Standard Drawings* have been developed for erosion and sediment control to formalize and expand on existing measures available to the designer. These guidelines will aid the designer in choosing the appropriate measures and frequency of their use. Although Rule 5 requires erosion- and sediment-control measures for a specified minimum disturbed area, these measures should be applied to each project if land is disturbed. Formal submittal to comply with Rule 5 is not required where less than the minimum specified area is disturbed. However, where soil is disturbed, an erosion- and sediment-control plan must be developed. This applies particularly where sediment can enter a waterway.

The goals of erosion control and sediment control are different. The purpose of erosion-control measures is to prevent sediment from being mobilized on the project site. The purpose of sediment-control measures is to recapture soil that has been mobilized and to prevent it from leaving the construction site. Water flowing through a construction-disturbed area is to be filtered of sediment before it mixes with water which is not affected by construction operations. These guidelines concentrate on temporary erosion and sediment control measures. It is the designer's responsibility to include permanent measures where warranted. Temporary erosion-control measures should be in compliance with the construction-zone clear-zone criteria shown in Chapter Eighty-two. The erosion- and sediment-control plan should address erosion and sediment control during the entire construction process. This may mean that different measures will be used during different phases of construction. Allowance should be made for changes in the field to fit existing conditions or the use of different measures where they are more appropriate. The erosion- and sediment-control measures have been listed in groups according to their use. Some of the measures may be used in multiple applications.

A copy of Rule 5 is available via the Indiana Department of Environmental Management (IDEM) website, at http://www.in.gov/idem/4221.htm. It lists items that should be submitted with the erosion- and sediment-control plan to the applicable SWCD. The designer is responsible for all items listed in Figure 37-1A, Erosion Control Sediment Plan Technical Review

Checklist. An editable version of this form is available on the Department's website, at. http://www.in.gov/dot/div/contracts/design/dmforms/index.html. The erosion- and sediment-control plan should be prepared and submitted to the appropriate SWCD. The Notice of Intent letter should be filed with IDEM. The submittals are as follows.

1. Plans developed for a Central-Office project will be filed by the Production Management Division's Permit Coordinator.

2. Plans developed by a district office will be filed by district personnel.

3. Plans developed for a local public agency project will be filed by the local agency.

Guidance on plans development in accordance with 327 IAC 15-5 (Rule 5) Plan Elements appears on IDEM's website, at http://www.in.gov/idem/5419.htm. Once on this page, click on the Guidance for Construction Plan/Storm Water Pollution Plan Development hyperlink, which appears under Step 1, fourth paragraph.

37-2.0 SITE ANALYSIS [REV. JAN. 2011]

The erosion- and sediment-control plan should identify control measures that will be used to minimize erosion and off-site sedimentation. It serves as a blueprint for the location, installation, and maintenance of these measures.

In preparing the erosion- and sediment-control plan, the designer should start by looking at local drainage patterns, soil types, and topography. The watershed area of each concentrated water flow entering the project site at various locations should be determined and shown on the erosion- and sediment-control plan at each point of entry. Where reasonable, off-site waters should be isolated and allowed to pass through the project site. Sediments from on-site sources should be captured prior to leaving the site. The method of treatment depends upon the drainage area and soil types.

Providing a vegetated ground cover is the most important factor in terms of preventing erosion. If the existing vegetation is to be disturbed, appropriate erosion- and sediment-control measures should be utilized. If utility features traverse the site, their relocation should be taken into consideration in designing these measures.

The following principles of erosion and sediment control should be utilized.

1. The physical characteristics of the site should be assessed, including topography, soils, and drainage to determine how to best minimize erosion and sedimentation.

2. The erosion- and sediment-control plan should be designed to include measures that will keep sediment on the construction site as much as possible.

3. Where reasonable, perimeter dikes or waterways should be used to divert or intercept off-site runoff. In evaluating the decision as to whether diversion or interception of off-site runoff is reasonable, the increased sizing of a proposed BMP structure should be considered to treat off-site runoff if such diversion is not performed.

4. Where reasonable, a perimeter dike or waterway should be used to divert or intercept off-site runoff.

5. Measures to slow runoff and allow deposition of sediment should be designed using grading and sediment barriers to break up long, steep slopes.

6. Temporary seeding should be utilized where applicable.

7. Runoff velocity should be reduced by maintaining existing vegetative cover, preserving a natural buffer strip around the lower perimeter of the disturbed land, and installing a perimeter control such as a sediment barrier, silt fence, filter, dike, or sediment basin or trap.

8. The amount of disturbed area on the construction site should be minimized.

9. Construction phasing and incremental final stabilization of areas should be considered where such measures can adequately satisfy the intended use requirements of such areas.

10. The impervious surface should be kept at the minimum required to satisfy public safety needs. The use of porous material should be encouraged outside the mainline pavement area.

The construction-zone clear zone should be determined in order to select the appropriate erosion-control measures. Chapter Eighty-two includes the information necessary to determine the construction-zone clear zone. Straw bales should be used instead of riprap for a ditch check in the construction-zone clear zone.

Temporary entrance points will likely be required for the contractor's use from off-road locations on to public roads. The INDOT *Standard Specifications* require the contractor to install stable construction entrances at these points in order to control movement of sediment off the project site by construction traffic. If the designer determines that such entrances are required, an undistributed quantity of 100 tons of No. 2 stone should be included in the preliminary estimate of quantities and cost estimate. This minimum quantity should be sufficient to provide two stable construction entrances. If the designer anticipates that additional entrances will be required, the quantity should be adjusted accordingly.

Within the construction clear zone, straw bales, fiber wattle rolls, or other approved means should be considered instead of riprap.

37-3.0 TEMPORARY EROSION- AND SEDIMENT-CONTROL MEASURES

37-3.01 Protection of Adjacent Area

These measures are used to minimize sediment to an area adjacent to the disturbed area. These measures include silt fence, vegetative filter strip, sediment trap, and sediment basin.

37-3.01(01) Silt Fence

A silt fence is a fabric barrier used to retain sediment from a small, sloping, disturbed area by reducing the velocity of sheet flow. See the INDOT *Standard Drawings* for details. A silt fence requires a trench for proper installation and should not be used on a fill slope. A silt fence captures sediment by ponding water to allow deposition, not by filtration. Although the practice works best in conjunction with other erosion-control measures, it can be effective when used alone under the proper field conditions. A silt fence is not recommended to divert water; nor is it to be used across a stream, channel, or where concentrated flow is anticipated.

Use of a silt fence is limited to a disturbed site in a drainage area. The use of a silt fence is further restricted by the slope (grade), as indicated in Figure 37-3A. A silt fence should be installed nearly level, approximately following the land contour. Ideally, a silt fence should be installed at least 10 ft from the toe of slope to provide a broad, shallow sediment pool with increased storage capacity.

The length of silt fence should be sufficient to encompass the boundaries of the toe of the slope with the ends of the fence terminated upslope. The silt fence should terminate at adjacent erosion control measures or at a stabilized area.

For an area greater or a slope steeper than provided in Figure 37-3A, additional erosion-control measures should be utilized such as a slope drain, sediment trap or basin, temporary or permanent stabilization, etc.

Where site conditions exceed the limits shown in Figure 37-3A, other appropriate erosion- and sediment-control measures should be implemented in conjunction with the silt fence.

37-3.01(02) Vegetative Filter Strip

Leaving existing grassy vegetation in place is the most effective method for erosion control. It may not be practical or possible to leave all existing vegetation in place. It is still best to maintain as much existing vegetation as possible; it will reduce erosion and will act as a filter trap for sediment from an upslope area. A vegetative filter strip is an area where the ground cover is to be left undisturbed to filter runoff from a drainage area. A filter strip is located between a sediment-producing site and a downslope site or watercourse. The locations which are not to be disturbed by the contractor should be shown in the erosion- and sediment-control plan.

Applications for this sediment-control measure include an area adjacent to right-of-way limits, roadside ditch, relocated or existing waterway, or wetlands.

The vegetative filter strip's effectiveness is increased when it is used in conjunction with other measures such as a silt fence, inlet protection, or sediment trap or basin, etc.

The effectiveness of a vegetative filter strip is dependent upon the slope of the undisturbed area. The designer should evaluate and identify all potential areas for use of this control measure.

It is preferable that the vegetative strip is on the flatter area beyond the toe of slope. This does not preclude leaving as much vegetation on the slope as possible. If site conditions exist that do not allow for locating a filter strip on the flatter ground, the designer should preserve vegetation on the slope. The vegetative filter strip may be considered for an undisturbed area within the construction limits.

Where a vegetative filter strip is to be used independent of other measures, the strip should be in accordance with or exceed the requirements shown in Figure 37-3C.

Where existing vegetation cannot be in accordance with the minimum requirements shown in Figure 37-3C, there are still advantages to leaving the vegetation in place. However, the vegetative strip should be used in conjunction with other appropriate practices. If a silt fence is to be used in conjunction with a filter strip, the fence should be installed 10 ft from the toe of the slope on the flatter ground.

37-3.01(03) Sediment Trap

A sediment trap is an area located in a ditch line that is used to temporarily pond runoff, which allows the sediment to be contained. It is the last measure used in a ditch to filter water before it enters another legal drainage body. If the ditch grade is 5% or steeper, or if the ditch is 3500 ft or longer, the last two measures in the ditch should be sediment traps. Revetment riprap should be used in construction of a sediment trap.

If used independent of other sediment-control measures, the sediment trap should be designed for up to a maximum drainage area of 5 ac. Where right of way is limited, the sediment trap should be designed considering the space rather than the drainage area. In this situation, other sediment-control measures should be specified in conjunction with the sediment trap. Ideally, the trap should be designed to store sediment for a minimum disturbed volume of 65 yd^3/ac. In order to determine the volume of the trap; the watershed that is tributary to the sediment trap should be calculated.

Figures 37-3E, 37-3F, and 37-3G indicate the minimum spillway design for a sediment trap.

The sediment-trap design depends on the geometric characteristics of the proposed ditch as follows:

1. ditch grade;
2. ditch shape (flat bottom or V-ditch); and
3. sideslopes (fore- and backslope).

Figure 37-3G indicates the minimum spacing for sediment traps, based on the flood pool length, so that the next measure would not encroach into the pool of the previous one. The procedure is as follows.

1. Select the largest sediment trap, by spillway height, which can physically fit in the proposed ditch cross section

2. Check the proposed ditch grade directly upstream of the approximate location of the sediment trap. The grade should be continuous.

3. From the table for the appropriate ditch, find the required sediment-traps spacing. The provided spacing should be at least that shown in Figure 37-3G.

37-3.01(04) Sediment Basin [Rev. Jan. 2011]

A sediment basin is a water-impoundment structure formed with an embankment or by excavating a basin. It is used to prevent offsite sedimentation by retaining sediment on the construction site. A sediment basin should be a primary consideration for a new-construction project where there is adequate right of way. It should be used within an interchange, rest area, weigh station, or replacement wetlands. Where right of way is limited, the sediment basin should be designed considering available space rather than drainage area. In this situation, other control measures should be specified in conjunction with the sediment basin depending on site conditions.

A sediment basin is the last control measure encountered by runoff before it leaves the construction site. It is about twice as long as it is wide, but it must be shaped to fit the area it will be used in. It should therefore be designed and detailed for each specific site, as no standard details have been developed. However, a schematic detail is shown in Figure 37-3H. If used independent of other sediment-control measures, the basin should be designed for a drainage range of 1 to 30 ac. The basin should be designed to store a minimum water volume of 65 yd^3/ac for the watershed. If the watershed area is greater than 30 ac, additional consideration should be given.

A wetland-replacement site or detention pond may be used temporarily as a sediment basin. For guidance in the use of a wetland-replacement site as a sediment basin, contact the Production Management Division's Hydraulics Team. If the permanent control structure of the wetland-replacement site or detention pond is a pipe, a temporary perforated riser should be used to dewater the basin allowing for adequate residence time in the basin.

This sediment-control measure should not be used where failure of the embankment can endanger life or property. Temporary right of way may be provided where possible.

37-3.02 Slope

The following measures are used to temporarily control erosion on a slope. These measures include interceptor ditch and slope drain, vegetative strip in a cut section, temporary seeding and temporary mulching, erosion control blanket, and surface roughening.

37-3.02(01) Interceptor Ditch and Slope Drain

An interceptor ditch, in combination with temporary or permanent seeding, protects a work area from runoff and diverts water to a sediment trap or a protected-flow area. An interceptor ditch is constructed to protect a work area, fill slope, or cut slope. It should be constructed and graded to drain to provide positive drainage. A slope drain is a pipe drain used in conjunction with an interceptor ditch to convey runoff down a slope without causing erosion. An interceptor ditch with a slope drain should be specified at the top of a fill slope to divert runoff from the top of the embankment, and control where the runoff is discharged. Where cut or fill height exceeds 10 ft, a slope drain should be used. The INDOT *Standard Drawings* specify the pipe diameter and its drainage area. This information is useful in determining the spacing of slope drains.

The contractor should be permitted to use a temporary-pipe slope drain or an open slope drain. The slope drain should be lengthened as the embankment is extended upward. A slope drain should not be outlet directly into a stream due to the possible conveyance of sediment from the top of the embankment. Instead, it should be outlet onto a riprap splash pad and into another sediment-control measure.

37-3.02(02) Interceptor Ditch or Vegetative Strip in Cut Section

An interceptor ditch in accordance with Section 37-3.02(01) may be warranted in a cut section at the right-of-way line to divert runoff from the construction site to adjacent properties. Another method for addressing this situation is to specify that a vegetative strip be left at the right-of-way line to filter runoff from the construction site. The strip should be shown on the erosion- and sediment-control plan, to indicate that the contractor should not clear the area.

37-3.02(03) Temporary Seeding and Temporary Mulching

Temporary seeding and mulching are used to reduce erosion and sedimentation damage by stabilizing a disturbed area where additional work is not scheduled for at least 15 calendar days. Temporary seeding reduces problems associated with mud or dust from a bare-soil surface during construction, and also reduces sediment runoff downstream by providing temporary stabilization. Mulching protects the soil from the impact of wind and water, prevents the soil from crusting, conserves moisture, and promotes seed germination and growth. Temporary seeding and mulching are often used in concert, although temporary mulching should be used where temporary seeding is not reasonable, such as during the winter months. The seasonal requirements for the use of temporary seeding and mulching are shown in the INDOT *Standard Specifications*.

The pay quantity for temporary seeding should be determined based on the contract type as follows:

1. <u>Bridge Contract</u>. Based on the same area as the permanent seeding.

2. <u>Road Contract</u>. Based on one-half the area of the permanent seeding.

3. <u>Maintenance, Traffic, or Resurfacing Contract</u>. A pay quantity should not be included unless an analysis of the project shows it to be necessary.

A different pay quantity for temporary seeding may be shown if an analysis of the project shows it to be warranted.

When mulch is used during certain periods of the year instead of temporary seed, a pay quantity for mulch should also be included in a road or bridge contract. The quantity specified should be doubled, to cover the areas to be temporarily and permanently seeded. An additional quantity of mulch should not be shown if a quantity is already included.

37-3.02(04) Erosion-Control Blanket and Surface Roughening

If an erosion-control blanket is required as a permanent measure, and the special provisions require their early installation, it may be used as a temporary erosion-control measure. Surface roughening is required by the INDOT *Standard Specifications* for construction of erosion-control methods. The designer should not consider the measure as part of the temporary erosion- and sediment-control plan.

37-3.03 Side Ditch

The measures used to control sediment in a side ditch include check dam, sediment trap, and grass- or riprap-lined channel. Figure 37-3L shows the measures to be used with a disturbed ditch. Figure 37-3M shows the measures to be used with an undisturbed ditch.

37-3.03(01) Check Dam

A check dam is used to reduce erosion in a drainage channel by slowing the velocity of the flow. A check dam is used in a channel that is degrading but where permanent stabilization measures are impractical due to their short period of usefulness, or in an eroding channel where construction delays or weather conditions prevent timely installation of erosion-resistant linings. A check dam should not be used in an intermittent or perennial stream. A check dam should be used only a drainage area less than or equal to 2 ac. A revetment-riprap check dam should be specified for use if it is outside the construction clear zone and is not exposed to public traffic. A straw-bale check dam should be specified for use only if it is inside the construction clear zone and exposed to public traffic. See the INDOT *Standard Drawings* for details.

A check dam should be wide enough to traverse the ditch section to force water to flow over the check dam instead of around the ends.

Figures 37-3J and 37-3K provide guidance in developing quantities based on the spacing of revetment-riprap check dams. The spacing dimensions listed below were determined based on a 4.0 ft bottom ditch with 4:1 fore- and backslopes. For another ditch cross section, the spacing should be recalculated. It is not necessary to show the spacing on the plans. The check dams should be spaced such that the top of the downstream check is at the same elevation as the toe of the adjacent upstream check dam.

The check-dam weight, W_{RR}, in tons, should be determined by using either of the formulas as follows:

For 2-ft depth, $W_{RR} = 1.5\left[\frac{a}{3} + \frac{(b+c)}{2}\right]$

For 3-ft depth, $W_{RR} = 1.5\left[\frac{a}{2} + (b+c)\right]$

The coarse aggregate No. 5 and geotextile fabric required with this work are not separate pay items, therefore, quantities need not be determined.

Geotextile is required under the riprap and as an apron as shown on the INDOT *Standard Drawings.* The area to be covered with geotextile is as follows:

Area (sys) = 1/9 [12a + 21 (b + c)]

37-3.03(02) Sediment Trap in Side Ditch

A sediment trap is used in a side ditch in lieu of, or in conjunction with, a check dam. It allows sediment to settle out of the water instead of damming the sediments as with a check dam. Riprap quantities are shown in Figures 37-3N, 37-3 O, and 37-3P. See Section 37-3.01(03) for details.

37-3.03(03) Grass- or Riprap-Lined Channel

Grass seed or riprap should be placed in a channel early in the construction process. This measure may be used as a temporary erosion- and sediment-control measure, and then retained as a permanent feature.

<u>**37-3.04 Stream**</u>

A sediment basin and silt fence is used to control sediment at a stream. Although all of the measures contribute to the reduction of sediment that could enter a stream, the measures described below apply adjacent to the stream.

37-3.04(01) Sediment Basin Alongside Stream

A sediment basin is used as a last sediment-control measure, in a line of several measures, before runoff is allowed to enter a waterway. This measure may not be necessary on a project with a small disturbed area. A sediment basin allows sediments to settle out of the water. See Section 37-3.01(04) for information concerning this measure.

37-3.04(02) Silt Fence Alongside Stream

Where a stream is adjacent to an exposed fill slope, the stream should be protected from sediment by use of a silt fence alongside the stream as described in Section 37-3.01(01).

37-3.05 Inlet

Prevention of sedimentation of a stream includes protection of each stormwater inlet. The inlet-protection measures described below have been established to handle a maximum drainage area of 1 ac. If the drainage area is greater than 1 ac per inlet, additional measures should be used in conjunction with the inlet-specific protection measures.

37-3.05(01) Ditch-Inlet Protection

Ditch-inlet protection is used to keep sediment from entering an inlet. Ditch-inlet protection is needed only where there is likelihood that sediment will enter the inlet. The designer should include ditch-inlet protection in the erosion- and sediment-control plan only where such plan calls for disturbing the area around the inlet. The contractor should be given the option of using silt-fence, slotted-barrel, or aggregate-ring inlet protection. Each measure captures sediment at the approach to a storm-drain inlet, allowing full use of the storm-drain system during the construction period. See the INDOT *Standard Drawings* for details.

37-3.05(02) Curb-Inlet Protection

Curb-inlet protection should be provided where a road is still closed to traffic, or is being used by the contractor as a haul road, and there is a reasonable potential for sediment to wash onto the road from surrounding areas or be tracked by construction equipment. There are no sediment-control measures that the designer may consider regarding this situation. Measures which the contractor must consider are described in the INDOT *Standard Specifications.*

The following is a check for the following items to be adequately addressed on the plans. *(The plans must include appropriate legends, scales, and north arrow.)*

PROJECT INFORMATION

Yes	No		
☐	☐	1A	Project location map *(Show project in relation to other areas of the county.)*
☐	☐	1B	Narrative describing the nature and purpose of the project
☐	☐	1C	Location of planned and/or existing roads, utilities[1], structures, highways, etc.
☐	☐	1D	Building locations[2]
____	____	1E	Land use of adjacent areas *(Show the entire upstream watershed and adjacent areas within 500 ft of the property lines.)*[3]

TOPOGRAPHIC, DRAINAGE, AND GENERAL SITE FEATURES

Yes	No		
☐	☐	2A	Existing vegetation *(Identify and delineate.)*
☐	☐	2B	Location/name of all wetlands, lakes, and water courses on and adjacent to site
☐	☐	2C	100-year floodplains, floodway fringes, and floodways *(not applicable if none.)*[4]
☐	☐	2D	Soils information *(If hydric soils are present, it is the responsibility of the owner to investigate the existence of wetlands and obtain appropriate permits.)*[5]
☐	☐	2E	Existing/planned contours[6] at intervals appropriate to indicate drainage patterns
☐	☐	2F	Locations of specific points where stormwater discharge will leave the site
☐	☐	2G	Identify all receiving waters *(If discharge is to a separate municipal storm sewer, identify the name of the municipal operator and the ultimate receiving water.)*
☐	☐	2H	Potential areas where storm water may enter groundwater *(note if none.)*
☐	☐	2 I	Location of stormwater system *(Include culverts, storm sewers, channels, and swales.)*

[1] Use best available information.
[2] Within project area.
[3] Attach the appropriate United States Geological Survey topographic map.
[4] This item is satisfied by showing the 100-year flood elevation on the plans.
[5] No formal submittal from the designer is necessary for this item.
[6] Profiles or contours where available.

LAND-DISTURBING ACTIVITIES

Yes	No		
☐	☐	3A	Location and approximate dimensions of all disturbed areas *[i.e., construction limits] (Areas where vegetation cover will be preserved should clearly be designated.)*
☐	☐	3B	Soil stockpiles and borrow areas[7] *(Show location or note if none.)*

EROSION AND SEDIMENT CONTROL MEASURES

Yes	No		
☐	☐	4A	Sequence of each measure to be implemented[7] *(Relative to earth-disturbing activities.)*
☐	☐	4B	Monitoring and maintenance guidelines for each measure[8]
☐	☐	4C	Perimeter sediment control measures *(Location, construction detail, dimensions, specifications.)*
☐	☐	4D	Temporary seeding *(Specifications including seed mix, fertilizer, lime, and mulch rates.)*[8]
☐	☐	4E	Temporary erosion and sediment control measures *(Location, construction details dimensions, specifications.)*
☐	☐	4F	Permanent erosion and sediment control measures *(Location, construction details, dimensions, specifications.)*
☐	☐	4G	Storm drain inlet protection *(Location, construction details, dimensions, specifications.)*
☐	☐	4H	Storm drain outlet protection *(Location, construction details, dimensions, specifications.)*
☐	☐	4 I	Stable construction entrance *(Location, construction details, dimensions, specifications.)*[7]
☐	☐	4J	Permanent seeding *(Specifications including seed mix, fertilizer, lime, and mulch rates.)*[8]

EROSION AND SEDIMENT CONTROL PLAN
TECHNICAL REVIEW CHECKLIST

[7]To be submitted by the contractor following contract award.
[8]This item addressed in Indiana Department of Transportation *Standard Specifications*.

Slope, *a*:1 (Grade, %)*	Maximum Slope Length, *B***
Flatter than 50:1 (< 2%)	100 ft
50:1 ≤ Slope < 20:1 (2% ≤ Grade < 5%)	80 ft
20:1 ≤ Slope < 10:1 (5% ≤ Grade < 10%)	50 ft
10:1 ≤ Slope < 5:1 (10% ≤ Grade < 20%)	30 ft
5:1 or Steeper (≥ 20%)	20 ft

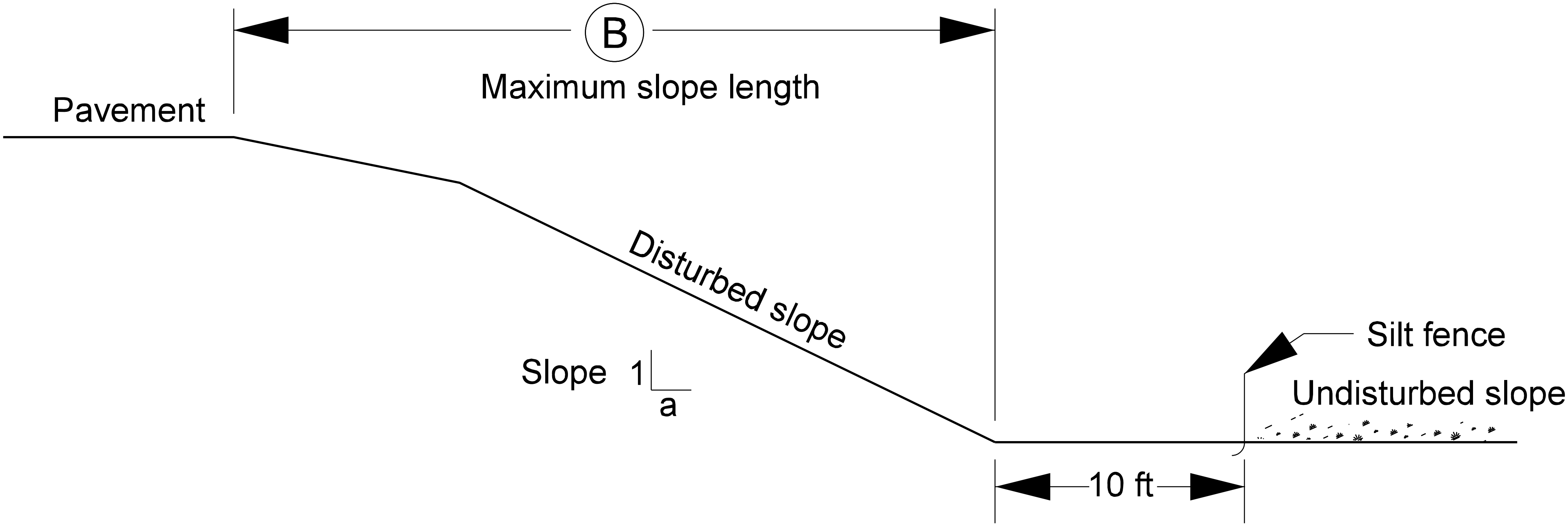
* *Steepest portion of the slope.*

** *The length of the slope above the fence that will be contributing runoff. This is not to be interpreted as a spacing distance between multiple rows of fence down a slope. Multiple or terraced rows of silt fence are not an approved application of this sediment-control measure.*

Figure 37-3B should be used with this figure.

SLOPE LENGTH FOR SILT FENCE

Figure 37-3A



SILT FENCE APPLICATION

Figure 37-3B

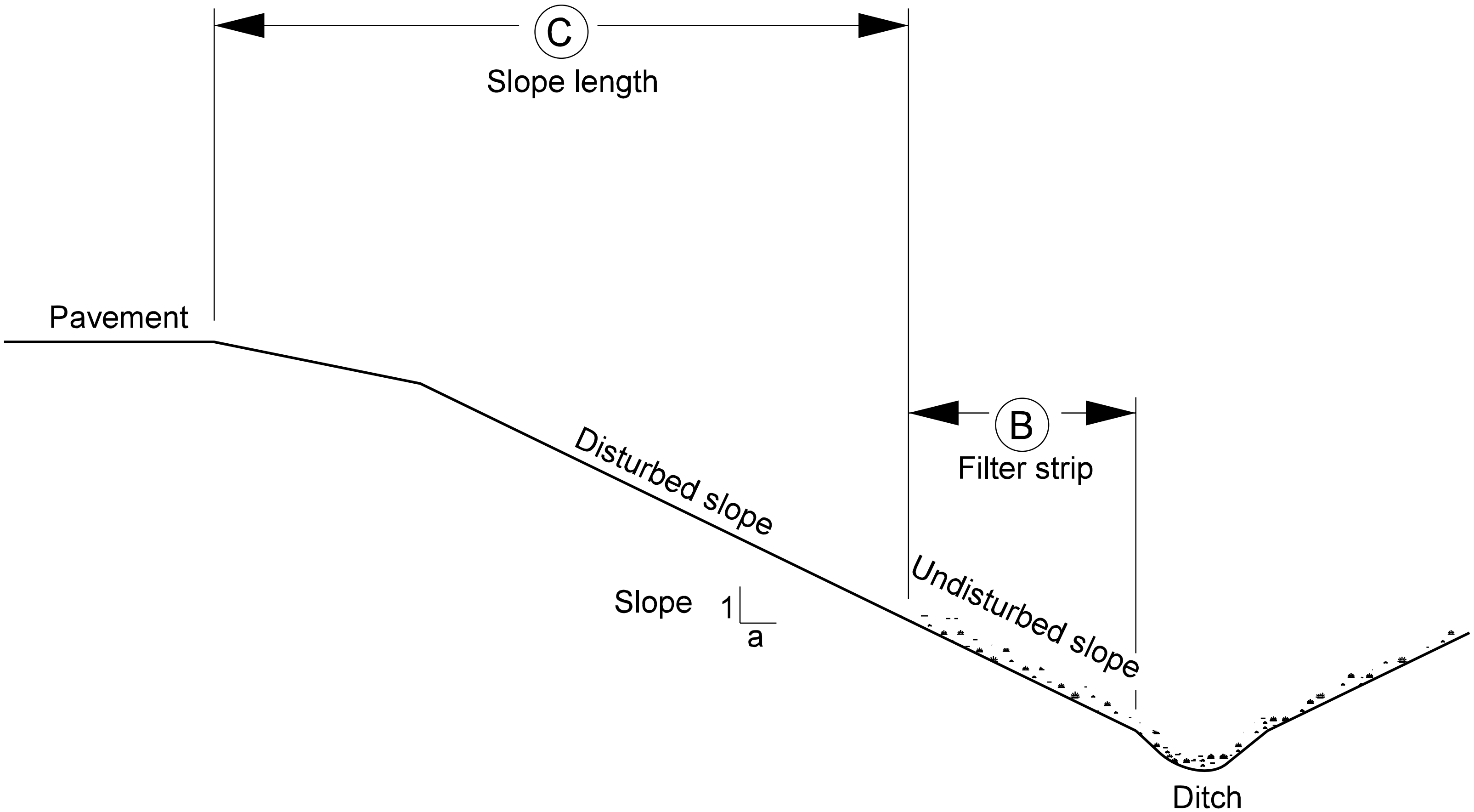
Slope, *a*:1 (Grade, %) *	Minimum Recommended Filter Width, *B*	Maximum Slope Length, *C* **
Flatter than 20:1 (< 5%)	20 ft	80 ft ≤ *C* < 100 ft
20:1 ≤ Slope < 10:1 (5% ≤ Grade < 10%)	40 ft	50 ft ≤ *C* < 80 ft
10:1 ≤ Slope < 5:1 (10% ≤ Grade < 20%)	60 ft	30 ft ≤ *C* < 50 ft
5:1 or Steeper (≥ 20%)	80 ft	20 ft ≤ *C* < 30 ft

*   *Steepest portion of the slope*
** *Length of the slope above the filter strip that will contribute runoff.*

Note: Figure 37-3D should be used with this figure.

MINIMUM FILTER-STRIP APPLICATION

Figure 37-3C



VEGETATIVE FILTER STRIP APPLICATION

Figure 37-3D

Spillway Height (ft)	STORAGE CAPACITY, ft^3 / SPACING, ft														
	2:1 Foreslope, 2:1 Backslope Ditch Grade, up to:					3:1 Foreslope, 2:1 Backslope Ditch Grade, up to:					3:1 Foreslope, 3:1 Backslope Ditch Grade, up to:				
	2%	3%	4%	5%	6%	2%	3%	4%	5%	6%	2%	3%	4%	5%	6%
2.0	388/67 *	247/43 *	177/33 *	141/27 *	141/20 *	460/83 *	317/53 *	247/40 *	176/33 *	177/27 *	565/100	388/67	283/50	212/40	177/33
2.5	741/127	494/87	388/63	282/50	247/43	918/160	636/107	459/80	388/63	317/53	1130/193	742/127	565/97	459/77	388/63
3.0	1271/220	847/147	635/110	529/90	423/73	1624/277	1059/183	812/137	636/110	529/93	1942/333	1271/220	953/167	777/133	636/110

** This spacing is for information only. Use the minimum spacing from Figure 37-3G instead.*

SEDIMENT TRAP IN V-DITCH

Figure 37-3E

Spillway Height (ft)	STORAGE CAPACITY, ft^3 / SPACING, ft														
	31 Foreslope, 31 Backslope Ditch Grade, up to:					41 Foreslope, 31 Backslope Ditch Grade, up to:					41 Foreslope, 41 Backslope Ditch Grade, up to:				
	2%	3%	4%	5%	6%	2%	3%	4%	5%	6%	2%	3%	4%	5%	6%
2.00	565/77 *	565/77 *	423/57	353/47	283/37	953/130	635/87	460/63	388/53	317/43	1059/147	706/97	530/73	424/60	353/47
2.50	1553/217	1060/143	777/110	636/87	530/73	1766/247	1165/193	883/123	706/100	600/83	1942/280	1306/187	953/140	777/113	636/93
3.00	2578/367	1730/243	1271/183	1024/147	848/123	2896/420	1942/280	1448/210	1165/167	953/140	3213/477	2154/317	1624/237	1271/190	1059/160

Note: The values are calculated for a 3-ft width flat-bottom ditch. They are also suitable for use for a 4-ft width ditch, due to the proximity of the values.

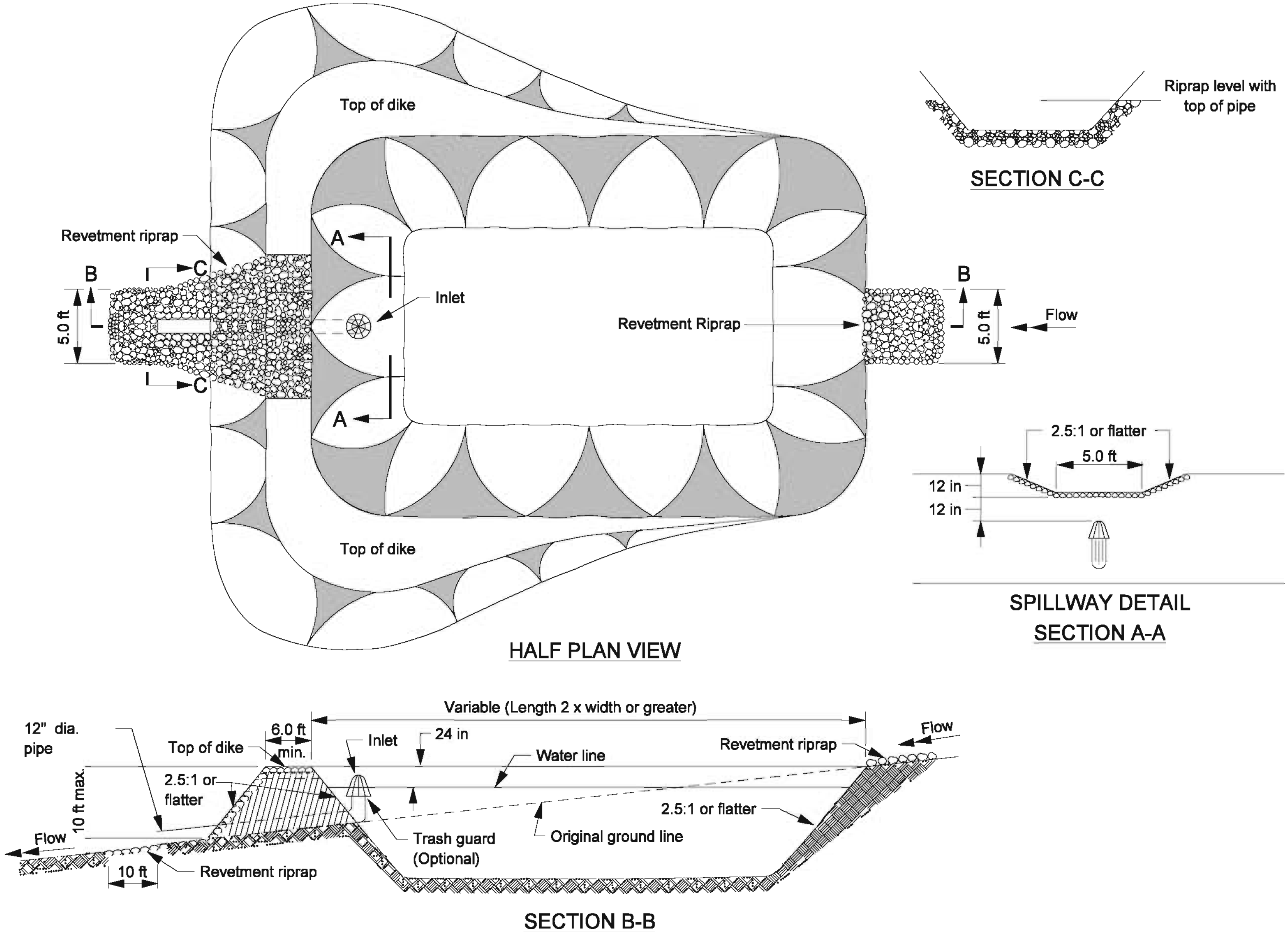
SEDIMENT TRAP IN FLAT-BOTTOM DITCH

Figure 37-3F

Spillway Height, ft	Ditch Grade, up to:				
	2%	3%	4%	5%	6%
2.0	100	70.0	50.0	40.0	35.0
2.5	125	84.0	65.0	50.0	40.0
3.0	150	100	75.0	60.0	50.0
3.5	175	117	90.0	70.0	60.0

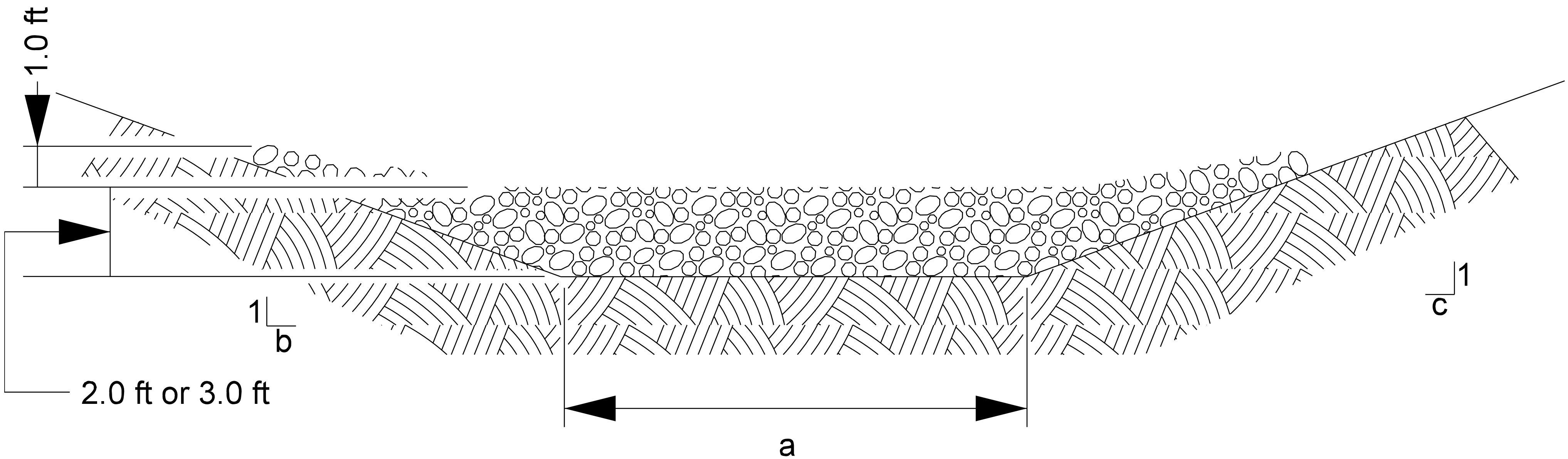
FLOOD-POOL LENGTH (ft)

Figure 37-3G



SEDIMENT BASIN DETAILS

Figure 37-3H



CROSS SECTION FOR DETERMINING MASS
OF REVETMENT RIPRAP CHECK DAM

Figure 37-3 I

Sideslopes, F = Fore., B= Back.				Ditch Grade, *G*	Check-Dam Spacing
4:1 F; 4:1 B	4:1 F; 3:1 B	4:1 or 3:1 F; 2:1 B	3:1 or 2:1 F; 2:1 B		
X	X	X	X	$\leq 0.5\%$	400 ft
X	X	X	X	$0.5\% < G \leq 0.75\%$	270 ft
X	X	X	X	$0.75\% < G \leq 1\%$	200 ft
X	X	X	X	$1\% < G \leq 1.25\%$	170 ft
X	X	X	X	$1.25\% < G \leq 1.5\%$	135 ft
X	X	X	X	$1.5\% < G \leq 1.75\%$	120 ft
X	X	X	X	$1.75\% < G \leq 2\%$	100 ft
X	X	X	X	$2\% < G \leq 3\%$	70 ft
X	X	X	X	$3\% < G \leq 4\%$	50 ft
*	X	X	X	$4\% < G \leq 5\%$	40 ft
*	*	X	X	$5\% < G \leq 6\%$	35 ft
*	*	*	X	$6\% < G \leq 7\%$	30 ft
*	*	*	*	$> 7\%$	Establish Permanent Erosion Control Measure

** A permanent erosion-control measure should be specified. Permanent erosion-control measures are to be established immediately. These include turf-reinforcement material, a channel liner, or riprap. Temporary measures are to be specified in order to protect permanent measures until construction is complete in that area.*

SPACING FOR CHECK DAM OF 2.0-ft HEIGHT AT SPILLOVER

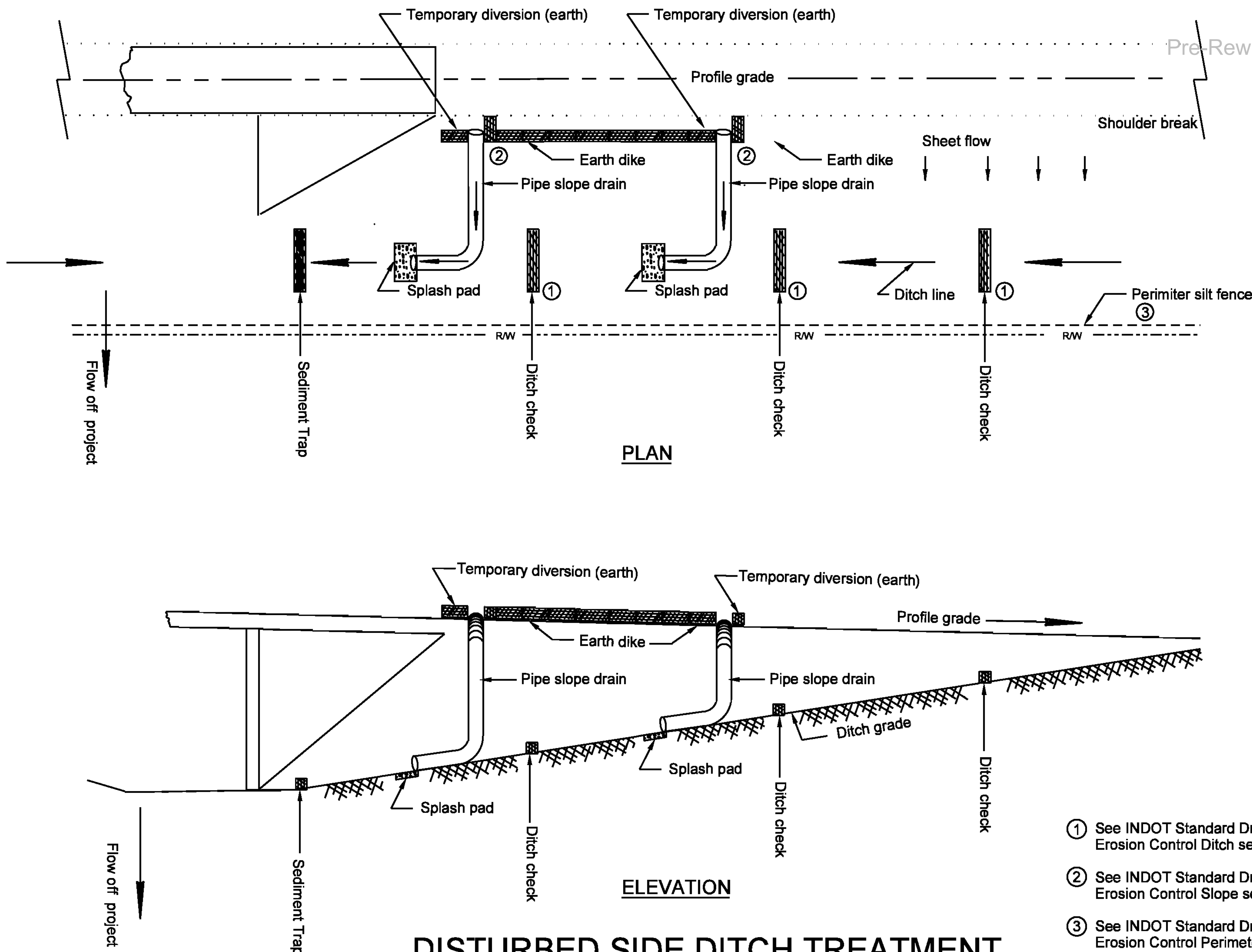
Figure 37-3J

Sideslopes, F= Fore., B = Back.				Ditch Grade, *G*	Check-Dam Spacing
4:1 F; 4:1 B	4:1 F; 3:1 B	4:1 or 3:1 F; 2:1 B	3:1 or 2:1 F; 2:1 B		
X	X	X	X	≤ 0.5%	600 ft
X	X	X	X	0.5% < *G* ≤ 0.75%	400 ft
X	X	X	X	0.75% < *G* ≤ 1%	300 ft
X	X	X	X	1% < *G* ≤ 1.25%	250 ft
X	X	X	X	1.25% < *G* ≤ 1.5%	200 ft
X	X	X	X	1.5% < *G* ≤ 1.75%	170 ft
X	X	X	X	1.75% < *G* ≤ 2%	150 ft
X	X	X	X	2% < *G* ≤ 3%	100 ft
X	X	X	X	3% < *G* ≤ 4%	85 ft
*	X	X	X	4% < *G* ≤ 5%	70 ft
*	*	X	X	5% < *G* ≤ 6%	50 ft
*	*	*	X	6% < *G* ≤ 7%	40 ft
*	*	*	*	> 7%	Establish Permanent Erosion Control Measure

** A permanent erosion-control measure should be specified. Permanent erosion-control measures are to be established immediately. These include turf-reinforcement material, a channel liner, or riprap. Temporary measures are to be specified in order to protect permanent measures until construction is complete in that area.*

SPACING FOR CHECK DAM OF 3.0-ft HEIGHT AT SPILLOVER

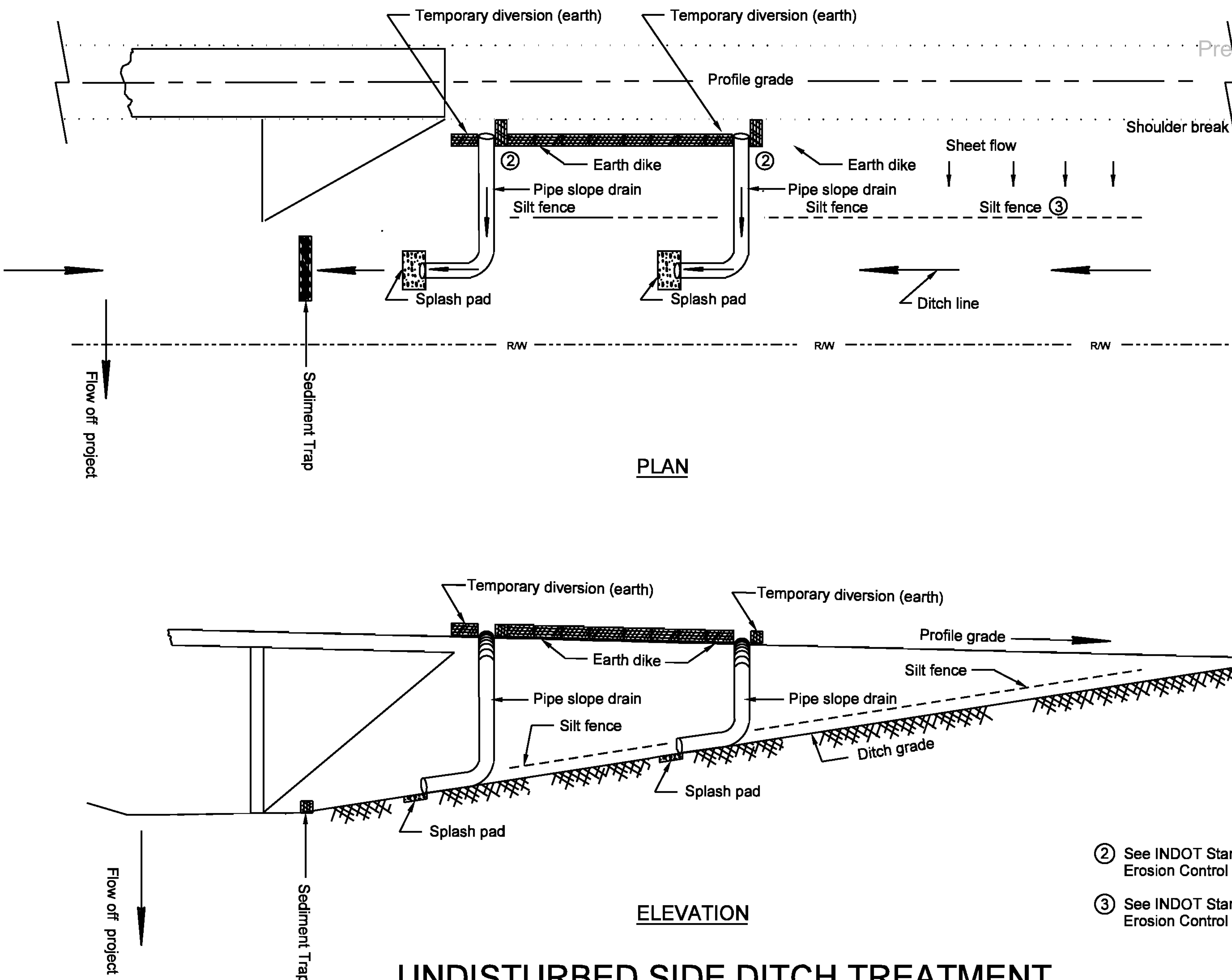
Figure 37-3K



1. See INDOT Standard Drawings Temporary Erosion Control Ditch series.
2. See INDOT Standard Drawings Temporary Erosion Control Slope series.
3. See INDOT Standard Drawings Temporary Erosion Control Perimeter series.

DISTURBED SIDE DITCH TREATMENT

Figure 37-3L



② See INDOT Standard Drawings Temporary Erosion Control Slope series.

③ See INDOT Standard Drawings Temporary Erosion Control Perimeter series.

UNDISTURBED SIDE DITCH TREATMENT

Figure 37-3M

Spillway Height, ft	Sideslope		
	2:1 F; 2:1 B	3:1 F; 2:1 B	3:1 F; 3:1 B
2.0	100	145	190
2.5	150	210	265
3.0	210	285	355
3.5	280	365	450

In Sideslopes columns, F = Foreslope, B = Backslope

RIPRAP VOLUME (ft^3) IN V-DITCH

Figure 37-3N

Spillway Height, ft	Sideslope		
	3:1 F; 3:1 B	4:1 F; 3:1 B	4:1 F; 4:1 B
2.0	310	350	390
2.5	415	465	525
3.0	530	600	670
3.5	665	750	840

In Sideslopes columns, F = Foreslope, B = Backslope

RIPRAP VOLUME (ft^3) IN 3.0-ft-BOTTOM DITCH

Figure 37-3 O

Spillway Height, ft	Sideslope		
	3:1 F; 3:1 B	4:1 F; 3:1 B	4:1 F; 4:1 B
2.0	350	390	430
2.5	460	515	575
3.0	590	660	730
3.5	735	825	910

In Sideslopes columns, F = Foreslope, B = Backslope

RIPRAP VOLUME (ft^3) IN 4.0-ft-BOTTOM DITCH

Figure 37-3P