U.S. Department
of Transportation
**Federal Highway
Administration**

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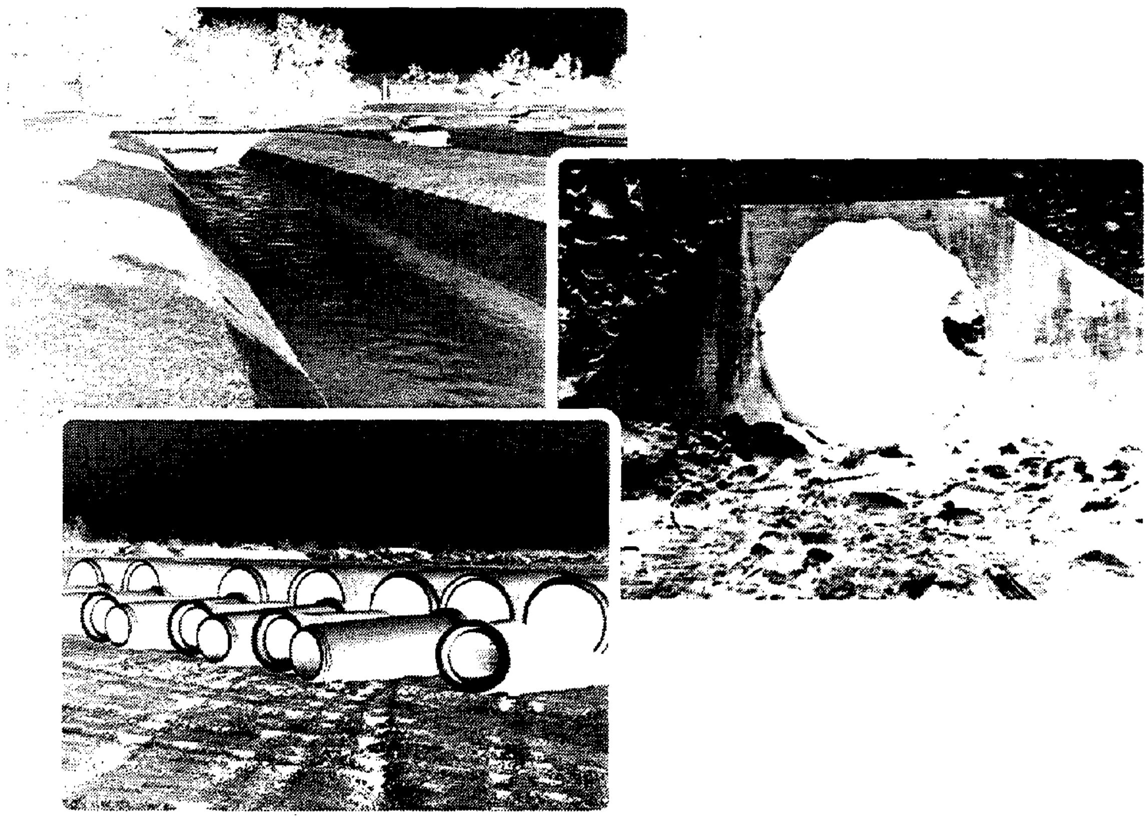
May 1997

NHI Course No. 13065

Introduction to Highway Hydraulics

A Training Course Based on Hydraulic Design Series No. 4

Participant Workbook



National Highway Institute

PB97-176614

INTRODUCTION TO HIGHWAY HYDRAULICS

DETAILED LESSON SCHEDULE

Day	Time	Length (min.)	Lesson	Topic	Method of Instruction	Resources	Reference
				INTRODUCTION			
1	8:00- 8:30	30		Welcome and Introduction	Lecture and Discussion	Slides	HDS-4 Ch. 1,3
	8:30- 9:30	60	1	Definitions and Hydraulic Factors			
	9:30- 9:45	15		Break			
				HYDROLOGY			
	9:45-10:45	60	2	Estimating Storm Runoff	Lecture	Slides	HDS-4
	10:45-11:00	15		Break			
				FUNDAMENTAL HYDRAULIC CONCEPTS			
	11:00-12:00	60	3	Continuity, Energy, and Momentum	Lecture	Slides	HDS-4 Ch. 3
	12:00- 1:30	90	3	Demonstration (Continuity and Energy) Group A (12:00-12:30) Group B (1:00-1:30)	Workshop	Demonstration Flume	
	1:30- 2:30	60	3	Hydrostatics, Orifice, and Weir Flow	Lecture	Slides & Video	
	2:30- 3:30	60	3	Demonstration (Weir and Orifice Flow) Group A (2:30-3:00) Group B (3:00-3:30)	Workshop	Demonstration Flume	
	3:30- 4:30	60		Hydrology Workshop Discussion			

INTRODUCTION TO HIGHWAY HYDRAULICS

DETAILED LESSON SCHEDULE

Day	Time	Length (min.)	Lesson	Topic	Method of Instruction	Resources	Reference
				OPEN-CHANNEL FLOW FUNDAMENTALS			
2	8:00- 9:15	75		Steady Uniform Flow	Lecture	Slides	HDS-4 Ch. 4
	9:15- 10:45	90	4	Concurrent Demonstration (Manning's Equation, Froude No.) and Problem Session	Workshop	Demonstration Flume	
	10:45-12:00	75	4	Steady Non-uniform Flow	Lecture	Slides & Video	
	12:00- 2:00	120	4	Demonstration (Specific Energy, Specific Discharge, Gradually Varied Flow, Rapidly Varied Flow) Group A (12:00-1:00) Group B (1:00-2:00)	Workshop	Demonstration Flume	
				OPEN-CHANNEL FLOW APPLICATIONS			
	2:00-3:00	60	5	Design of Stable Channels	Lecture & Workshop	Slides	HDS-4 Ch. 5
	3:00-3:15	15		Break			
	3:15-4:15	60	5	Pavement Drainage Design	Lecture & Workshop		
	4:15-4:45	30	5	Home Study Problem			

INTRODUCTION TO HIGHWAY HYDRAULICS

DETAILED LESSON SCHEDULE

Day	Time	Length (min.)	Lesson	Topic	Method of Instruction	Resources	Reference
				HOME STUDY PROBLEM			
3	8:00-9:00	60		Discuss Home Study Problem	Workshop		
				CLOSED-CONDUIT FLOW FUNDAMENTALS			
	9:00-9:45	45	6	Fundamentals of Closed-Conduit Flow	Lecture	Slides	HDS-4 Ch. 6
	9:45-10:00	15		Break			
				CLOSED-CONDUIT FLOW APPLICATIONS			
	10:00-11:30	90	7	Culvert Design Procedures	Lecture	Slides	HDS-4 Ch. 7
	11:30-12:00	30	7	Storm Drain Design Procedures	Lecture	Slides	HDS-4 Ch. 7
	1:00- 3:30	150	7	Concurrent Demonstration and Problem Session (Culvert and Storm Drain)	Workshop	Demonstration Flume	
				ENERGY DISSIPATORS			
	3:30-4:30	60	8	Concepts & Design of Energy Dissipators	Lecture	Slides, Video	HDS-4 Ch. 8

INTRODUCTION TO HIGHWAY HYDRAULICS

DETAILED LESSON SCHEDULE

Day	Time	Length (min.)	Lesson	Topic	Method of Instruction	Resources	Reference
				COMPREHENSIVE DESIGN PROBLEM			
4	8:00-8:30	30	9	Introduction to Comprehensive Problem	Workshop		
	8:30-11:30	180	9	Comprehensive Highway Drainage Design Problem	Workshop		
				REVIEW AND CRITIQUE			
	11:30-12:00	30		Critique			

FHWA HYDRAULICS PUBLICATIONS

The **PUBLICATIONS** listed below are available from NTIS, National Technical Information Service, 5285 Port Royal Rd, Springfield, VA 22161, (703) 487-4650. **NO COPIES ARE AVAILABLE FROM FHWA**

	HYDRAULIC DESIGN SERIES (HDS)	YEAR	FHWA-#	NTIS-#
HDS 1	Hydraulics of Bridge Waterways	1978	EPD-86-101	PB86-181708
HDS 2	Highway Hydrology	1996	SA-96-067	PB97-134290
HDS 3	Design Charts for Open-Channel Flow	1961	EPD-86-102	PB86-179249
HDS 4	Introduction to Highway Hydraulics	1997		
HDS 5	Hydraulic Design of Highway Culverts	1985	IP-85-15	PB86-196961

	HYDRAULIC ENGINEERING CIRCULARS (HEC)	YEAR	FHWA-#	NTIS-#
HEC 1	Selected Bibliography of Hydraulic & Hydrologic Subjects		1983	EPD-86-104
	PB86-179256			
HEC 2	3 and 4 are superseded by HEC19			
HEC 5	Hydraulic Charts for... Culverts, superseded by HDS5	1965	EPD-86-105	PB86-181138
HEC 6	Superseded by HDS 4 - HEC 7&8 by HY8 and HDS 5			
HEC 9	Debris-Control Structures	1971	EPD-86-106	PB86-179801
HEC 10	Capacity Charts for the Hydr. Des. of Highway Culverts	1972	EPD-86-107	PB86-185691
HEC 11	Design of Riprap Revetment	1989	IP-89-016	PB89-218424
HEC 12	Drainage of Highway Pavements	1984	TS-84-202	PB84-215003
HEC 13	Hydr. Des. of Improved Inlets for Culverts (see HDS 5)	1972	EPD-86-109	PB86-179710
HEC 14	Hydr. Des. of Energy Dissipators for Culverts & Channels		1983	EPD-86-110
	PB86-180205			
HEC 15	Des. of Roadside Channels w. Flexible Linings	1988	IP-87-7	PB89-122584
HEC 16	Superseded by HIRE (see below)			
HEC 17	Des. of Encroachments on Flood Plains using Risk Analysis 1981			EPD-86-112
	PB86-182110			
HEC 18	Evaluating Scour at Bridges, Edition 2 (English)	1993	IP-90-017	PB93-186138
HEC 18	Evaluating Scour at Bridges, Edition 3 (SI)	1995	HI-96-031	PB96-163498
HEC 19	Hydrology	1984	IP-84-15	PB85-182954
HEC 20	Stream Stability at Highway Structures (English)	1991	IP-90-014	PB91-198788
HEC 20	Stream Stability at Highway Structures, Edition 2 (SI)	1995	HI-96-032	PB96-163480
HEC 21	Bridge Deck Drainage Systems	1993	SA-92-010	PB94-109584
HEC 22	Urban Drainage Design Manual	1996	SA-96-078	

	IMPLEMENTATION REPORTS (IMP)	YEAR	FHWA-#	NTIS-#
HIRE	Highways in the River Environment	1990	HI-90-016	PB90-252479
IMP	Design of Urban Highway Drainage	1979	TS-79-225	PB83-259903
IMP	Underground Disposal of Storm Water Runoff, Design Guidelines Manual	1980	TS-80-218	PB83-180257
IMP	Manual for Highway Storm Volume 1	1982	IP-82-17, V1	PB84-152727
	Water Pumping Stations Volume 2 (Apdx)	1982	IP-82-17, V2	PB84-152785
IMP	Guide for Selecting Manning's Roughness Coef. for Nat. Channels & Flood Plains	1984	TS-84-204	PB84-242585
IMP	Culvert Inspection Manual	1986	IP-86-2	PB87-151809
IMP	Use of Riprap for Bank Protection (Lit Rev)	1986	TS-86-211	PB86-217197
IMP	Structural Design Manual	1983	IP-83-6	PB84-153485

INTRODUCTION TO HIGHWAY HYDRAULICS

A Training Course Based on FHWA Hydraulic Design Series No. 4

WELCOME AND INTRODUCTIONS

I. Host Welcome

II. Introduction of Instructors

III. Introduction of Participants

LESSON 1

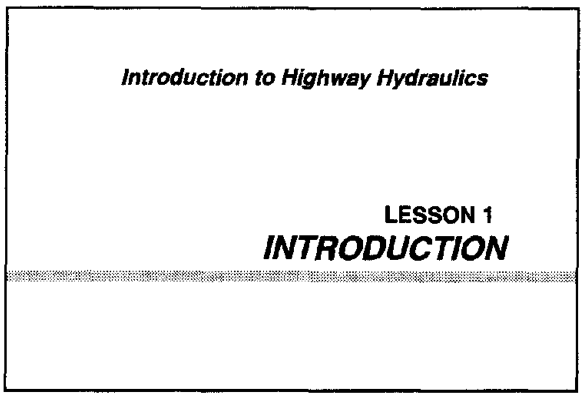
INTRODUCTION

OVERVIEW: Method of Instruction: Lecture

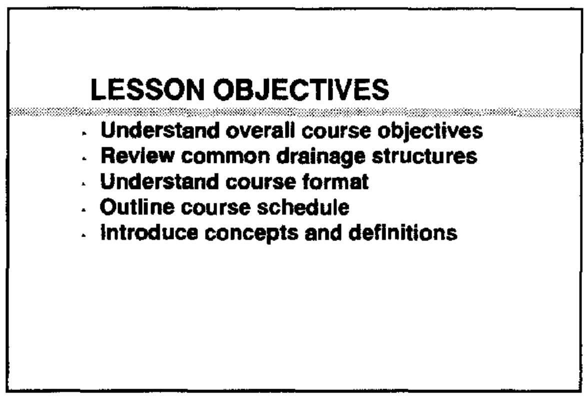
Lesson Length: 60 minutes

Resources:

Lesson Outline
HDS-4, Chapters 1 and 3
Slides
Course Schedule



1.0 Title



1.1 Lesson 1, Part 1

OBJECTIVES: The objectives of this lesson are to:

1. Understand overall course objectives.

2. Review common drainage structures.

3. Understand course format.

4. Outline course schedule.

5. Introduce concepts and definitions.

LESSON 1

INTRODUCTION

Part 1: Course Objectives and Organization

Introduction to Highway Hydraulics

LESSON 1, Part 1

COURSE OBJECTIVES AND ORGANIZATION

1.2 Lesson 1, Part 1

BACKGROUND

- **Highway hydraulic structures convey, divert and remove water from right-of-way**
- **Drainage design covers many disciplines, including**
  - **hydrology**
  - **hydraulics**

1.3 Background

I. Background

A. Highway hydraulic structures perform the vital function of conveying, diverting, or removing surface water from the highway right-of-way.

1. They should be designed to be commensurate with risk, construction cost, importance of the road, economy of maintenance, and legal requirements.

2. One type of drainage facility will rarely provide the most satisfactory drainage for all sections of a highway. Therefore, the designer should know and understand how different drainage facilities can be integrated to provide complete drainage control.

B. Drainage design covers many disciplines, of which two are hydrology and hydraulics.

1. The determination of the quantity and frequency of runoff, surface and groundwater, is a hydrologic problem.

2. The design of structures with the proper capacity to divert water from the roadway, remove water from the roadway, and pass collected water under the roadway is a hydraulic problem.

II. Course Objectives

COURSE OBJECTIVES

- **Understand basic hydraulic concepts**
- **Apply these concepts to highway design and maintenance**

1.4 Course objectives

OTHER NHI COURSES

Course Number	*Course Name*
13010	Highways in the River Environment
13027	Urban Drainage Design
13035	Bridge Backwater Computer Program
13046	Stream Stability and Scour at Bridges
13047	Stream Stability and Scour for Inspectors
13056	Culvert Design
13057	HYDRAIN
13065	Introduction to Highway Hydraulics
13067	Practical Highway Hydrology

1.5 Other NHI courses

A. At the end of this course, the Participant should understand basic hydraulic concepts and principles in open channels and closed conduits. Highway drainage facilities can be broadly classified into two major categories based on construction: (1) open-channel or (2) closed-conduit facilities.

1. Open-channel facilities include roadway channels, median swales, curb and gutter flow, etc.

2. Closed-conduit facilities include culverts and storm drain systems. Note that from a hydraulic classification of flow condition, open-channel or free surface flow can occur in closed-conduit facilities.

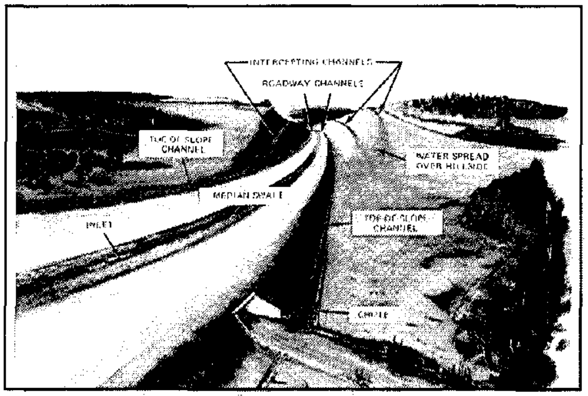
B. At the end of this course, the Participant will be able to apply these concepts to highway design and maintenance such as surface pavement drainage, flow in storm drains, operation of culverts and flow in open channels.

C. Other training courses available through the National Highway Institute provide detailed instruction on specific topics.

COMMON STRUCTURES

- **Lined and natural channels**
- **Bends**
- **Pavement drainage**
- **Storm drains**
- **Inlets**
- **Culverts**
- **Energy dissipators**

1.6 Common structures



1.7 Highway drainage overview

III. Common Highway Drainage Structures

A. Common highway drainage structures include lined and natural channels, storm drains, culverts, energy dissipators.

B. Slide 1.7 illustrates some of the typical drainage facilities that might be used on a highway project.

1. At the outer edge of the right-of-way, channels on the natural ground intercept off-site flows outside the cut-and-fill or on benches breaking the cut slope.

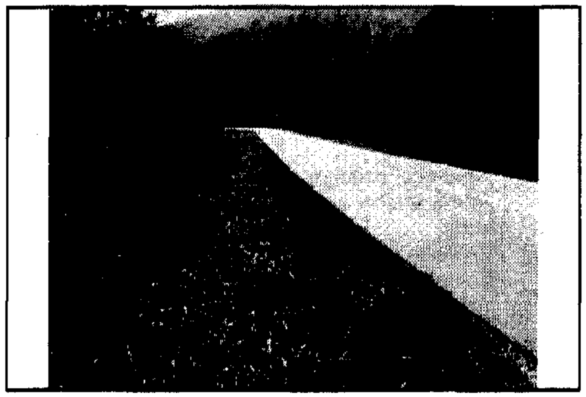
2. Roadway channels between the cut slope and shoulder of the road and the toe-of-slope channels collect flow from the roadway channels and convey it along or near the edge of the roadway embankment to a point of disposal.

3. A shallow depression or swale drains the median to an inlet that conveys water to the culvert. Where channels are steep and flow velocities are high, lining may be necessary.

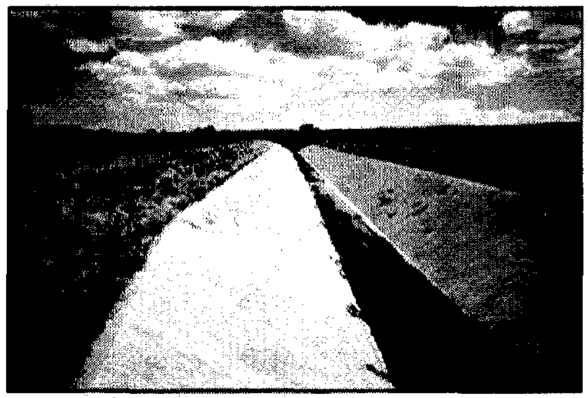
4. The culvert provides for cross drainage of a relatively large stream channel.

C. The following slides illustrate common highway drainage structures.


1.8a Unlined channel

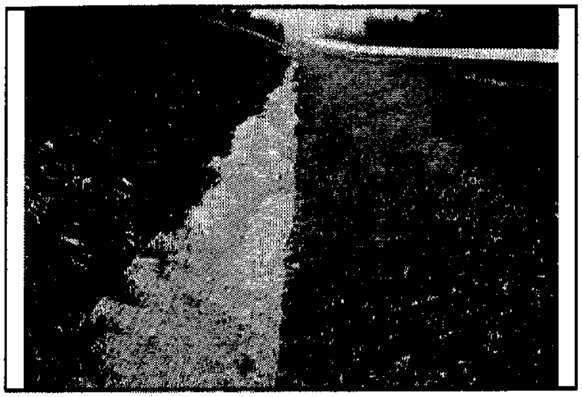

1.8b Grass-lined channel


1.8c Concrete-lined channel


1.8d Concrete-lined channel


1.8e Commercial block lining

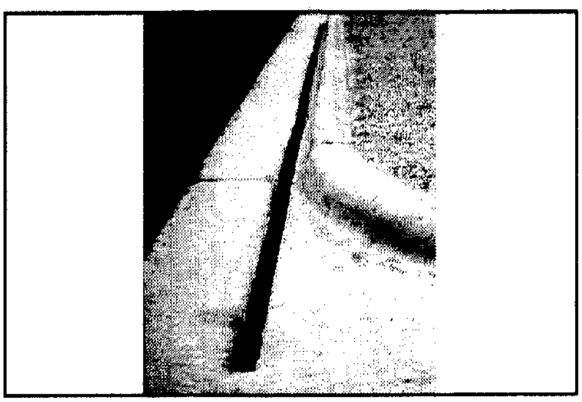

1.8f Soil cement bank lining


1.8g Jute lining

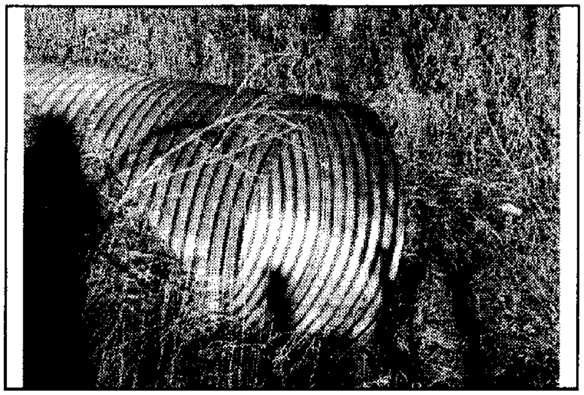

1.8h Rock riprap channel

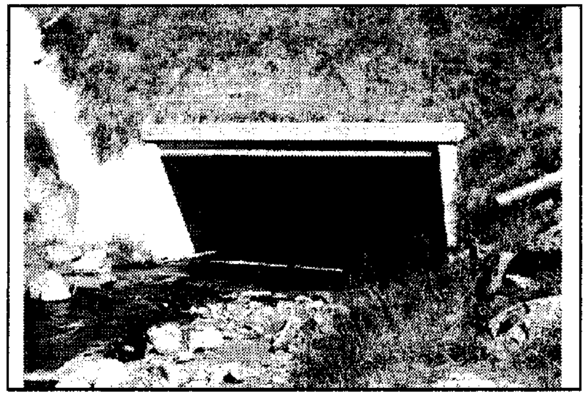

1.8i Grate inlet

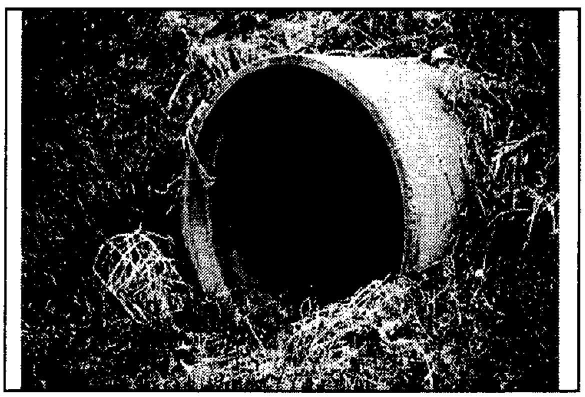

1.8j Median Inlet

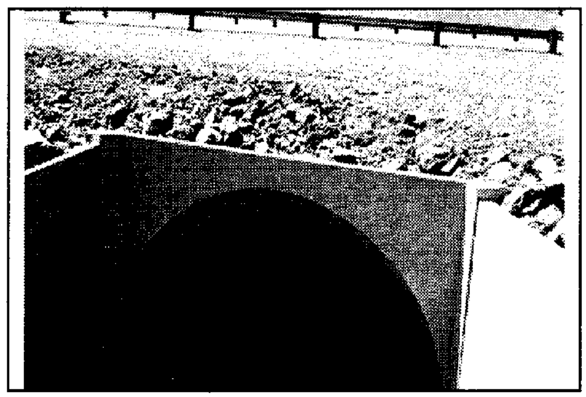

1.8k Slotted inlet

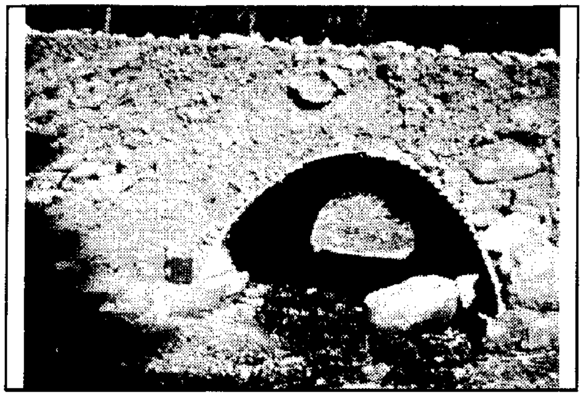

1.8l Storm drain pipe


1.8m Corrugated metal pipe (CMP) culvert


1.8n Reinforced concrete box (RCB) culvert

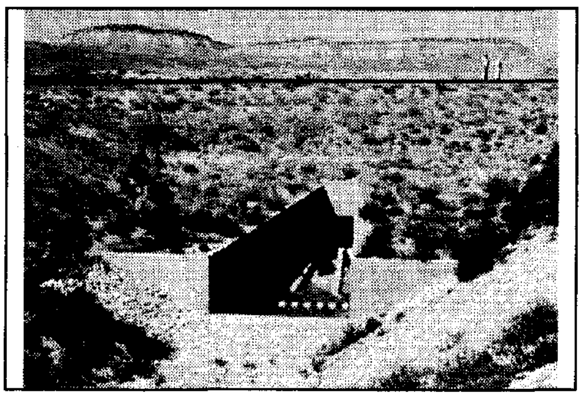

1.8o Reinforced concrete pipe (RCP) culvert


1.8p Beveled inlet RCP culvert



1.8q Steel arch culvert

1.8r U.S.Bureau of Reclamation (USBR) impact basin energy dissipator



1.8s St. Anthony Falls (SAF) energy dissipator

1.8t Rock-riprap drop structure

IV. Important Topics not Covered

TOPICS NOT COVERED

- **Computer model demonstrations**
- **Risk and economics**
- **Construction versus maintenance costs**
- **Legal requirements**
- **Environmental considerations**

1.9 Topics not covered

A. Although some or all of these items may be a major aspect of any highway project, they are considered beyond the scope of this introductory course.

1. Computer models are valuable tools in any hydraulic design.

   a. Hydrologic models

   b. Open-channel flow hydraulics

   c. Culvert analysis

2. Risk analysis is used to balance construction costs with future maintenance and property damage costs. Construction, maintenance and economic issues are discussed in HDS-4.

3. Legal requirements includes right-of-way considerations and permitting.

4. Environmental considerations also include permitting, effects on water quality, and meeting regional and federal regulations.

V. Course Features

COURSE FEATURES

- **Lectures and videos**
- **Group participation**
- **Problem sessions**
- **Demonstrations**
- **Homework problems**
- **Metric system (SI)**

1.10 Course features

A. This course largely consists of lectures using slides and videos.

B. Problem sessions of practical problems and demonstrations of the hydraulic principles are included to aid understanding and to promote group participation.

C. Because much material is presented in only four days, home study problems are included.

D. This course uses the metric system exclusively. The SI metric system is an abbreviation for International System of Units.

1. The metric system of measurement has been in use for hundreds of years. A modern day metric system was established by international agreement in 1960.

2. The SI system has been favorably received throughout the world and provides a standard international language to describe measurement.

3. Additional information is available from the course materials for the Federal Highway Administration, National Highway Institute Course No. 12301, "Metric (SI) Training for Highway Agencies."

VII. Course Schedule

COURSE SCHEDULE

- Day 1
  - Introduction
  - Definitions & hydraulic factors
  - Hydrology
  - Basic Hydraulic Concepts

1.11a Course schedule

COURSE SCHEDULE (cont.)

- Day 2
  - Open channel flow fundamentals
    - steady, uniform flow
    - steady, non-uniform flow
  - Open channel flow Applications
    - design of stable channels
    - pavement drainage design
  - Home study problem

1.11b Course schedule (cont.)

COURSE SCHEDULE (cont.)

- Day 3
  - Closed-conduit flow fundamentals
  - Closed-conduit flow applications
    - culvert design procedures
    - storm drain design procedures
    - culvert demonstration and workshop
    - storm drain demonstration and workshop
  - Energy Dissipators

1.11c Course schedule (cont.)

COURSE SCHEDULE (cont.)

- Day 4
  - Comprehensive design problem
  - Review and critique

1.11d Course schedule (cont.)

LESSON 1

INTRODUCTION

Part 2: Concepts and Definitions

Introduction to Highway Hydraulics

LESSON 1, Part 2

CONCEPTS AND DEFINITIONS

1.12 Lesson 1, Part 2

OBJECTIVES

- **Purpose of highway drainage facilities**
- **Design philosophy**
- **Define terms**

1.13 Objectives

PURPOSE OF HIGHWAY DRAINAGE FACILITIES

- **To convey, divert or remove surface water from highway right-of-way**
  - **drainage facilities for offsite flow**
  - **drainage facilities for onsite flow**

1.14 Purpose of highway drainage facilities

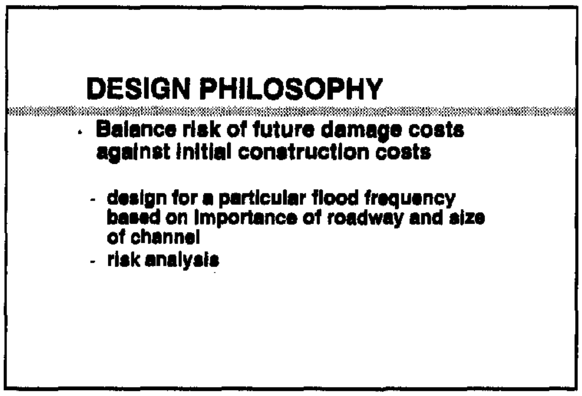
I. Purpose of Hydraulic Structures

A. Highway hydraulic structures perform the vital function of conveying, diverting, or removing surface water from the highway right-of-way.

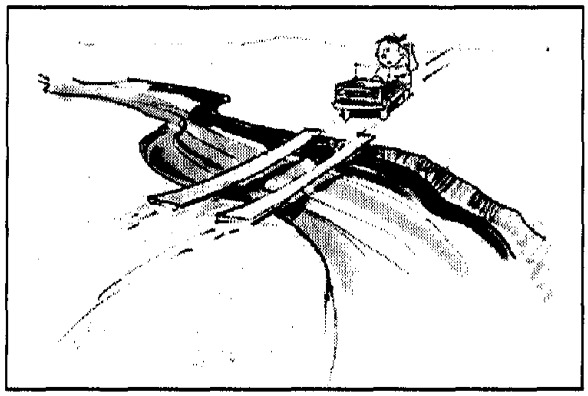
1. Bridges and culverts are designed to convey offsite water across a roadway without causing flow overtopping of the roadway for the design event.

2. Other structures remove onsite water from the roadway or road right-of-way.

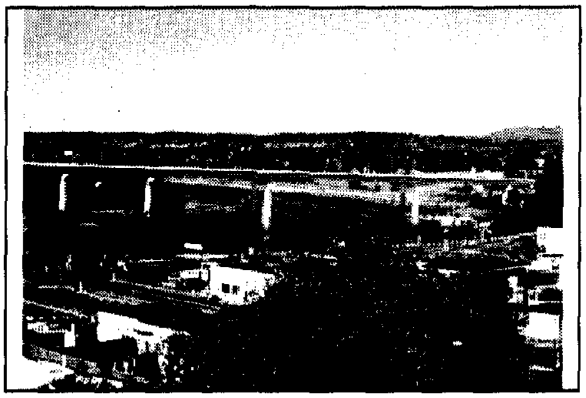
II. Design Philosophy



1.15 Design philosophy



1.16a Low initial cost, high maintenance



1.16b High initial cost, low maintenance

A. The primary purpose of highway drainage facilities is to prevent surface runoff from reaching the roadway, and to efficiently remove rainfall or surface water from the roadway.

1. Designing drainage facilities to accomplish this purpose requires balancing the risk of future damages from runoff events (whose occurrence, in time or magnitude, cannot be forecast accurately) against the initial construction cost. Since this is not easy to accomplish, it is customary that a particular **flood frequency** be selected for each class of highway to establish the design discharge for sizing drainage facilities. This design frequency is then adjusted based on evaluating a check flood to better account for the risk involved considering traffic conditions, structure size, and value of adjacent property.

RANGE OF FLOODS

- **Base flood**
- **Super flood (check flood)**
- **Overtopping flood**
- **Maximum historical flood**
- **Probable maximum flood**
- **Design flood**

1.17a Range of floods

CONVENTIONAL DESIGN FREQUENCIES

Classification	***Design Frequency***
RURAL	
Principal Arterial System	50-year
Minor Arterial System	25- to 50-year
Collector System, major	25-year
Collector System, minor	10-year
URBAN	
Principal Arterial System	25- to 50-year
Minor Arterial System	25-year
Urban Collector System	10-year

1.17b Conventional design frequencies

a. For costly or high risk facilities, a range of discharges with a range of flood frequencies are used to evaluate drainage facilities. The range of floods considered usually includes:

- **Base flood**: The base flood is defined as the flood (storm or tide) having a one percent chance of being equalled or exceeded in any given year (100-year flood).

- **Super flood**: A super flood is significantly greater than the base flood. For example, a 0.2 percent discharge (a 500-year flood) is one possible super flood.

- **Overtopping flood**: Flow at which the roadway is at incipient overtopping conditions.

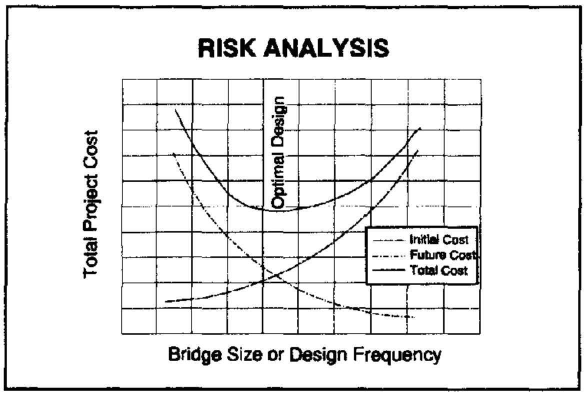
- **Maximum historical flood**: Largest recorded flood event for the reach.

- **Probable maximum flood**: Largest possible flood based on the most extreme weather conditions conceivable.

- **Design flood**: Flow used for design purposes.

b. A range of floods is also typically assumed to evaluate pavement drainage design.

- Pavement drainage is normally designed for the 10-year flood, except in sag vertical curves where water cannot escape other than through a storm drain. In these locations, a 50-year event is often used for design to prevent ponding to a depth that could drown people if they were to drive into it. The use of a lessor frequency event, such as the 50-year storm, to assess hazards at critical locations where water can pond to appreciable depths is commonly referred to as the check storm or check event. The spread of water on the pavement during a check storm can also be evaluated, with a typical criteria being at least one lane of traffic open during the check event.



1.18 Risk analysis

2. One way to select the design flood frequency is through the concept of economics (**risk analysis**) by establishing the least total expected cost for the structure. This concept considers the capital costs, maintenance costs, and the flood hazard costs that are incurred due to damage by a range of flooding events. The flood frequency that generates the least total expected cost for the life of the project would be the one chosen for the design of the structure.

III. Definitions

DEFINITIONS

- **Metric system (SI)**
- **Hydrology**
- **Hydraulics**
- **Velocity**
- **Discharge**
- **Uniform flow**
- **etc.**

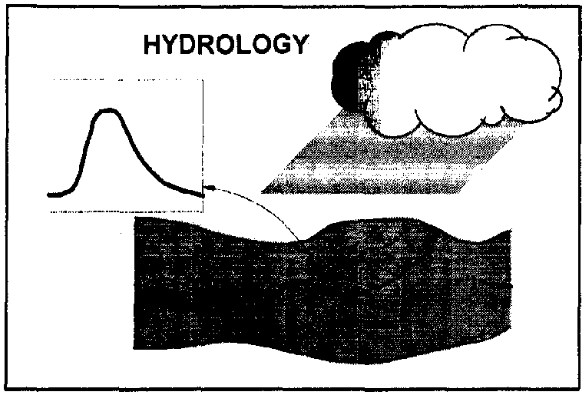
1.19 Definitions

OVERVIEW OF SI

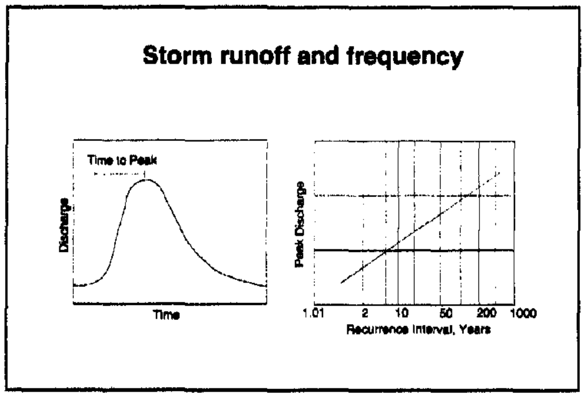
BASE UNITS	UNITS	SYMBOL	CONVERSIONS
length	meter	m	25.4 mm = 1 inch
mass	kilogram	kg	0.4536 kg = 1 lb
time	second	s	
DERIVED UNITS			
area	square meter hectare	m^2 ha	4047 m^2 = 1 acre 1 ha = 10,000 m^2
volume	cubic meter liter	m^3 L	0.02832 m = 1 f^3 1 L = 1000 cm^3
force	newton	N	4.448 N = 1 lb
pressure	pascal	Pa	47.88 Pa = 1 psf

1.20 Metric system overview

A. Metric System: The metric system of measurement has been in use for hundreds of years. The SI metric system is an abbreviation for International System of Units, a modern day metric system established by international agreement in 1960. The SI system has been favorably received throughout the world and provides a standard international language to describe measurement. The United States is the only remaining industrialized nation to convert to the metric (SI) system.

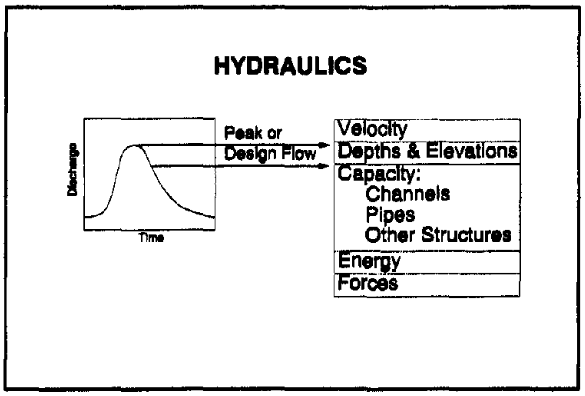


1.21 Hydrology



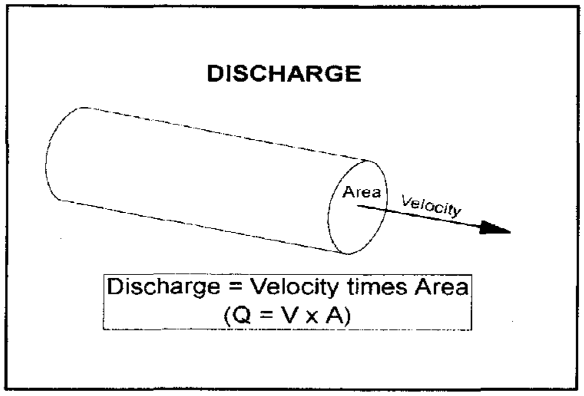
1.22 Storm runoff and frequency

B. Hydrology: Study of the hydrologic cycle, or movement of water due to precipitation, as surface or subsurface flow and evaporation. For this course, hydrology is the determination of quantity and rate of flow based on precipitation and basin conditions or from statistical analyses.



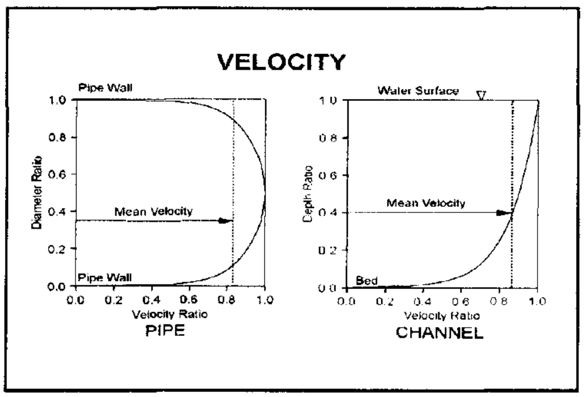
1.23 Hydraulics

C. Hydraulics: Hydraulics is the determination of flow properties (velocity, depth, etc.) and sizing or determining the capacity of structures for a given discharge.



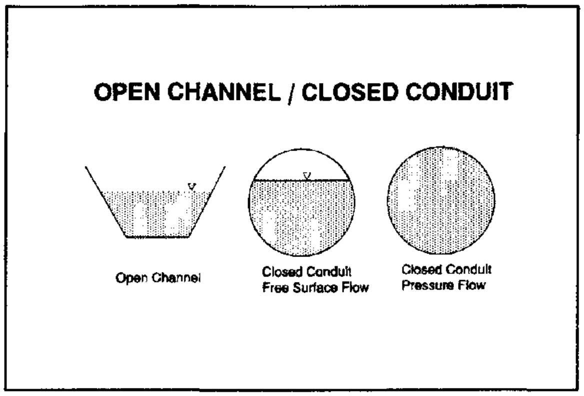
1.24 Discharge

D. Discharge: The quantity of water moving past a given plane (cross section) in a given unit of time. Units are meters cubed per second (m^3/s). The plane or cross section must be perpendicular to the velocity vector.



1.25 Velocity

E. Velocity: The time rate of movement of a water particle from one point to another. The units are meters per second (m/s). Velocity is a vector quantity, that is it has magnitude and direction.

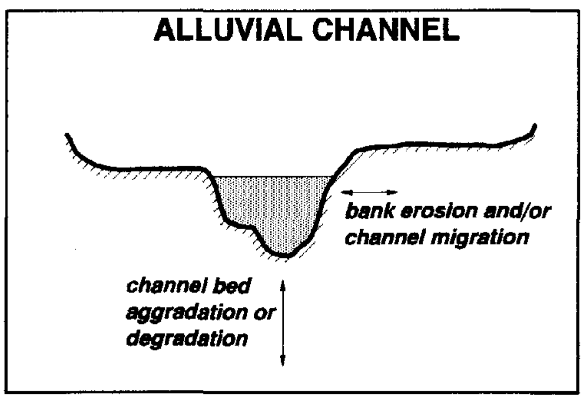


1.26 Open-channel and closed-conduit flow

F. Open-channel flow: Open-channel flow is flow with a free surface. Closed-conduit flow or flow in culverts is open-channel flow if they are not flowing full and there is a free surface.

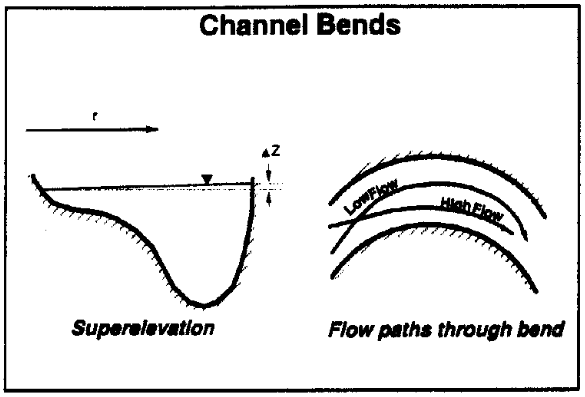
G. Closed-conduit flow: Flow in a pipe, culvert, etc. where there is a solid boundary on all four sides. Examples are pipes, culverts, and box culverts.

H. Pressure flow: Flow in a closed conduit or culvert that is flowing full with water in contact with the total enclosed boundary.



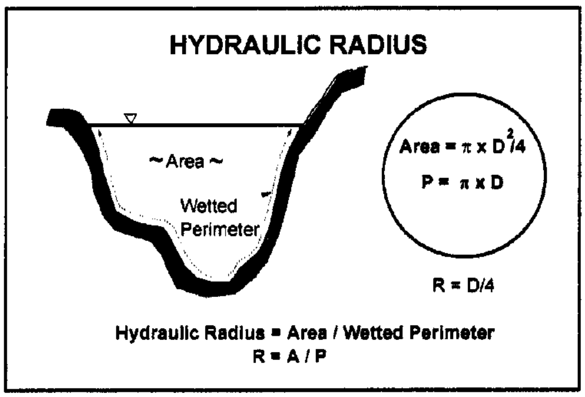
1.27 Alluvial channel

I. Alluvial channel: Flow in an open channel where the bed is composed of bed material that has been deposited by the flow.



1.28 Channel bends

J. Channel bends: The deflection of flow around a bend causes superelevation of the outside water surface and can be extreme for high velocity flow conditions. The flow path can be different for low and high flow conditions.



1.29 Hydraulic radius

K. Hydraulic radius: The hydraulic radius is a length term used in many of the hydraulic equations that is determined by dividing the flow area by the length of the cross section in contact with the water (wetted perimeter). The hydraulic radius is in many of the equations to help take into account the effects of the shape of the cross section on the flow. Note that the hydraulic radius for a circular pipe flowing full is equal to the diameter divided by four (D/4), and for wide, shallow channels the hydraulic radius is approximately equal to the flow depth.

LESSON 2

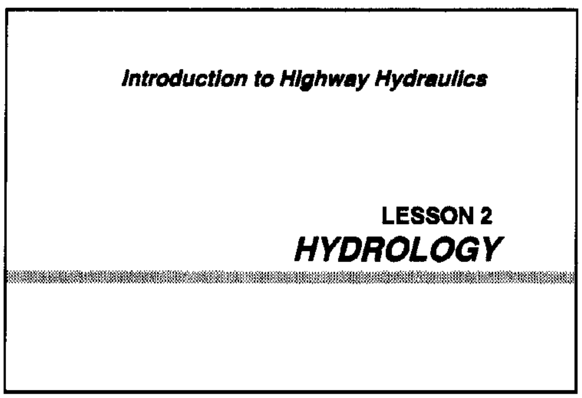
HYDROLOGY

OVERVIEW: Method of Instruction: Lecture

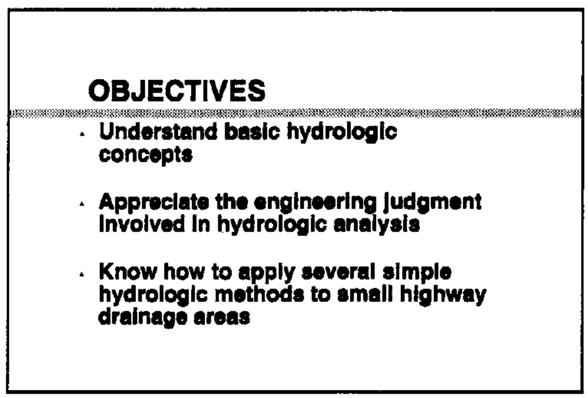
Lesson Length: Lecture - 60 minutes
Workshop - 30 minutes

Resources:

Lesson Outline
Slides
HDS-4



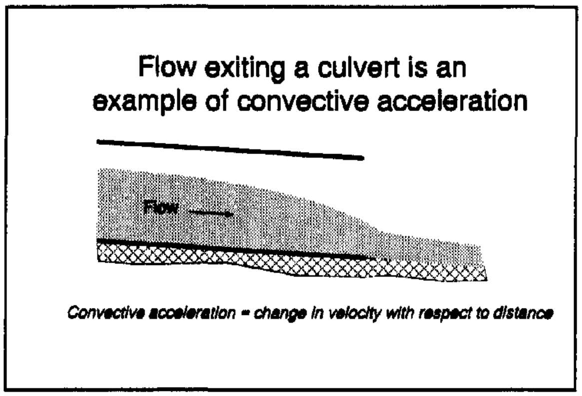
2.0 Title



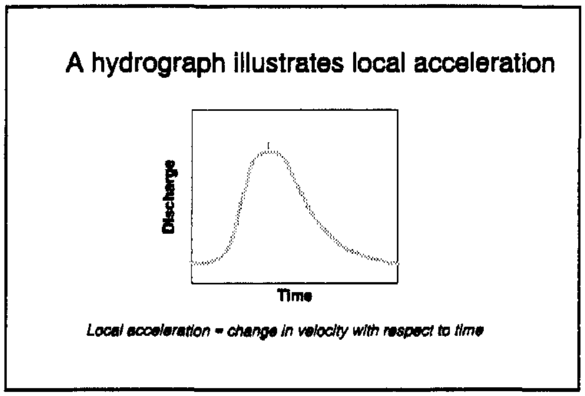
2.1 Objectives

OBJECTIVES: At the conclusion of this lesson, the Participant should:

1. Understand basic hydrologic concepts.
2. Appreciate the engineering judgment involved in hydrologic analysis.
3. Know how to apply several simple hydrologic methods applicable to small highway drainage areas.

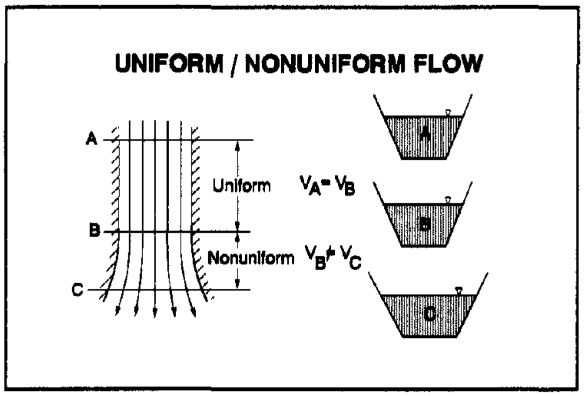


1.30 Culvert outlet illustrates convective acceleration



1.31 Hydrograph illustrates local acceleration

L. Acceleration: Acceleration is the time rate of change in magnitude or direction of the velocity vector. Units are meters per second per second (m/s^2). It is a vector quantity. Convective acceleration is the change in velocity with respect to distance only; local acceleration is the change in velocity with respect to time only.

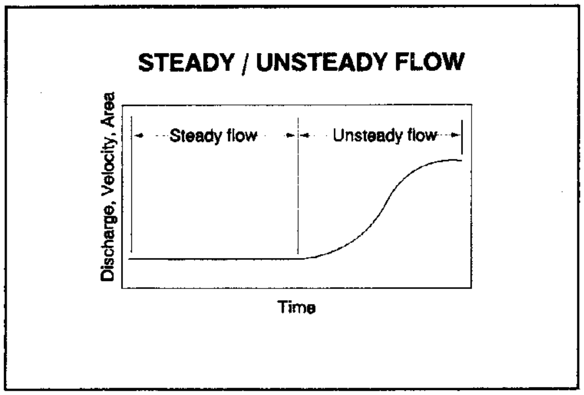


1.32 Uniform and nonuniform flow

M. Uniform flow: In uniform flow the velocity and depth of the flow does not change with distance. Examples are flow in a straight pipe of uniform cross section flowing full or flow in a straight open channel with constant slope and all cross sections of identical form, roughness and area, resulting in a constant mean velocity.

Uniform flow conditions are rarely attained in open channels, but the error in assuming uniform flow in a channel of fairly constant slope and cross section is small in comparison to the error in determining the design discharge.

N. Nonuniform flow: In nonuniform flow the velocity of flow changes in magnitude or direction or both with distance. Examples are flow around a bend or flow in expansions or contractions.



1.33 Steady and unsteady flow

O. Steady flow: In steady flow, the velocity at a point or cross section does not change with time. The local acceleration is zero.

P. Unsteady flow: In unsteady flow, the velocity at a point or cross section varies with time. The local acceleration is not zero. A flood hydrograph where the discharge in a stream changes with time is an example of unsteady flow. Unsteady flow is difficult to analyze unless the time changes are small.

FROUDE NUMBER

$$Fr = \frac{V}{\sqrt{g\,y}}$$

RATIO OF
INERTIAL FORCE
TO
GRAVITATIONAL FORCE

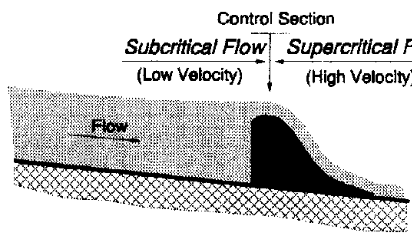
1.34 Froude Number

Q. Froude Number: The Froude Number ($Fr = \frac{V}{\sqrt{gy}}$) is the ratio of inertial forces to gravitational forces. The Froude Number is also the ratio of the flow velocity V to the celerity, which is the rate that a small gravity wave progresses in the flow ($c = \sqrt{gy}$).

R. Subcritical flow: Open-channel flow's response to changes in channel geometry depends upon the depth and velocity of the flow. Subcritical flow (or tranquil flow) occurs on mild slopes where the flow is deep with a low velocity and has a Froude Number less than 1. In subcritical flow, the boundary condition (control section) is always at the downstream end of the flow reach.

S. Supercritical flow: Supercritical flow occurs on steep slopes where the flow is shallow with a high velocity and has a Froude Number greater than 1. In supercritical flow, the boundary condition (control section) is always at the upstream end of the flow reach.

T. Critical flow: When the Froude Number equals 1, the flow is critical and surface disturbances remain stationary in the flow.



Flow over a spillway illustrates subcritical and supercritical flow

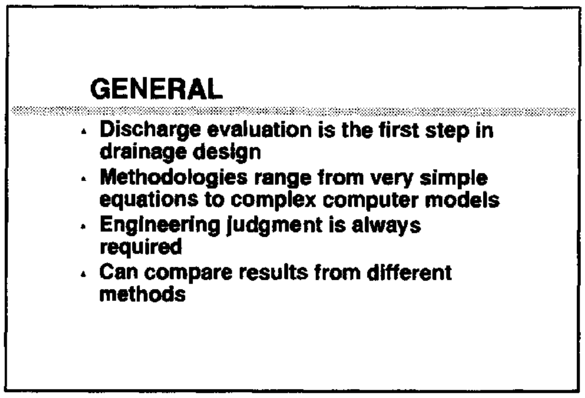
1.35 Water surface profiles

U. Water surface profiles: Water surface profile computations require identification of the hydraulic control section and whether the channel slope is steep or mild. Channel hydraulic calculations often commence from a location where the water surface can be uniquely defined for a given discharge. Computation progress upstream from the control section for mild slopes and downstream from the control section for steep slopes. Steep and mild slopes are defined by comparing the normal depth to the critical depth. These depths will be defined later in the course.

NOTES:

LESSON 2

HYDROLOGY



2.2 General

I. General

A. The first step in designing a drainage facility is to determine the quantity of water the facility must carry. This can be a major component of the overall design effort.

B. The level of effort required depends on the data available and the sophistication of the analysis technique. Methodologies range from very simple equations to complex computer models.

C. Regardless of the methodology, hydrologic analysis always involves engineering judgment; unlike many other aspects of engineering design, the quantification of runoff is not an exact science.

D. It is generally a good idea to compare results from several appropriate methods.

WHAT IS RUNOFF?
PRECIPITATION - LOSSES = RUNOFF

2.3a What is runoff?

WHAT ARE LOSSES?
· Evaporation and interception losses · Infiltration and depression losses · Rate of losses varies with intensity, temperature and land conditions

2.3b What are losses?

II. What is Runoff?

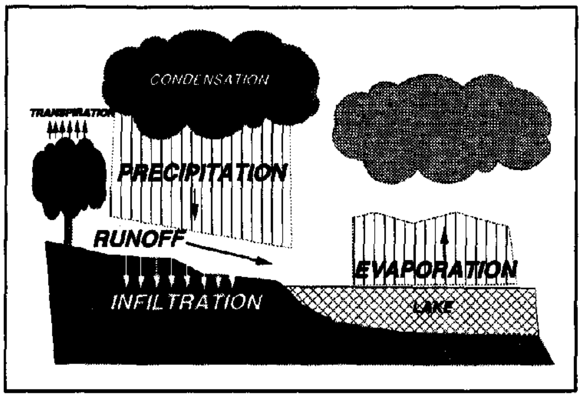
A. Precipitation (rain or snow) falling on land and water surfaces produces watershed runoff. Not all the precipitation that falls becomes runoff; some is lost to other purposes/uses.

B. Part of the precipitation evaporates; part is intercepted by vegetation.

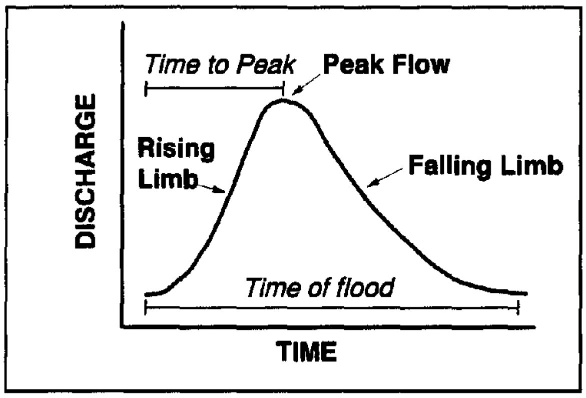
C. Of the precipitation that reaches the ground, a portion infiltrates the ground, a portion fills depressions in the ground surface, and the remainder flows over the surface (overland flow) to reach defined watercourses.

D. The storm runoff that must be carried by highway drainage facilities is then the residual precipitation after losses (extractions for evaporation, interception, infiltration, and depression storage).

E. Rate of losses depends on rainfall intensity, temperature, and characteristics of land surface (including antecedent moisture).



2.4 Hydrologic cycle



2.5 Hydrograph

F. Hydrologic cycle illustrates rainfall/runoff process.

G. Storm runoff is represented by a hydrograph, a plot of the variation of discharge with time. The area under this curve represents the volume of runoff.

INFORMATION NECESSARY FOR DESIGN

- **Hydrograph results useful when storage information necessary**
- **Peak flow results used for design of culverts, storm drains, etc**
- **Peak flow methods will be emphasized in this lesson**

2.6 Information for design

III. What Runoff Information is Necessary for Drainage Design?

A. Knowledge of the complete hydrograph is seldom necessary for small highway drainage facilities. Knowledge of the peak flow during the hydrograph is usually sufficient.

B. For example, design of median drainage facilities, a storm drain and inlets to protect a fill slope, or a culvert draining a small area isolated by roadway fill, can all be designed based on peak flow only.

C. Information in this lesson concentrates on peak flow estimation only.

METHODS FOR PEAK FLOW ESTIMATION

- Sites with measured stream gage data
  - statistical analysis
  - typically larger channels
  - limited application in drainage structure design
- Sites without gaged data
  - empirical equations
  - regression equations
  - both procedures will be discussed

2.7 Peak flow methods

IV. Methods for Peak Flow Estimation

A. Sites with measured stream gage data.

1. Design peak runoff estimated by statistical analysis of the flow record.

2. Measured data usually only available for larger channels.

3. Limited application to small areas that contribute runoff to highway drainage structures, and only brief overview will be provided in this lesson (for detailed information, see HDS-2).

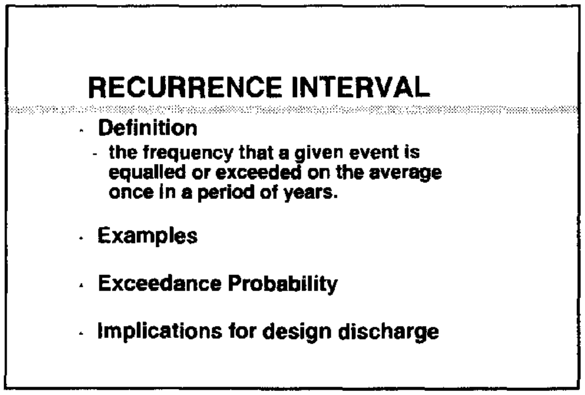
B. Sites without gaged data.

1. Without data, design peak runoff can be estimated by empirical equations (e.g., Rational method) or by regional regression equations.

2. Regional regression equations generally applied to larger areas, and the Rational method to smaller areas (less than 80 hectares).

3. Both procedures will be discussed.

2.8 USGS gaging station



2.9 Recurrence interval

V. Concept of Recurrence Interval

A. The recurrence interval (also called return period) defines the frequency that a given event (rainfall or runoff) is equalled or exceeded on the average once in a period of years.

B. For example, if the 25-year frequency discharge is 100 m^3/s, a runoff event of this size or greater would be expected to occur, on average, once every 25 years.

C. The exceedance probability is the reciprocal of recurrence interval. Therefore, in the example above, a discharge of 100 m^3/s would have a 0.04 probability, or a 4 percent chance of occurrence in any given year.

D. Recurrence interval is important in selecting the design discharge.

1. A channel designed for a 1-year flood would have a low first cost, but the maintenance costs would be high because the channel and roadway could be damaged by storm runoff almost every year.

2. A channel designed for the 100-year flood would be high in first cost, but low in maintenance cost.

ANALYSIS OF GAGED DATA

- **USGS Streamgaging program, and other data sources**
- **Statistical analysis based on assumed standard frequency distribution**
- **Typical distribution is log-Pearson Type III**

2.10 Analysis of gaged data

VI. Analysis of Gaged Data

A. The U.S. Geological Survey (USGS) collects and publishes much of the stream gage data available in the United States. These data are reported in USGS Water Supply Papers, and available by computer access through CD-ROM.

B. Statistical analysis of these data can provide a good estimate of peak discharge. Statistical analysis allows better extrapolation in time of the available stream flow record, by assuming that the data can be fit by some standard frequency distribution.

C. For example, 20-30 years of streamflow data would be a good record; however, a simple ranking of these data would probably not provide good insight on the magnitude of the 100-year flood, since it is possible that the 100-year flood did not occur in that 20-30 year period. A statistical frequency distribution typical for streamflow would allow better extrapolation.

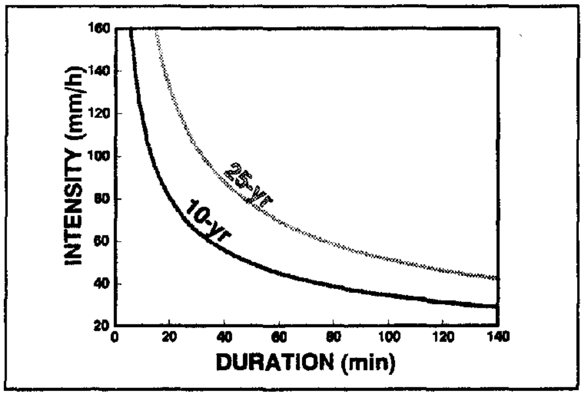
D. The frequency distribution that is most useful in hydrologic data analysis is the log-Pearson Type III distribution.

E. A comprehensive treatment of analysis of gage data, and the application of these frequency distributions is provided in HDS-2.

RAINFALL ANALYSIS

- Rainfall is the driving force in runoff
- Intensity is rate of rainfall (mm/h)
- Recurrence interval depends on duration, since intensity varies with duration
- Relationships usually presented as intensity-duration-frequency (IDF) curves
- IDF information usually available

2.11 Rainfall analysis



2.12 IDF curves

VII. Rainfall Analysis

A. All discharge analysis must start with knowledge of the precipitation, the driving force in generating runoff.

B. Intensity of rainfall is the rate at which rain falls. Intensity is usually stated in mm/h, regardless of the duration, although it can be tabulated as the total rainfall in a given time period (duration).

C. Frequency can be expressed in terms of the average interval (return period or recurrence interval) between rainfall intensities of a given or greater amount. The frequency of rainfall cannot be stated without specifying the duration, since intensity varies with duration.

D. The relationships between intensity, duration, and frequency are typically presented as Intensity-Duration-Frequency (IDF) curves. IDF curves are used in analysis of peak discharge.

E. IDF information can be derived from point rainfall data collected by the National Weather Service, the National Oceanic and Atmospheric Administration, and other agencies. However, in most localities, the necessary rainfall data, including IDF curves are available from city, county, state agencies and it is seldom necessary to begin with raw rainfall data. IDF curves can also be calculated using HYDRAIN.

RATIONAL METHOD
Equation and Assumptions

▸ **Equation**

$$Q = \frac{C\ i\ A}{360}$$

2.13 Rational method

VIII. The Rational Method - Equation and Assumptions

A. One of the most common equations for peak flow estimation is the rational formula:

$$Q = \frac{CiA}{360}$$

where Q = Peak rate of runoff, m^3/s

C = Dimensionless runoff coefficient assumed to be a function of the cover of the watershed

i = Average rainfall intensity, mm/h, for the selected frequency and for duration equal to the time of concentration

A = Drainage area, in hectares, tributary to the point under design

RATIONAL METHOD
Equation and Assumptions

▸ Equation

$$\frac{C\ i\ A}{360}$$

▸ **Fundamental Assumption**

For a uniform rainfall falling on a surface, the maximum rate of runoff occurs when all areas are contributing; the rate of runoff is then constant

2.14 Basic assumption

B. The basic assumption in the rational formula is that if a uniform rainfall of intensity (i) were falling on an area of size (A), the maximum rate of runoff at the outlet to the drainage area would be reached when all portions of the drainage area were contributing; the runoff rate would then become constant.

TIME OF CONCENTRATION

- **Definition**

The time required for runoff from the most hydraulically remote point to arrive at the outlet

2.15 Time of concentration

C. The time required for runoff from the most hydraulically remote point (point from which the time of flow is greatest) of the drainage area to arrive at the outlet is called the time of concentration (t_c).

LIMITATIONS OF RATIONAL METHOD

- **Rainfall intensity seldom constant**
- **Even when fundamental assumption is satisfied, the rate of runoff is not constant due to differences in land conditions and antecedent moisture**
- **These concerns limit application to small areas (less than 80 hectares)**

2.16 Limitations

D. Actual runoff is far more complicated than the rational formula indicates.

1. Rainfall intensity is seldom the same over an area of appreciable size or for any substantial length of time during the same storm.

2. Even if a uniform intensity of rainfall of duration equal to the time of concentration were to occur on all parts of the drainage area, the rate of runoff would vary in different parts of the area because of differences in the characteristics of the land surface and the nonuniformity of antecedent conditions.

E. Under some conditions, maximum rate of runoff occurs before all of the drainage area is contributing due to storage in the channels and large differences in time of concentration between subareas in a larger watershed. The potential errors increase as the size of the drainage area increases.

F. For design of highway drainage structures, the method should be limited to areas less than 80 hectares.

SUMMARY OF ASSUMPTIONS

- **Peak flow occurs when all the watershed is producing**
- **Intensity is uniform over a duration equal to time of concentration**
- **Frequency of rainfall equals frequency of runoff**

2.17 Summary

G. In summary, the assumptions involved in using the Rational method are:

1. The peak flow occurs when all the watershed is contributing.

2. The rainfall intensity is uniform over a time duration equal to the time of concentration, which is the time required for water to travel from the most hydraulically remote point to the outlet or point of interest. Note that the most hydraulically remote point is defined in terms of time, not necessarily distance.

3. The frequency of the computed peak flow is equal to the frequency of the rainfall intensity. In other words, the 10-year rainfall intensity is assumed to produce the 10-year flood.

RATIONAL METHOD
Runoff Coefficient (C)

- **"C" is ratio of rate of runoff to rate of rainfall**
- **C values are tabulated by land use; however, value also varies with intensity, infiltration and other abstractions**
- **Composite C values**

2.18 Runoff coefficient

IX. Rational Method - Runoff Coefficient

A. The runoff coefficient (C) in the rational formula is the ratio of the rate of runoff to the rate of rainfall at an average intensity (i) when all the drainage area is contributing.

B. The runoff coefficient is tabulated as a function of land use conditions; however, the coefficient is also a function of slope, intensity of rainfall, infiltration and other abstractions.

TYPICAL RUNOFF COEFFICIENTS

TYPE OF SURFACE	RUNOFF COEFFICIENT
RURAL AREAS	
PAVEMENT	0.8-0.9
TURF MEADOWS	0.1-0.4
CULTIVATED FIELDS	0.2-0.4
URBAN AREAS	
FLAT RESIDENTIAL (30% IMPERVIOUS)	0.40
STEEP RESIDENTIAL (50% IMPERVIOUS	0.65
FLAT COMMERCIAL (90% IMPERVIOUS)	0.90

2.19 Table of C values

C. HDS-4, table 11, provides a range of C values for various land-use types.

D. Where the drainage area is composed of several land-use types, the runoff coefficient can be weighted according to the area of each type of cover present However, the accuracy of the Rational method is better when the land use is fairly consistent over the entire area. Significantly different land-use conditions can lead to inconsistent estimates of the time of concentration, and hence the intensity, and potential errors in establishing the most appropriate (C).

RATIONAL METHOD
Time of Concentration

- **Total time of concentration is sum of overland flow time, and travel times in gutters, swales and storm drains**
- **Extreme precision not warranted**
- **Methods of calculating each component**

2.20 Time of concentration

X. Rational Method - Time of Concentration

A. The time of concentration varies with the size and shape of the drainage area, the land slope, the type of surface, the intensity of rainfall, and whether flow is overland or channelized.

B. The time of concentration can be considered the sum of an overland flow time and the travel times in gutters, swales, storm drains, etc.

C. Extreme precision is not warranted in determining time of concentration, particularly for small area drainage facility design; however, since the peak discharge (Q) is generally quite sensitive to the time of concentration, care should be taken to ensure an appropriate value is obtained.

RATIONAL METHOD
Time of Concentration

- **Overland flow travel time based on kinematic wave theory**

$$t = K \frac{n^{0.6} \; L^{0.6}}{i^{0.4} \; S^{0.3}}$$

2.21 Overland flow travel time

D. For overland flow the most physically correct approach is based on kinematic wave theory (ASCE, 1992):

$$t = K \frac{n^{0.6} L^{0.6}}{i^{0.4} S^{0.3}}$$

where

t = Minutes

L = Overland flow length, m

n = Manning's roughness coefficient

i = Rainfall rate, mm/h

S = Average slope of the overland flow area

K = 6.99

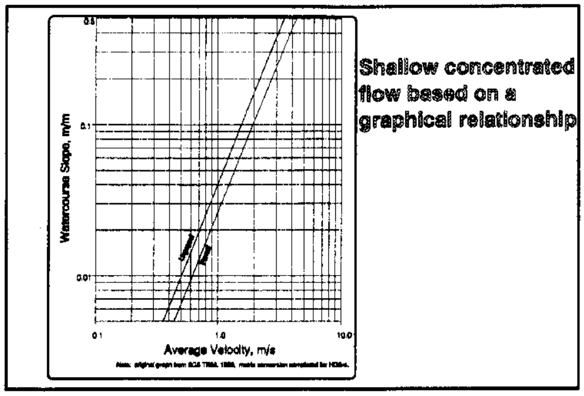
Important factors in applying this equation include:

1. An iterative solution is required since time of concentration and rainfall intensity are unknown.

2. For application to natural watershed conditions, the n value should be quite large (e.g., 0.5) to account for the large relative roughness resulting from water running through vegetation rather than over it as compared to channel flow conditions.

3. For paved conditions an n value in the normally accepted range for smooth surfaces (e.g., 0.012 - 0.016) is appropriate.

4. For applicable Manning's n values, see HDS-2 or Soil Conservation Service TR-55.



2.22 Shallow flow average velocities

E. The velocity in shallow concentrated flow may be estimated from HDS-4, figure 4.

F. The travel time in gutter, storm drain and channel flow is typically estimated from basic hydraulic data (t = distance/velocity).

RATIONAL METHOD
Time of Concentration

- **Gutter, storm drain and channel flow travel time based on**

  t = distance/velocity

2.23 Channel travel time

RATIONAL METHOD
Time of Concentration

- **Change from overland flow to shallow concentrated flow not always obvious**
- **Maximum overland flow length of 130 m is reasonable**
- **Minimum time of concentration of 5 minutes should be assumed**

2.24 Tc guidelines

G. It is not always apparent when flow changes from overland flow to shallow concentrated flow. If rilling or other signs of concentrated flow are not evident in the field, it is reasonable to assume a maximum overland flow length of 130 m.

H. If the computed total time of concentration is less than 5 minutes, a minimum time of concentration of 5 minutes should be used.

XI. Rational Method - Rainfall Intensity

A. Rainfall intensity data for a given return period should be determined as discussed above (Section VII).

B. If rainfall depths are provided, calculate the intensity by dividing the rainfall depth by duration in hours.

RATIONAL METHOD
Rainfall Intensity (I)

- **Determined from appropriate IDF curves**
- **If rainfall depths provided, calculate intensity by dividing depth by hours**

2.25 Rational method intensity

RATIONAL METHOD
Drainage Area (A)

- **Drainage area in hectares, based on topographic map or field survey**

2.26 Rational method area

XII. Rational Method - Drainage Area

A. The drainage area, in hectares, contributing flow to the point in question, can be measured on a topographic map or determined in the field during the preliminary field survey.

B. The data required to determine time of concentration and the runoff coefficient should also be noted at the time of the preliminary field survey.

RATIONAL METHOD
Application to Complex Areas

- **The longest time of concentration should be used**
- **It is possible for the maximum runoff to occur from a single subarea at a time of concentration less than the entire basin.**

2.27 Complex area application

XIII. Rational Method - Application to Complex Drainage Areas

A. Application of the Rational method to any concentration point (e.g., storm drain inlet or culvert) is based on the longest time of travel to the point for which the discharge is to be determined.

B. On some combinations of drainage areas, it is possible that the maximum rate of runoff will be reached from the higher intensity rainfall for periods less than the time of concentration for the whole area, even though only a part of the drainage area is contributing. This might occur where a part of the drainage area is highly impervious and has a short time of concentration, and another part is pervious and has a much longer time of concentration. Unless the areas or times of concentration are considerably out of balance, the accuracy of the method does not warrant checking the peak flow from only a part of the drainage area. This is particularly true for the relatively small drainage areas associated with highway pavement drainage facilities.

REGRESSION METHODS

- **Regional equations used to estimate peak flows at ungaged sites**
- **Equations have been developed for both rural and urban areas**
  - **statewide basis**
  - **hydrophysiographic basis**
- **Studies by FHWA/USGS/others resulted in statewide equations throughout US**

2.28 Regression methods

XIV. Regression Methods

A. Regional regression equations are commonly used for estimating peak flows at ungaged sites or sites with insufficient data.

B. Regional regression equations relate peak flow for a specified return period to the physiographic, hydrologic, and meteorologic characteristics of the watershed.

C. Equations have been developed for both rural and urban areas on either a state-by-state basis, or by definition of hydrophysiographic regions that may cross state boundaries.

D. In a series of studies by the USGS, in cooperation with the Federal Highway Administration (FHWA) and various other agencies, statewide regression equations have been developed throughout the United States.

USGS REGRESSION EQUATIONS

- **Rural Equations**
  - **developed primarily for unregulated, natural or nonurbanized watersheds**
- **Urban Equations**
  - **modify rural discharge**
  - **seven parameters are: drainage area, channel slope, 2-yr rainfall intensity, reservoir/swamp storage, impervious area, rural discharge and BDF**

2.29a USGS regression equations

TYPICAL USGS REGRESSION EQUATIONS

- **Rural:**

$$RQ10 = 3.14\ A^{0.78}\ S^{0.62}$$

- **Urban:**

$$UQ10 = 9.51\ A^{0.16}\ (13 - BDF)^{-0.36}\ RQ10^{0.79}$$

2.29b Typical rural/urban equations

The resulting set of equations, referred to as the USGS rural regression equations, were developed primarily for unregulated, natural, nonurbanized watersheds. A discussion of the accuracy of the equations and limitations in their application is provided in HDS-2.

BASIN DEVELOPMENT FACTOR

- **BDF is primary factor describing urbanization**
- **BDF based on extent of:**
  - **channel improvements**
  - **channel lining**
  - **storm drains**
  - **curb and gutter streets**

2.30 BDF

E. To estimate peak discharge in urban areas, equations were developed that modify the rural peak discharge. The primary factor describing urbanization is the basin development factor (BDF),which is based on a combination of several manmade changes to the drainage basin including channel improvements, channel linings, storm drains and curb and gutter streets. A complete discussion of this method is provided in USGS Water Supply Paper 2207 (Sauer et al., 1983) and HDS-2, Highway Hydrology.

NATIONAL FLOOD FREQUENCY PROGRAM

- **Regression equations are being widely used**
- **All current equations were compiled in the NFF computer program**
- **A summary of NFF is available in HDS-2**

2.31 National Flood Frequency (NFF) Program

F. As a result of the widespread use of various regression equations, the USGS, in cooperation with the FHWA and the Federal Emergency Management Agency (FEMA), compiled all current statewide and metropolitan area regression equations into a computer program titled the National Flood Frequency (NFF) program. A summary of the NFF, and the applicability and limitations of the regression equations in the NFF, is provided in HDS-2. A complete discussion of NFF program is provided in USGS Water Resources Investigations Report 94-4002 (USGS, 1994).

HYDRAIN COMPUTER PROGRAM

- **HYDRAIN includes seven hydrologic and hydraulic programs**
- **Hydrologic programs are**
  - **HYDRO (log Pearson III, Rational Method, and regression relationships as well as IDF calculations)**
  - **HYDRA (urban storm drain model)**

2.32 HYDRAIN

XV. Hydrologic Analysis with HYDRAIN

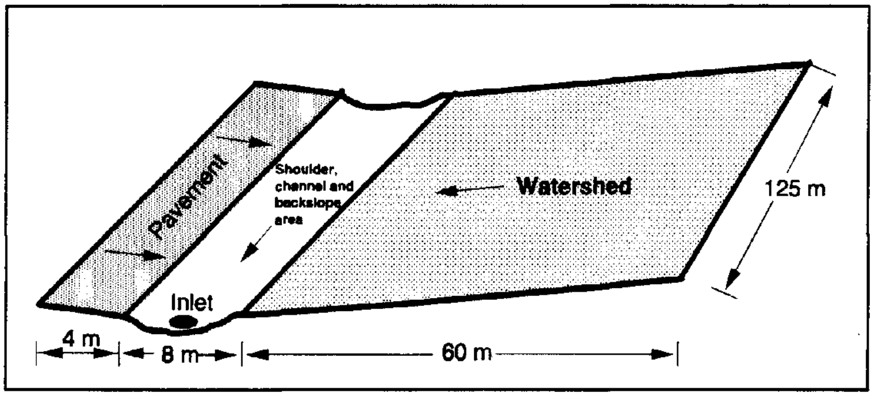
A. HYDRAIN is a comprehensive computer system for highway drainage analysis and design that was developed by FHWA and numerous state transportation agencies. The HYDRAIN system includes nonproprietary computer programs to complete various hydrologic and hydraulic analysis. Many of the programs were developed to facilitate both simple and complex design analysis for problems commonly encountered by transportation agencies.

B. The HYDRAIN system has modules for computing rural flood frequency relationships (HYDRO) and urban relationships (HYDRA). HYDRO is composed of three different discharge volume forecasting methods: Log Pearson III, Rational method, and multiple regression relationships. HYDRO also will develop an intensity-duration-frequency curve for a given latitude and longitude.

HYDRA is a sophisticated urban storm drain model that typically would have only limited application in the small area hydrologic analyses discussed above. Nevertheless, HYDRAIN is a powerful analytical tool that may facilitate some hydrologic analyses described in this lesson, including Rational method calculations and computation of the composite C coefficient.

XVI. Rational Method Workshop

Given: An area tributary to a roadway channel that is 125 m long on a grade of 0.5 percent. The tributary area has a fairly uniform cross section as follows: 4 m of concrete pavement; 8 m gravel shoulder, channel, and backslope; 60 m of forested watershed. The length of the tributary area of 125 m. The overland flow slope in the watershed is also 0.5 percent.



Find: Discharge for 10-year frequency rainfall at a storm drain inlet near the lower end of the roadside channel.

Solution:

A. Evaluate the runoff coefficient C (see tabulated C values provided in this example).

Type of Surface	C	Area (hectare)	CA (hectare)
Concrete pavement			
Shoulder, channel, and backslope			
Forested watershed			
TOTAL			

Weighted C =

B. Overland flow travel time

The overland flow travel time is obtained from the kinematic wave equation (HDS-4, equation 2). Assume n = 0.5 for the watershed.

t =

C. Channel travel time

For the channel (125 m long), the travel time will be estimated based on average velocity and travel distance. From the shallow concentrated flow nomograph provided with this example, the average velocity for an unpaved surface in shallow concentrated flow is about _______ for a slope of ______. For the 125 m channel the travel time is then $t_{channel}$ = ________________________________min.

D. Calculate the total time of concentration

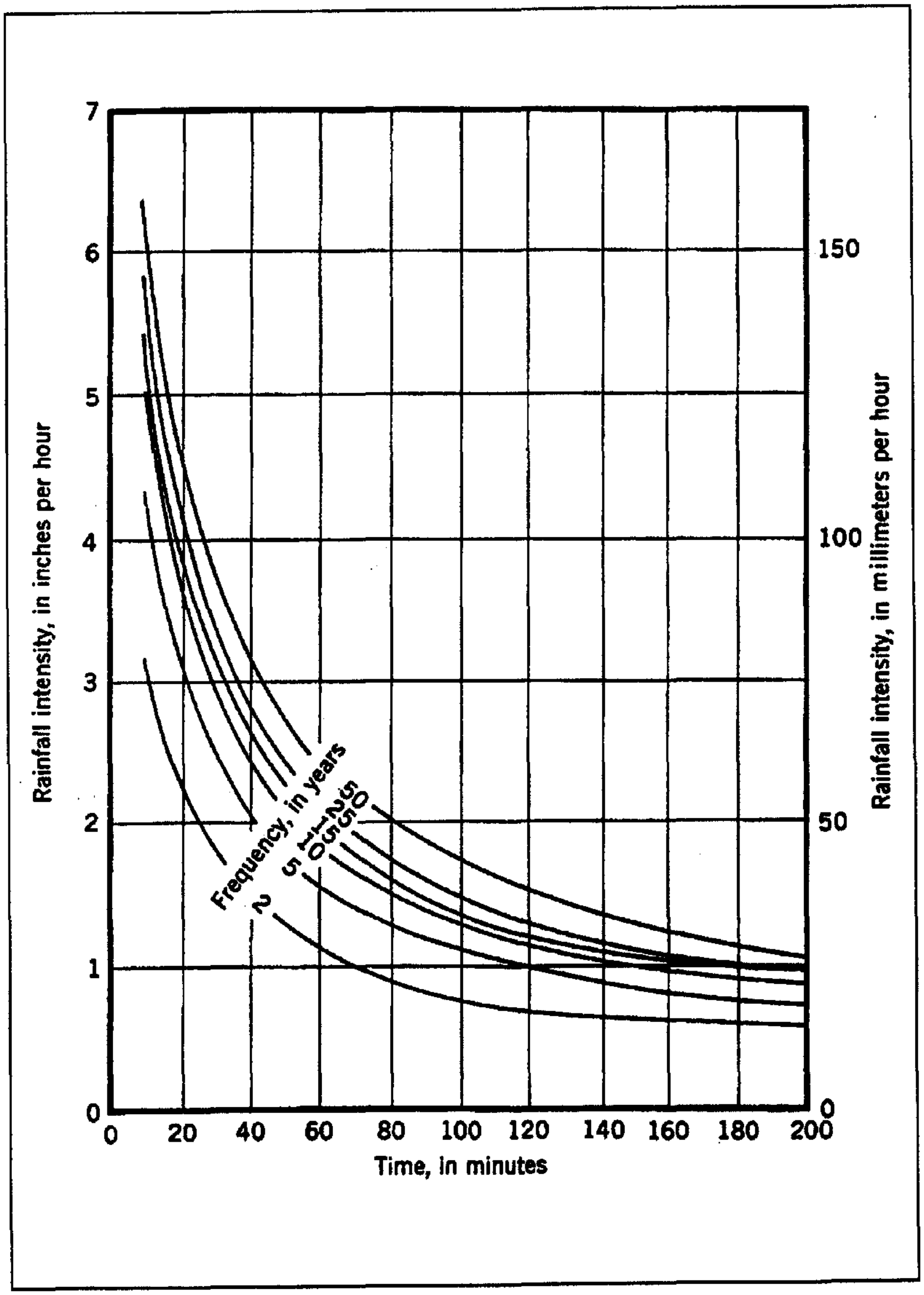
The total t_c =

E. Evaluate the rainfall intensity for the duration equal to the total time of concentration.

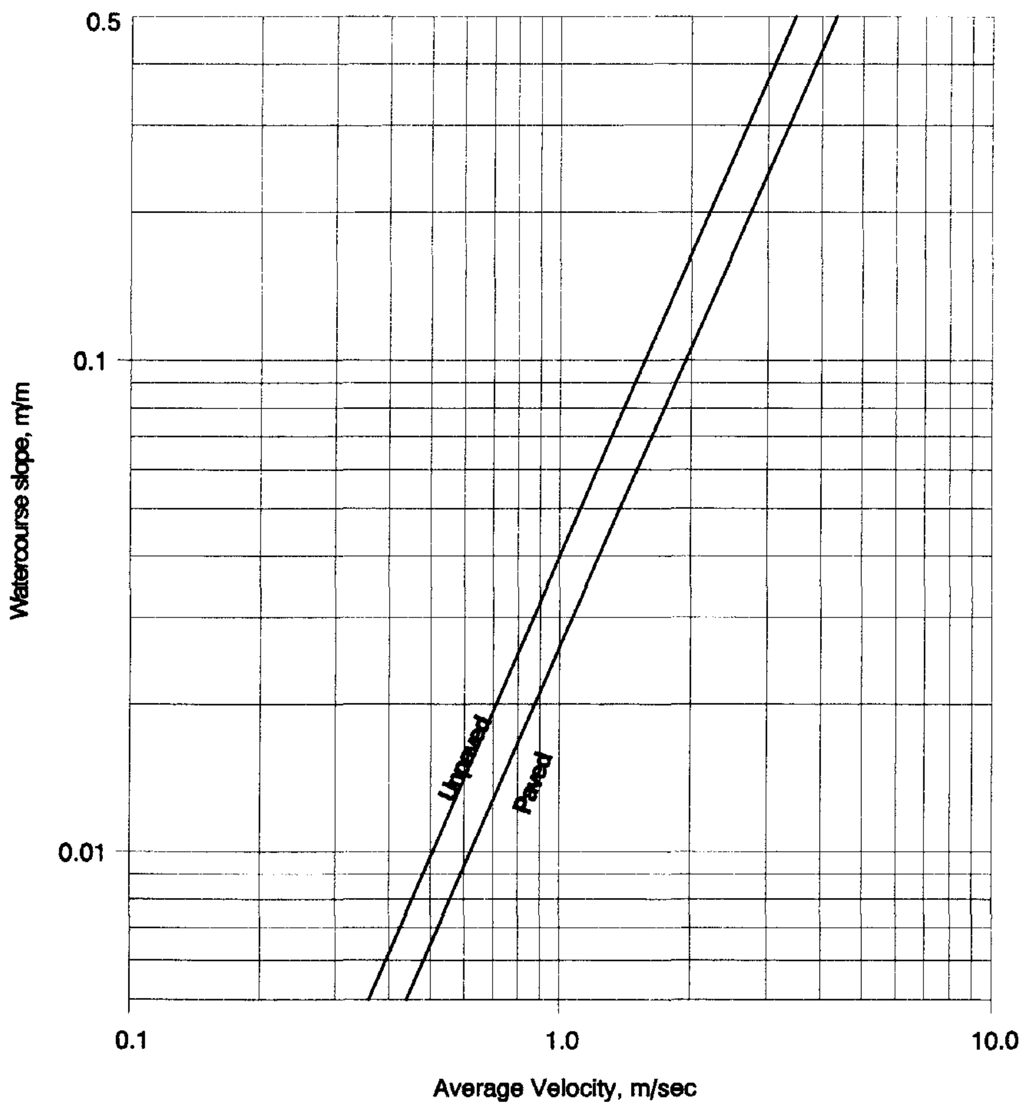
F. Calculate the 10-year discharge at the outlet of the channel

The calculated discharge at the outlet of the channel is:

$Q =$



Typical Intensity-duration rainfall curves (HDS-4, figure 3)



Note: original graph from SCS TR55, 1986; metric conversion completed for HDS-4.

Average velocities for estimating travel time for shallow concentrated flow (HDS-4, figure 4)

Values of Runoff Coefficients, C, for Use in the Rational Method (HDS-4, Table 11).	
Type of Surface	Runoff Coefficient (C^1)
Rural Areas	
Concrete or sheet asphalt pavement	0.8 - 0.9
Asphalt macadam pavement	0.6 - 0.8
Gravel roadways or shoulders	0.4 - 0.6
Bare earth	0.2 - 0.9
Steep grassed areas (2:1)	0.5 - 0.7
Turf meadows	0.1 - 0.4
Forested areas	0.1 - 0.3
Cultivated fields	0.2 - 0.4
Urban Areas	
Flat residential, with about 30 percent of area impervious	0.40
Flat residential, with about 60 percent of area impervious	0.55
Moderately steep residential, with about 50 percent of area impervious	0.65
Moderately steep built up area, with about 70 percent of area impervious	0.80
Flat commercial, with about 90 percent of area impervious	0.80

[1]For flat slopes and permeable soil, use the lower values. For steep slopes and impermeable soil, use the higher values.

NOTES:

LESSON 3

FUNDAMENTAL HYDRAULIC CONCEPTS

BASIC EQUATIONS OF FLOW USED IN DRAINAGE DESIGN

- **Continuity**
- **Energy**
- **Momentum**

3.3 Fundamental hydraulic equations

BASIS FOR HYDRAULIC UNDERSTANDING

- **These equations form the basis for derivation of other hydraulic equations using**
  - **mathematics**
  - **laboratory experiments**
  - **field studies**

3.4 Basis for hydraulic understanding

I. General

A. The design of drainage structures requires the use of the continuity, energy and momentum equations.

B. From these fundamental equations, other equations are derived by a combination of:

1. Mathematics
2. Laboratory experiments
3. Field studies

USE OF EQUATIONS

- **Equations are used differently to analyze**
  - **open-channel flow**
  - **closed conduits flowing full**
- **A closed conduit flowing partially full is open-channel flow**

3.5 Use of equations

OPEN CHANNEL FLOW COMPLEXITIES

- Compared to closed conduits flowing full, open channel flow is complicated by
  - a free surface where the pressure is atmospheric
  - free surface is controlled only by laws of fluid mechanics
  - bed of the stream or conduit may be composed of natural materials such as sand, gravel, boulders or rock that is *moveable*

3.6 Open-channel flow complexities

C. These equations are used differently to analyze open-channel flow and closed conduits flowing full.

D. A closed-conduit flowing partially full is open-channel flow. Compared to closed conduits flowing full, open-channel flow has the complexity of:

1. A free surface where the pressure is atmospheric

2. The free surface is controlled only by the laws of fluid mechanics.

E. Another complexity in open-channel flow is introduced when the bed of the stream or conduit is composed of natural material such as sand, gravel, boulders or rock that is movable.

LIMITATIONS

- Concepts presented in this course assume one-dimensional flow
- Equations and methods are given, however they will not be derived
- Refer to standard fluid mechanics texts, "Highways in the River Environment" or other cited references for more information

3.7 Limitations

LESSON 3

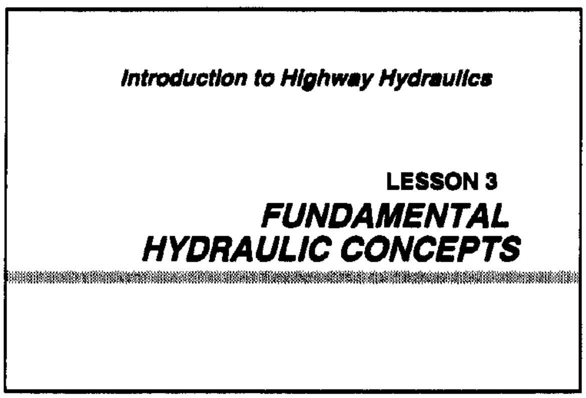
FUNDAMENTAL HYDRAULIC CONCEPTS

OVERVIEW: Method of Instruction: Lecture and demonstration

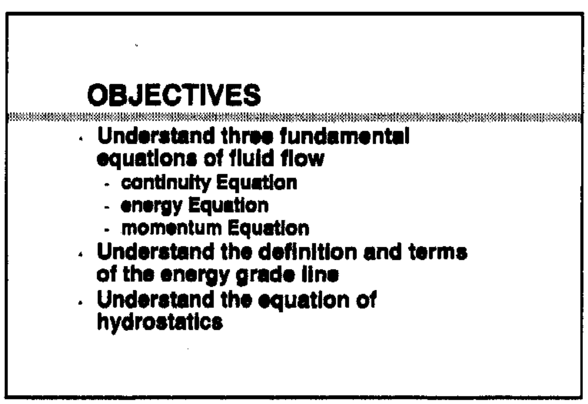
Lesson Length: Lecture - 120 minutes
Demonstration - 60 minutes

Resources:

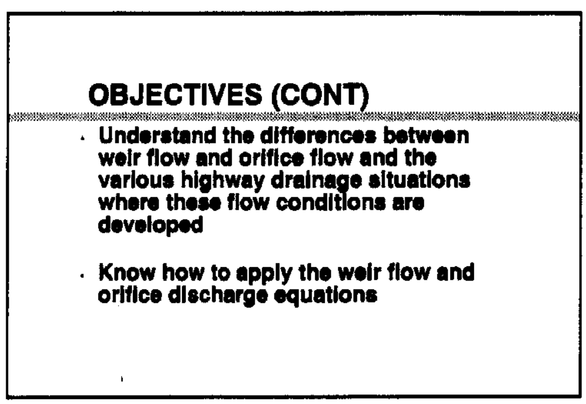
Lesson Outline
Slides
HDS-4
Video
Demonstration Flume



3.0 Title



3.1 Objectives



3.2 Objectives (cont)

OBJECTIVES: At the conclusion of this lesson the Participant should:

1. Understand the three fundamental equations of fluid flow: Continuity, Energy & Momentum

2. Recognize the differences between pressure flow and open channel flow and understand some of the added flow complexities that may be encountered under open channel flow

3. Understand the definition and terms of the Energy Grade Line (EGL)

4. Understand the definition and terms of the Hydraulic Grade Line (HGL)

5. Have a basic familiarity with the equation of hydrostatics and potential acceleration effects

6. Understand the differences between weir flow and orifice flow and the various highway drainage situations where these flow conditions are developed

7. Know how to apply the weir flow and orifice flow equations

F. In the following sections:

1. Concepts presented based on one-dimensional flow assumption (flow assumed to occur only perpendicular to the cross section).

2. The fundamental equations, derived equations and definitions of terms will be given.

3. The equations and methods will **not** be derived.

4. The user is referred to standard textbooks, FHWA publications and the literature cited for additional information.

II. Introduction of the Basic Equations

CONTINUITY EQUATION

- **Derived from the principle of conservation of mass**
- **Except for energy exchange, matter can neither be created nor destroyed**

3.8 Continuity equation principles

A. The basic equations of flow are continuity, energy and momentum. They are derived from the laws of:

1. Conservation of mass - another way of stating that (except for mass-energy interchange) matter can neither be created nor destroyed.

ENERGY EQUATION

- Based on the first law of thermodynamics which states that energy must be conserved at all times

3.9 Energy equation principles

MOMENTUM EQUATION

- Based on Newton's second law of motion (F = ma)
- A mass (of fluid) accelerates in the direction and in proportion to the applied force

3.10 Momentum equation principles

2. Conservation of energy - based on the first law of thermodynamics which states that energy must at all times be conserved.

3. Conservation of linear momentum - based on Newton's second law of motion which states that a mass (of fluid) accelerates in the direction of and in proportion to the applied forces on the mass.

CONSIDERATIONS

- Flow problems are greatly simplified if either:
  - there is no flow acceleration
  - acceleration is primarily in one direction
- Inaccurate results may be determined if accelerations are assumed to be zero or small when they are not

3.11 Considerations

B. Analysis of flow problems are much simplified if there is no acceleration of the flow or if the acceleration is primarily in one direction (one-dimensional flow, with the accelerations in other directions being negligible). However, a very inaccurate analysis may occur if one assumes accelerations are small or zero when in fact they are not.

C. The concepts given in this course assume one-dimensional flow. Only the equations will be given. The user is referred to standard fluid mechanics texts or "Highways in the River Environment" for their derivations.

III. Continuity Equation

CONTINUITY EQUATION

$$V_1 A_1 = V_2 A_2 = Q = V A$$

Applicable When:
- **fluid density is constant**
- **flow is steady**
- **no significant lateral inflow or seepage (or they are accounted for)**
- **velocity is perpendicular to area**

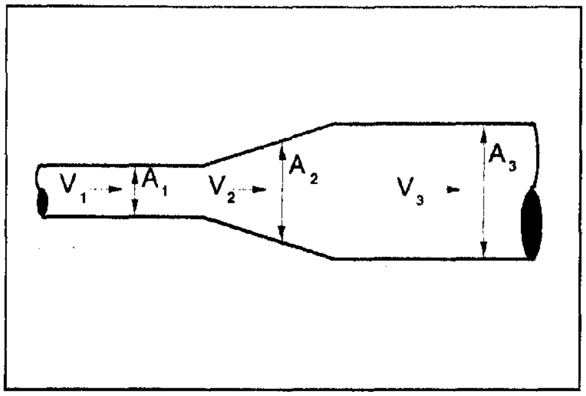
3.12 Continuity equation

A. The continuity equation is:

$$V_1 A_1 = V_2 A_2 = Q = VA$$

where

V = Average velocity in the cross section perpendicular to the area, m/s
A = Area perpendicular to the velocity, m^2
Q = Volume flow rate or discharge, m^3/s



3.13 Sketch of continuity concept

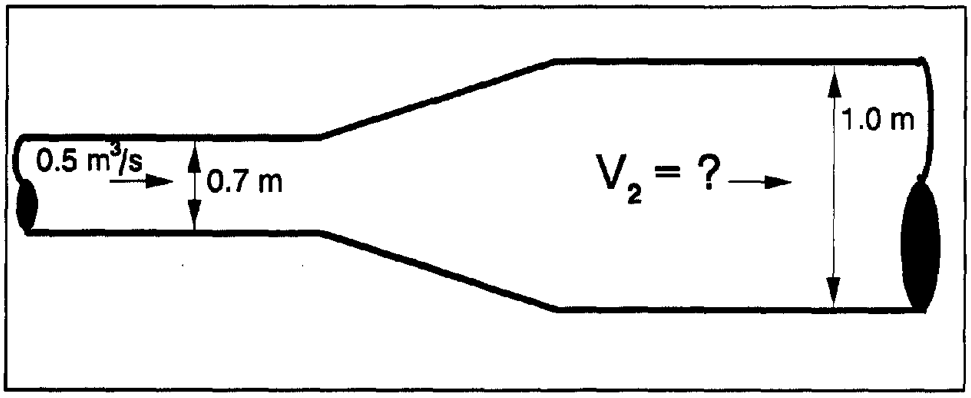
B. The continuity equation is applicable when:

1. Fluid density is constant
2. Flow is steady
3. There is no significant lateral inflow or seepage (or they are accounted for)
4. Velocity is perpendicular to the area

NOTES:

D. Example Problem

Given: A storm drain flowing full transitions from 0.7 to 1.0 m diameter pipe. Determine the average velocity in each section of pipe for a discharge 0.5 m^3/s.



Find: (a) Velocity at section 1 (0.7 m pipe)
(b) Velocity at section 2 (1.0 m pipe)

Solution: Since the discharge at the beginning of the pipe must equal the discharge at the end of the pipe, the continuity equation can be used:

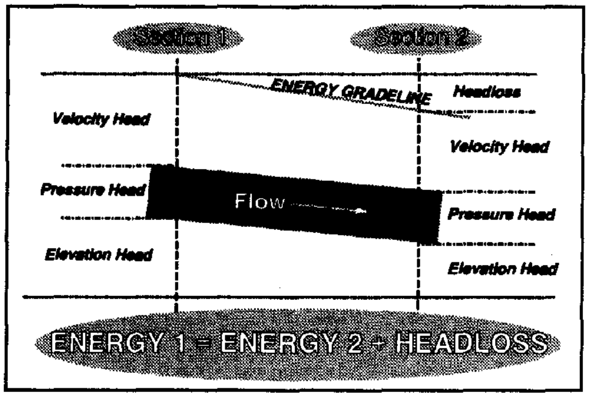
Basic equation: $Q = VA$ Rearrange to get $V = \dfrac{Q}{A}$

For a circular pipe: $A = \dfrac{\pi D^2}{4}$

At cross section 1:

At cross section 2:

IV. Energy Equation



3.14 Energy equation

ENERGY EQUATION

$$\alpha_1 \frac{V_1^2}{2g} + \frac{p_1}{\gamma} + Z_1 = \alpha_2 \frac{V_2^2}{2g} + \frac{p_2}{\gamma} + Z_2 + h_L$$

3.15 Energy equation

A. The energy equation is:

$$\alpha_1 \frac{V_1^2}{2g} + \frac{p_1}{\gamma} + Z_1 = \alpha_2 \frac{V_2^2}{2g} + \frac{p_2}{\gamma} + Z_2 + h_L$$

where

- α = Kinetic energy correction factor, normally assumed to be one
- V = Average velocity in the cross section, m/s
- g = Acceleration of gravity, 9.81 m/s^2
- p = Pressure, N/m^2 or Pa
- γ = Unit weight of water, 9,800 N/m^3 at 15°C
- Z = Elevation above a horizontal datum, m
- h_L = Headloss due to friction and form losses, m

ENERGY GRADE LINE (EGL)
Definition and Components

- **Total energy at any given cross section**
- **Sum of three components**

Velocity head	$\alpha \frac{V^2}{2g}$
Pressure head	$\frac{p}{\gamma}$
Elevation head	z

3.16 Energy grade line components

B. The energy grade line (EGL) represents the total energy at any given cross section, defined as the sum of the three components of energy represented on each side of the energy equation. These components of energy are often referred to as the:

1. Velocity head
2. Pressure head
3. Elevation head

HYDRAULIC GRADE LINE (HGL) - Definition

- **Defined As:**

Energy Grade Line - Velocity Head

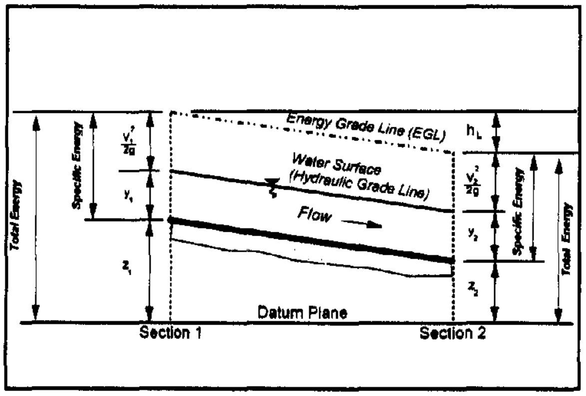
or

Pressure head + Elevation head

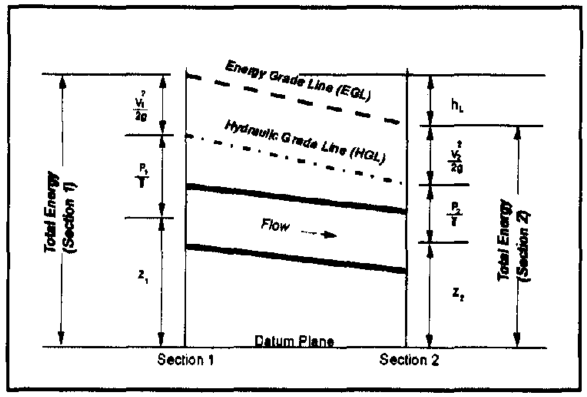
3.17 Hydraulic grade line (HGL) definition

C. The hydraulic grade line (HGL) is below the EGL by the amount of the velocity head, or is the sum of just the pressure head and the elevation head.

D. The energy equation can be applied to open-channel and pressure flow.



3.18 Application of energy principle to open-channel flow

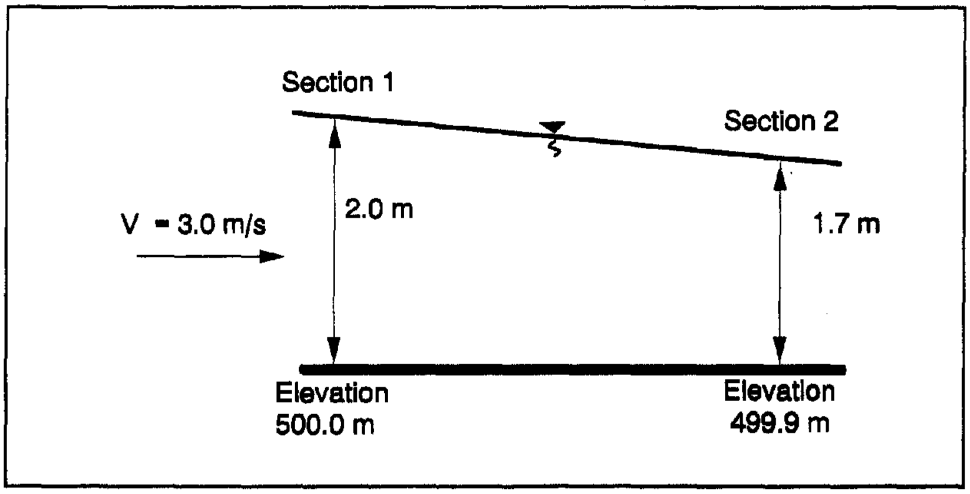


3.19 Application of energy principle to pressure flow

NOTES:

G. Example Problem

Given: The velocity at the upstream end of a rectangular channel 1 m wide is 3.0 m/s, and the flow depth is 2.0 m. The depth at the downstream end is 1.7 m. The elevation at section 1 is 500 m and at 2 is 499.90 m. Assume the kinetic energy correction factor is 1.



Find: (a) Headloss (h_L) due to friction.

Solution: Step 1: Use the continuity equation to find the velocity at section 2.

$$Q = VA$$

Step 2: Use the energy equation to find the headloss, h_L

$$\frac{V_1^2}{2g} + Y_1 + Z_1 = \frac{V_2^2}{2g} + Y_2 + Z_2 + h_L$$

V. Momentum Equation

MOMENTUM EQUATION

- In the x direction for steady flow with constant density

$$\sum F_x = \rho Q (\beta_2 V_{x2} - \beta_1 V_{x1})$$

- Vector Equation - similar terms used for y and z
- Momentum Coefficient (β) generally taken to equal 1.0 since correction is less than 10% even for a very nonuniform velocity distribution

3.20 Momentum equation

A. The momentum equation is:

$$\sum F_x = \rho Q \ (\beta_2 V_{x2} \ - \ \beta_1 V_{x1})$$

where

F_x = Forces in the x direction, N
ρ = Density, 1,000 Kg/m^3
β = Momentum coefficient, normally assumed to be 1
Q = Volume flow rate or discharge, m^3/s
V = Velocity in the x direction, m/s

B. The momentum equation is a vector equation and similar equations are used for the y and z directions.

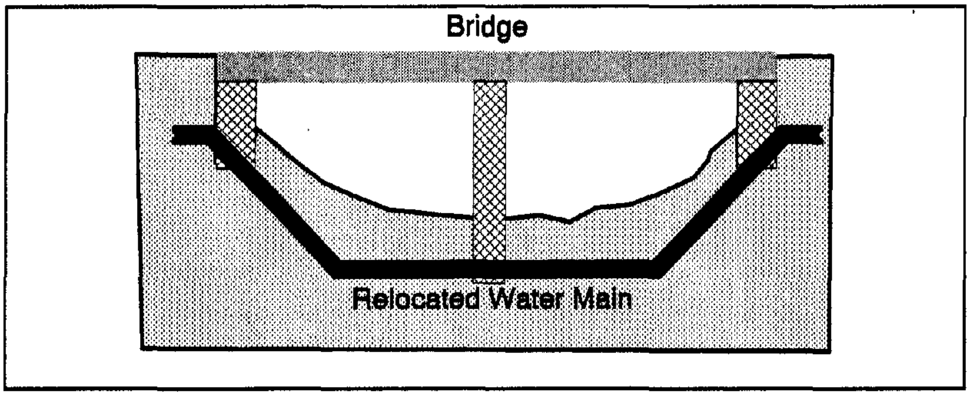
C. The momentum concept is important in grate design, energy dissipators, paved channels intersecting at angles, and hydraulic jump analysis.

D. Video clip on continuity, energy and momentum.

E. Example Problem

Given: For a bridge widening project, an existing city water main must be relocated. The water main is 900 mm in diameter and carries 9.0 m^3/s. Relocation of the water main will require a 45 degree bend in the pipe. The pressure in the pipe at the location of the bend is 78,000 N/m^2.

Find: The forces that an anchor on the pipe at the bend needs to withstand.



Solution:

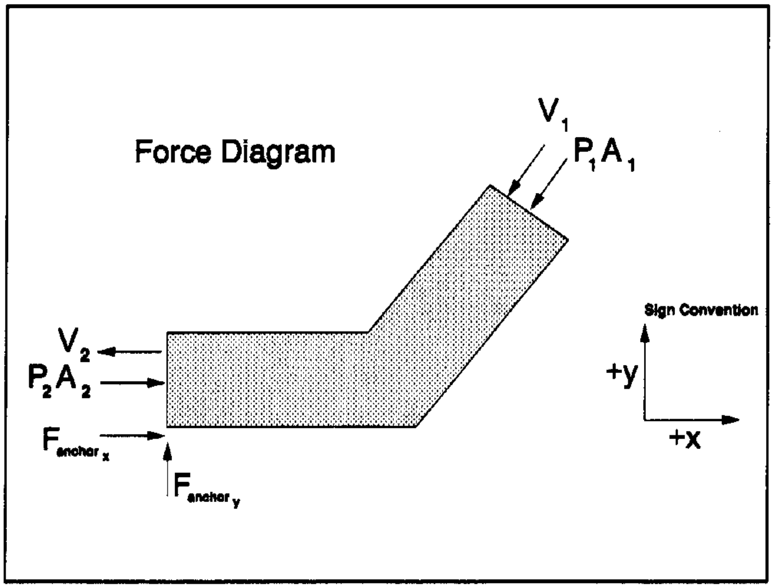
1. Determine velocity in pipe

$$A = \frac{\pi D^2}{4} = \frac{\pi(0.9)^2}{4} = 0.64 \text{ m}^2$$

$$Q = VA \rightarrow V = \frac{Q}{A} = \frac{9.0 \text{ m}^3/\text{s}}{0.64 \text{ m}^2} = 14.1 \text{ m/s}$$

2. Use the momentum equation to find the forces on the bend in the x- and y-directions.

a. First, draw a diagram of the bend and label the forces. (Note: sign convention is important in drawing a force diagram).



b. For forces in the x direction, the Momentum equation states

$$\Sigma F_x = \rho Q\ (V_{2x} - V_{1x})$$

c. The forces acting on the pipe include pressure on either side of the pipe and the resisting of the anchor.

$$F_{anchor_x} + P_{2x} A_{2x} - P_{1x} A_{1x} \cos 45 = \rho Q\ (V_{2x} - V_1 \cos 45)$$

d. First, determine the forces in the x direction (assuming no change in pressure through the bend):

$$F_{anchor_x} + 78{,}000(0.64) - 78{,}000(0.64) \cos 45 = 1000(9)\ (-14.1 + 14.1 \cos 45)$$

$$F_{anchor_x} + 49{,}900 - 35{,}300 = -37{,}200$$

$$F_{anchor_x} = -37{,}200 - 49{,}900 + 35{,}300$$

$$F_{anchor_x} = -51{,}800 \text{ N}$$

e. Next, determine the forces in the y direction:

$$\Sigma F_y = \rho Q\,(V_{2y} - V_{1y})$$

$$F_{anchor_y} + P_{2y}\,A_{2y} - P_{1y}\,A_{1y} = \rho Q\,(V_{2y} - V_{1y})$$

$$F_{anchor_y} + 0 - 78{,}000(0.64)\ \sin 45 = 1000(9)\ (0 + 14.1\ \sin 45)$$

$$F_{anchor_y} - 35{,}300 = 89{,}700$$

$$F_{anchor_y} = 125{,}000\ \text{N}$$

f. Total force is (magnitude and direction)

$$F_{total} = \sqrt{51{,}800^2 + 125{,}000^2}$$

$$F_{total} = 135{,}300\ \text{N}$$

The direction of this force is about 113 degrees counterclockwise from the +x axis. This force equivalent to the weight of 10 automobiles sitting atop the anchor!

VI. Hydrostatics

HYDROSTATICS

- **Steady uniform flow (or no flow) the acceleration is zero**
  - **equation of hydrostatics is:**

$$\frac{p}{\gamma} + Z = \text{Constant}$$

- **When fluid acceleration is small (gradually varied flow) the pressure distribution is considered hydrostatic**

3.21 Equation of hydrostatics

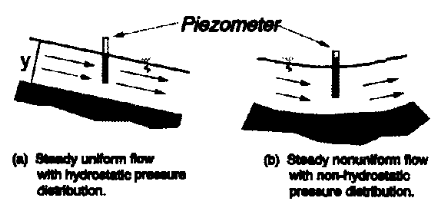
A. In steady uniform flow (and for zero flow), the acceleration is zero and we obtain the equation of hydrostatics.

$$\frac{p}{\gamma} + Z = \text{Constant}$$

ACCELERATION EFFECTS ON PIEZOMETRIC HEAD

- Piezometric head term is not constant and varies in the flow field when there is acceleration
- In curvilinear flow the pressure is larger than γy_o due to acceleration caused by a change in direction

3.22 Acceleration effects on piezometric head



3.23 Acceleration effects

B. However, when there is acceleration, the piezometric head term ($p/\gamma+Z$) varies in the flow field. That is, the piezometric head is not constant in the flow.

HYDROSTATIC PRESSURE

- The constant in the equation for hydrostatics equals zero for gage pressure at the free surface of a liquid
- For flow with hydrostatic pressure throughout
  - steady, uniform flow or
  - gradually varied flow

  the pressure head p/γ is equal to the vertical distance below the free surface

3.24 Hydrostatic pressure

NON-HYDROSTATIC PRESSURE CONDITIONS

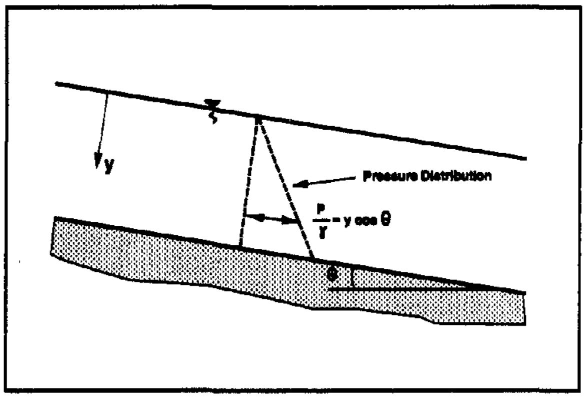
- *Rapidly varying flow*
  - streamlines are converging, expanding or have substantial curvature (curvilinear flow)
  - flow accelerations are not small and the pressure distribution is not hydrostatic

3.25 Non-hydrostatic pressure distributions

C. In general, when fluid acceleration is small (as in gradually varied flow) the pressure distribution is considered hydrostatic.

D. However, for rapidly varying flow where the streamlines are converging, expanding or have substantial curvature (curvilinear flow), fluid accelerations are not small and the pressure distribution is not hydrostatic.

E. The hydrostatics equation constant is equal to zero for gage pressure at the free surface of a liquid, and for flow with hydrostatic pressure throughout (steady, uniform flow or gradually varied flow) it follows that the pressure head p/γ is equal to the vertical distance below the free surface.



3.26 Pressure head in channels with steady uniform flow

F. In sloping channels with steady uniform flow, the pressure head p/γ at a depth y below the surface is equal to

$$\frac{p}{\gamma} = y \cos \theta$$

1. Note that y is the depth (perpendicular to the water surface) to the point.

2. For most channels, θ is small and $\cos \theta \approx 1$.

WEIRS

- Typically a notch of regular shape
  - rectangular
  - square
  - triangular
- Free surface flow condition
- Weir crest
  - edge or surface that the water flows over

3.27 Weir description

WEIR TYPES

- Sharp crested weir
  - weir where water springs free of the crest at the upstream side
- Round edge or suppressed weir
  - short weir crest length where the water does not spring free of the crest
- Broad crested weir
  - sufficiently long horizontal or sloping weir to develop hydrostatic pressure

3.28 Weir types

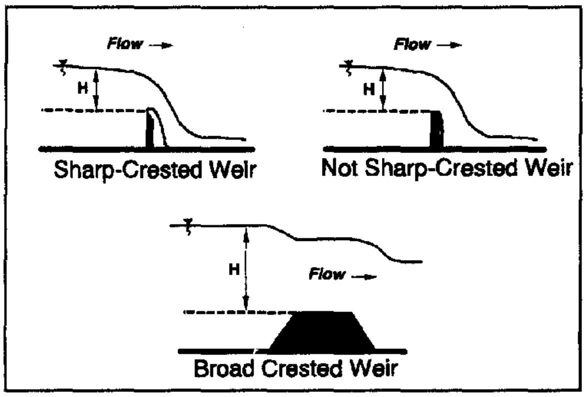
VII. Weirs

A. A weir is typically a notch of regular shape (rectangular, square, or triangular), with a free surface.

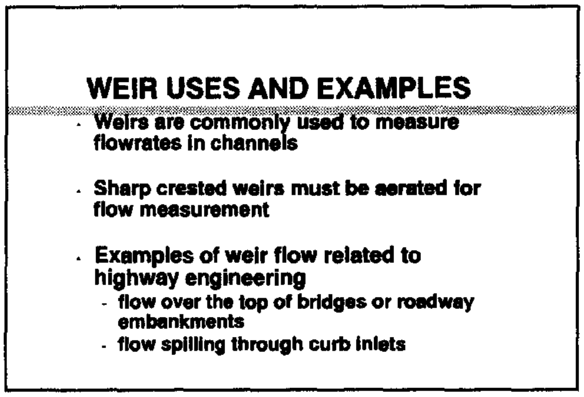
B. The edge or surface over which the water flows is called the crest.

C. A weir with a crest where the water springs free of the crest at the upstream side is called a sharp-crested weir.

D. If the water flowing over the weir does not spring free and the crest length is short, the weir is called a not sharp-crested weir, round edge weir, or suppressed weir.



3.29 Weir types



3.30 Weir uses and examples

E. If the weir has a horizontal or sloping crest sufficiently long in the direction of flow that the flow pressure distribution is hydrostatic it is called a broad- crested weir.

F. As with orifices, weirs can be used to measure water flow.

G. Strictly speaking a sharp-crested weir, for measurement purposes, must be aerated on the downstream side and the pressure on the nape downstream be atmospheric.

H. Examples of weir flow that are of interest to the highway engineer are:

1. Flow-over approach embankments

2. Flow spilling through curb inlets

WEIR EQUATION

- Discharge across a weir (sharp-crested or broad-crested) is:

$$Q = C_D L H^{3/2}$$

- discharge coefficients are available in handbooks
- metric C_D = english $C_D \times 0.552$
- submergence (tailwater) corrections

3.31 Weir equation

I. The discharge across a weir (sharp-crested or broad-crested) is:

$$Q = C_D L H^{3/2}$$

where

Q = Discharge, m^3/s
C_D = Coefficient of discharge for weirs
L = Flow length across the weir, m
H = The head on the weir, m. The depth of flow above the weir crest upstream of the weir (typically measured at a distance of about 2.5H upstream of the weir)

NOTES:

1. Coefficients of discharge are given in most handbooks for the different types of weirs or flow conditions.

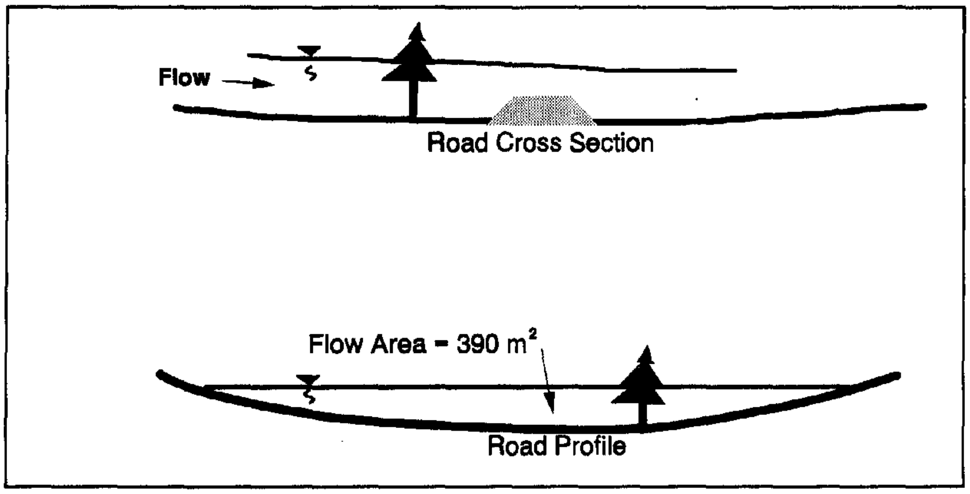
2. C_D has units of $\sqrt{g}$ and values tabulated in English units must be converted to metric units by multiplying by the factor $\sqrt{9.81} / \sqrt{32.2}$ or 0.552.

3. Correction factors are also available if the weir is submerged.

J. Example Problem

Given: During a flood, water overtops a roadway embankment at a sag in the roadway profile.

Find: Determine the amount of flow over the road and its velocity if the inundated roadway length = 130 m and C_D = 3.1 (English units). The flow area was calculated to be 390 m^2 based on a high-water mark on a tree and the roadway profile.



Find: Discharge and velocity

Solution: Use the broad-crested weir equation and the continuity equation since the road acts as a weir.

$$Q = C_D L H^{3/2} \quad \text{and} \quad Q = VA$$

a. Since the flow depth changes across the length of the road, use the hydraulic depth for H (area / topwidth)

b. Find the velocity of the flow from the continuity equation.

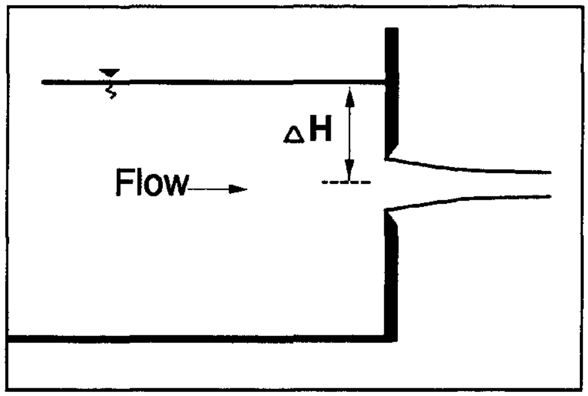
$$Q = VA$$

VIII. Orifices

ORIFICES

- Orifice is an opening with regular shape
  - circular
  - square
  - rectangular
- Water is in total contact with the perimeter
- If opening is only partially full, orifice becomes a weir

3.32 Orifice definition



3.33 Orifice flow representation

A. An orifice is an opening with a regular shape (circular, square or rectangular) through which water flows in contact with the total perimeter.

B. If the opening is flowing only partially full, the orifice becomes a weir.

ORIFICE FLOW CONDITIONS

- Free discharge orifice - orifice jet discharges into the air
- Submerged orifice - discharges under water
- Common flowrate measuring devices
- Orifice flow conditions are also developed under circumstances where head losses, backwater effects need to be determined

3.34 Orifice conditions

C. An orifice with a sharp upstream edge is called a sharp-edged orifice.

D. If the jet of water from the orifice discharges into the air, it is called a free discharge.

E. If it discharges under water, it is called a submerged orifice.

F. Orifices are common fluid discharge measuring devices, but orifice-type flow occurs under other circumstances where headloss, backwater, etc. needs to be determined.

ORIFICE FLOW CONDITIONS

- Examples of orifice flow of interest to highway engineers
  - flow through bridges when they are overtopped
  - flow through culvert inlets
  - curb inlets flowing full
- When a bridge is overtopped
  - flow through the bridge is orifice flow
  - flow over the bridge is weir flow

3.35 Orifice flow conditions

G. Examples of orifice flows of interest to highway engineers are:

1. Flow through bridges when they are overtopped
2. Flow-through culvert inlets
3. Curb inlets flowing full

H. When a bridge is overtopped the flow through the bridge is orifice flow, but the flow over the bridge is weir flow.

ORIFICE EQUATION

- Equation
- Discharge coefficients are given in most handbooks

$$Q = C_D A\sqrt{2g\,\Delta H}$$

3.36 Orifice equation

I. The discharge through an orifice is:

$$Q = C_D A \sqrt{2g\Delta H}$$

where Q = Discharge, m^3/s
C_D = Coefficient of discharge
A = Area of the orifice, m^2
g = Acceleration of gravity = 9.81 m/s^2
ΔH = Difference in head across the orifice, m

J. Coefficients of discharge are given in most handbooks. For an unsubmerged orifice, the difference in head is measured from the centerline of the orifice to the upstream water surface. For a submerged orifice, it is the difference between the upstream and downstream water surface elevations.

IX. Demonstration

Purpose: To use the demonstration flume to illustrate the following concepts:

1. Continuity principle
2. Energy principle
3. Weir flow
4. Orifice flow

A. Demonstration 1

The continuity equation will be validated by comparing the measured discharge with the product of the measured velocity and area. The discharge will be measured with a bucket and stop watch. The velocity will be measured with a floating ball and stop watch, and the area will be directly measured.

The application of the continuity principle will be illustrated by a side wall contraction in open-channel flow. The discharge rate is the same in both reaches. This can be demonstrated by comparing a velocity-area measurement in the upstream reach with a bucket/stop watch measurement at the outlet. Knowing the discharge, the velocity at any cross section in the contraction can be determined by applying the continuity equation after measuring the flow area.

NOTES:

B. Demonstration 2

The energy principle will be demonstrated by measuring the three components of the energy equation. The application of the energy equation will be illustrated by comparing the EGL between the headbox and flume with the sluice gate in place.

NOTES:

C. Demonstration 3

Both broad- and sharp-crested weir flow conditions will be demonstrated. By adjusting tailwater conditions, both free overall and submerged weir flow conditions will be created. For a given discharge, the differences in headwater will be observed between broad- and sharp-crested weirs (remember that a sharp-crested weir is one where the water springs clear of the crest).

The coefficient of discharge for both broad- and sharp-crested weirs will be determined experimentally and compared to tabulated values (e.g., Handbook of Hydraulics, by Ernest Brater and Horace King, sixth edition, McGraw Hill Book Company, pg 5-40 for broad-crested weirs; pages 5-7 to 5-8 for sharp-crest weirs). Note that the coefficient of discharge for broad-crested weirs in English units is about 2.8-3.2, while for sharp-crest weirs it is typically about 3.5.

NOTES:

D. Demonstration 4

Orifice flow conditions will be demonstrated for a small orifice with a free discharge. For a given discharge, the change in headwater will be measured between a sharp-edged orifice and an orifice with a rounded inner face to illustrate the importance of inlet condition. The effects of submergence will also be demonstrated.

The coefficient of discharge for a sharp-edged orifice will be determined experimentally and compared to tabulated values (e.g., Handbook of Hydraulics, by Ernest Brater and Horace King, sixth edition, McGraw Hill Book Company, pg 4-31). Note that the coefficient of discharge for a sharp-edged orifice is about 0.61.

NOTES:

LESSON 4

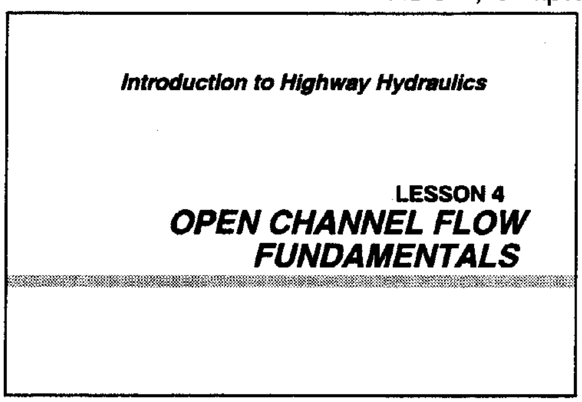
OPEN-CHANNEL FLOW FUNDAMENTALS

OVERVIEW: Method of Instruction: Lecture

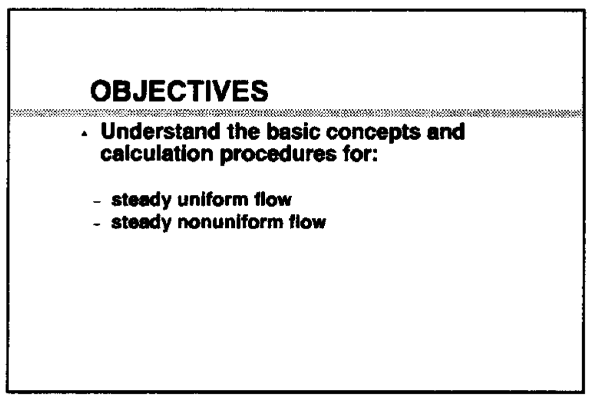
Lesson Length:	Steady Uniform Flow	-	75 minutes
	Steady Nonuniform Flow	-	75 minutes
	Demonstration	-	105 minutes

Resources:

Lesson Outline
Slides
Video
Demonstration Flume
HDS-4, Chapter 4



4.0 Title



4.1 Objectives

OBJECTIVES: At the conclusion of this lesson, the Participant should:

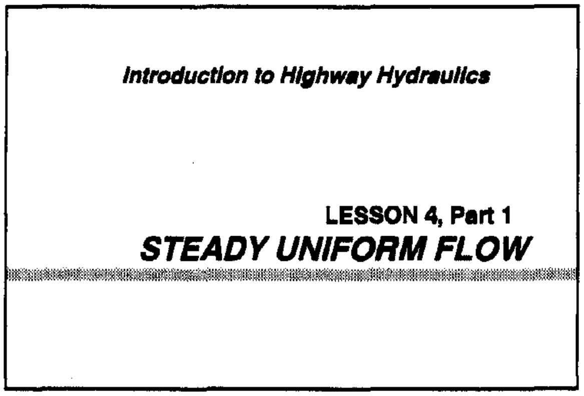
1. Understand the basic concepts and calculation procedures for steady uniform flow (Manning's equation, roughness determination, velocity distribution and shear stress and Froude Number).

2. Understand the basic concepts and calculation procedures for steady nonuniform flow (rapidly varied versus gradually varied flow, specific energy and specific discharge, hydraulic jump, standard step method).

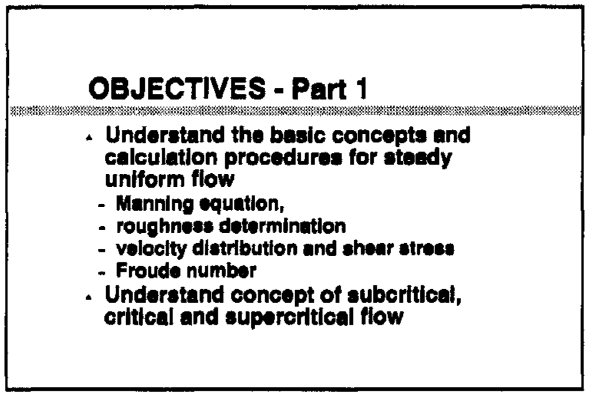
LESSON 4

OPEN-CHANNEL FLOW FUNDAMENTALS

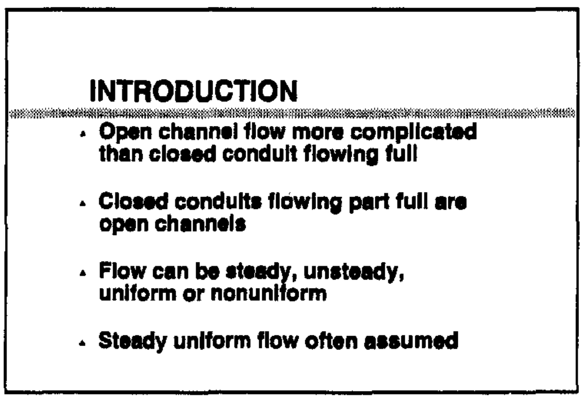
PART 1 - STEADY UNIFORM FLOW



4.2a Part 1, Steady uniform flow



4.2b Part 1 objectives



4.3 Introduction

I. Introduction

A. Open-channel flow is more complex than closed-conduit flow flowing full, because of a water surface. An additional complexity is introduced when the boundary is mobile (e.g., alluvial channel conditions).

B. Open-channel flow can occur in open channels or closed conduits flowing partly full.

C. Open-channel flow can be steady or unsteady, uniform or nonuniform. The simplest flow condition is steady uniform flow.

STEADY UNIFORM FLOW

- Steady uniform flow is rare in natural channels
- Steady uniform flow assumption is reasonable for many practical problems
- Many flows are near steady uniform conditions under "normal" conditions
- Definition of variables in normal depth flow conditions

4.4 Steady uniform flow

D. In steady uniform flow the depth, water area, velocity and discharge are constant over time (steady) at every section of the channel (uniform). Additionally, the energy grade line, water surface and channel bottom are all parallel (that is, their slopes are all equal).

E. Steady uniform flow is rare in natural streams; however, this condition is frequently assumed in the computation of flow conditions. The results obtained are understood to approximate the real flow conditions.

F. Steady uniform flow is often assumed because it provides a relatively simple and satisfactory solution to many practical problems.

G. For all practical purposes, steady uniform flow can be assumed under "normal" conditions. That is, in the absence of flood flows or varied conditions caused by boundary irregularities.

H. The depth in steady uniform flow is called the normal depth. The symbol for it is given the subscript o as in Y_o. The velocity V is often given the same subscripts, i.e., V_o. Other variables of interest for steady uniform flow are (1) the discharge Q, (2) the velocity distribution v_y in the vertical, (3) the headloss H_L through the reach, and (4) the shear stress, both local and at the bed τ_o. All these variables are interrelated.

MANNING'S EQUATION

- **Water flows in a sloping channel due to force of gravity**
- **Flow is resisted by friction between water and wetted surface**
- **This relationship described by Manning's Equation**

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

4.5 Manning's equation

II. Manning's Equation for Mean Velocity and Discharge

A. Water flows in a sloping drainage channel because of the force of gravity. The flow is resisted by the friction between the water and wetted surface of the channel. The quantity of water flowing (Q), the depth of flow (y), and the velocity of flow (V) depend upon the channel shape, roughness (n), and slope (S_0).

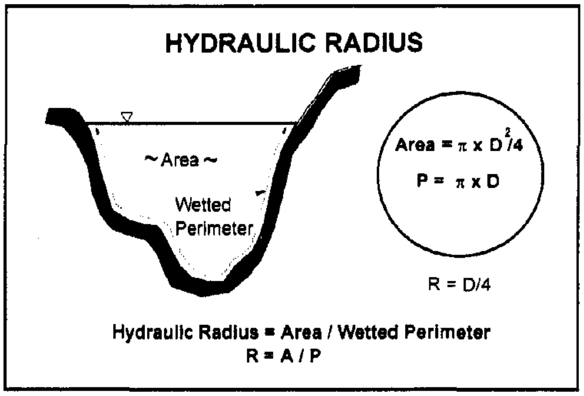
B. The metric form of Manning's equation for the velocity of flow in open channels is:

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

where
V = Mean velocity, m/s
n = Manning's coefficient of channel roughness
R = Hydraulic radius, m
S = Energy slope, m/m
For steady uniform flow $S = S_0$

C. Manning's equation is applicable only for steady, uniform flow.

D. Estimation of n value is a critical part of applying Manning's equation.



4.6 Hydraulic radius

E. Note that the hydraulic radius, R, is a shape factor that depends only upon the channel dimensions and the depth of flow. It is computed by the equation:

$$R = \frac{A}{P}$$

where

A = Cross-sectional area of the flowing water perpendicular to direction of flow, m^2

P = Wetted perimeter or the length of wetted contact between a stream of water and its containing channel, perpendicular to the direction of flow, m

For a circular pipe flowing full, the hydraulic radius simplifies to the diameter divided by four.

MANNING'S EQUATION

- From continuity

$$Q = V\,A$$

- Therefore,

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

4.7 Manning's equation for discharge

F. The discharge Q is determined from the equation of continuity:

$$Q = AV$$

where
- Q = Discharge, m^3/s
- A = Cross-sectional area, m^2
- V = Mean velocity, m/s

By combining these equations, the Manning's equation can be used to compute discharge directly, or given the discharge, can be used to solve for flow conditions.

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

ESTIMATING "n" VALUES

- Tabulated values
- Equations
- Photographs

4.8 Estimating n values

III. Estimating Manning's n Value

A. The Manning's n value can be estimated from tabulated values by knowing the general nature of the channel boundaries (see HDS-4, table 12).

B. Equations also exist for calculating Manning's n values.

C. One of the more accurate methods for estimating Manning's n values is to review case study photographs with n values calculated from field data.

CASE STUDY "n" VALUE INFORMATION

- "Roughness Characteristics of Natural Channels," USGS Water-Supply Paper 1849
- "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains," USGS Water-Supply Paper 2339

4.9 Sources for case study information

4.10a Channel n = 0.030

4.10b Channel n = 0.045

4.10c Channel n = 0.070

4.11a Floodplain n = 0.12

4.11b Floodplain n = 0.20

CONVEYANCE

- Cross section properties can be grouped in a term called conveyance

$$K = \frac{1}{n} A R^{2/3}$$

- Manning Equation is then

$$Q = K S^{1/2}$$

4.12 Conveyance

IV. Conveyance and the Manning's Equation

A. In many computations, it is convenient to group the cross-sectional properties in the Manning's equation into a term called conveyance, K,

$$K = \frac{1}{n} A R^{2/3}$$

then

$$Q = K S^{1/2}$$

CONVEYANCE CALCULATION

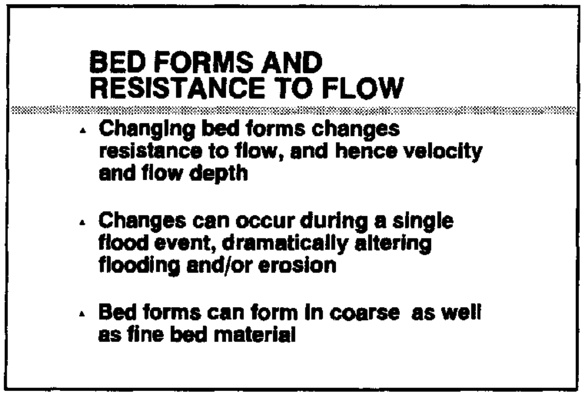
- Irregular shapes must be subdivided
- Variable roughness must be subdivided
- Conveyance curve can be drawn
- Conveyance can be used to evaluate distribution of flow through multiple bridge openings.

4.13 Calculating conveyance

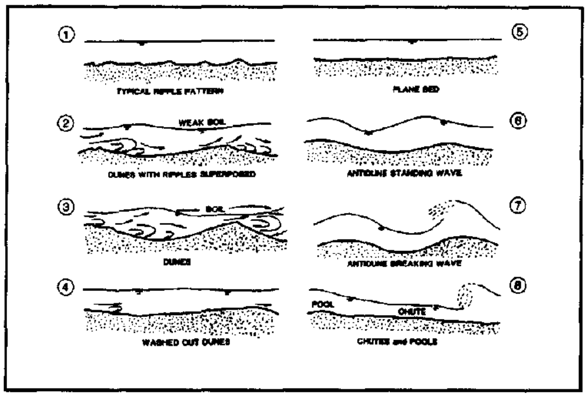
B. When a channel cross section is irregular in shape such as one with a relatively narrow deep main channel and wide shallow overbank area, the cross section must be subdivided and the conveyance and/or discharge computed separately for the main channel and overbank area.

C. Concrete channels and culverts may have an alluvial boundary because of deposition of bed material in the invert.

D. In sand-bed channels the bed material is easily eroded and continually being moved and shaped by the flow. The interaction between the flow of the water-sediment mixture and the sand bed creates different bed configurations which change the resistance to flow, velocity, water surface elevation, and sediment transport.



4.16 Bedforms and resistance to flow



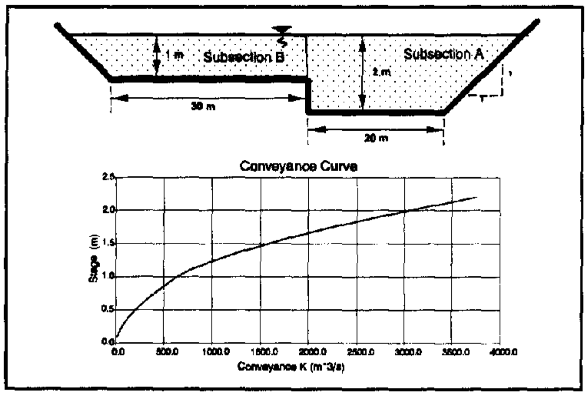
4.17 Forms of bed roughness

E. Flow in alluvial channels is divided into two regimes separated by a transition zone. Forms of bed roughness in sand channels are shown in the following figure. The impact of changing bedforms can dramatically change resistance flow, and hence velocity and flow depth creating either increased flooding or erosion.

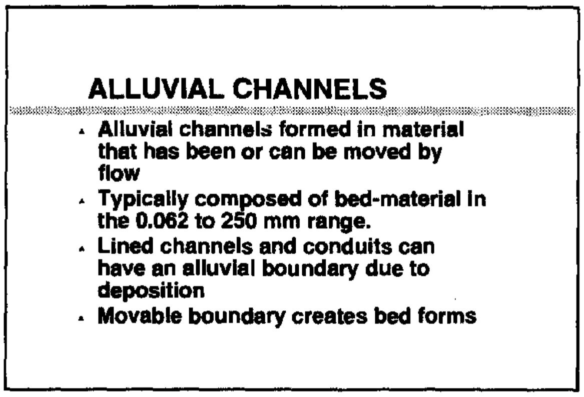
C. The same procedure is used when different parts of the cross section have different roughness coefficients. In computing the hydraulic radius of the subsections, the water depth common to the two adjacent subsections is not counted as wetted perimeter.

D. Conveyance can be computed and a curve drawn for any channel cross section as a function of depth or stage. If the section was subdivided, the conveyance of each subsection (K_a, K_b,...K_n) is computed and the total conveyance of the channel is the sum of the conveyances of the subsections.

E. The concept of channel conveyance is useful when computing the distribution of overbank flood flows in the stream cross section and the distribution through the openings in a proposed stream crossing. The discharge through each opening can be assumed to have the same ratio to the total discharge as the ratio of conveyance of the opening bears to the total conveyance of the channel.



4.14 Conveyance curve for a subdivided compound channel



4.15 Alluvial channels

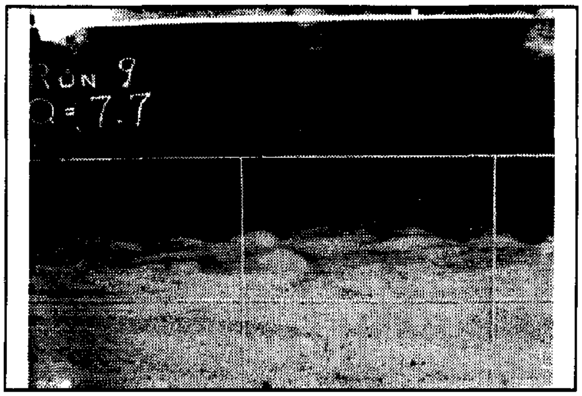
V. Resistance to Flow in Alluvial Channels

A. Alluvial channels are channels formed in material that has been and can be transported by the flow.

B. They are commonly made up of bed material composed of sand-, gravel-, and cobble-sized material in sizes ranging from 0.062 mm for fine sand to 250 mm for large cobbles.

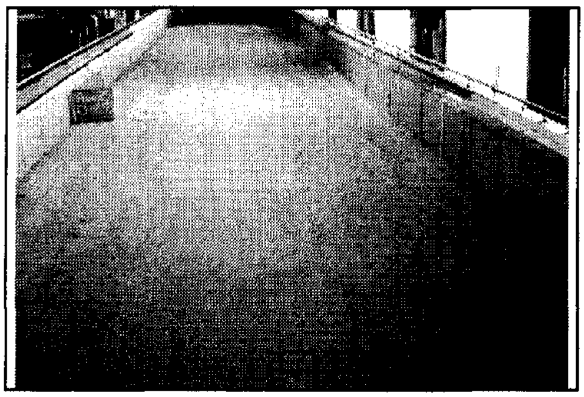
F. In contrast to sand-bed channels, coarse-bed channels may not move at low flow, but at moderate or large flows, the material may become mobile. With the movement of coarse bed material, large bars may form which will be residual at low flow. These bars can re-direct flow and cause bank erosion, scour holes, and clog drainage channels. However, coarse bed material in drainage channels can have a beneficial effect by decreasing erosion by armoring of the bed.

NOTES:

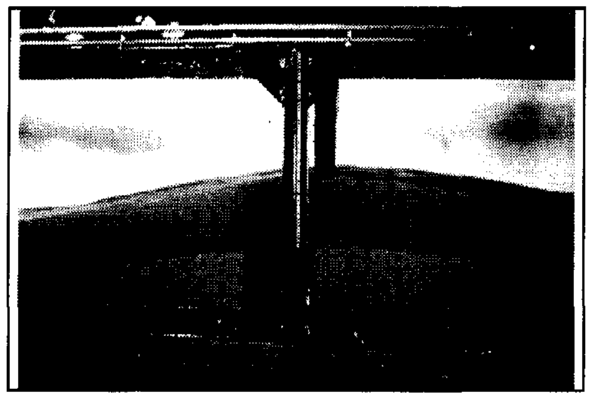


4.18a Ripples

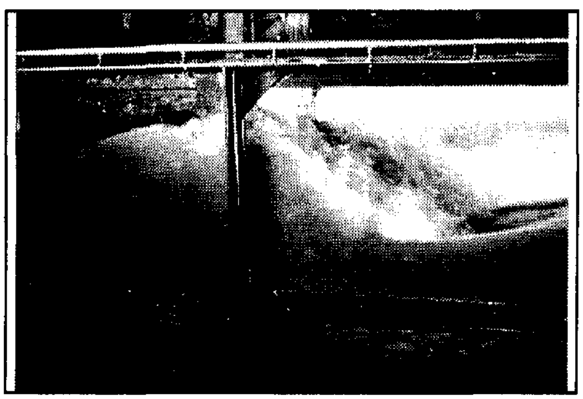
4.18b Dunes



4.18c Plane bed



4.18d Antidunes



4.18e Breaking antidunes

MANNING'S "n" VALUES FOR BEDFORM CONDITIONS

Ripples	0.018-0.030
Dunes	0.020-0.040
Plane Bed	0.010-0.013
Standing Waves	0.010-0.015
Antidunes	0.012-0.020

4.19 Manning's n values for bedforms

AIDS FOR SOLVING MANNING EQUATION

- Formulas for geometric properties of various prismatic channel shapes
- Nomographs
- programmable calculators
- computers
  - spreadsheets
  - special software

4.20 Aids for solving Manning's equation

VI. Aids for Solution of the Manning's Equation

A. Formulas for geometric properties of various channel shapes are readily available.

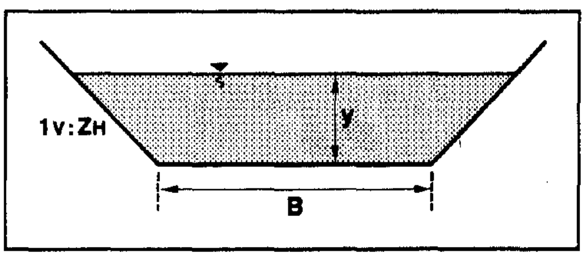
B. Procedures based on nomographs and tabulated values are available; however, calculators and computers have made this approach obsolete. Computer applications based on a spreadsheet, or special software, are available.

C. For rectangular and trapezoidal channels, the equations for the computation of Area A, wetted perimeter, P, and hydraulic radius, R, are:

$$A = B\,y + Z\,y^2$$

$$P = B + 2y\sqrt{1+Z^2}$$

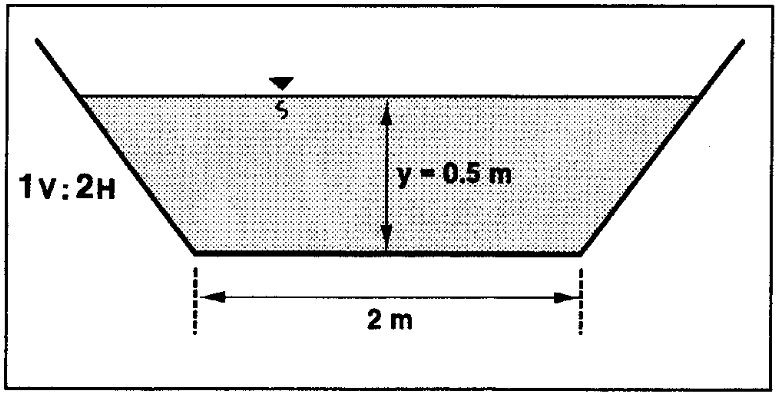
$$R = \frac{By + Zy^2}{B + 2y\sqrt{1+Z^2}}$$



VII. Manning's Equation Problems

Part 1: Application of Manning's equation to determine velocity and discharge.

Given: Trapezoidal earth channel B = 2 m, sideslope 1V:2H, S = 0.003 m/m, Normal depth y = 0.5 m, n = 0.02.



Find: Velocity V and discharge Q

Solution:

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

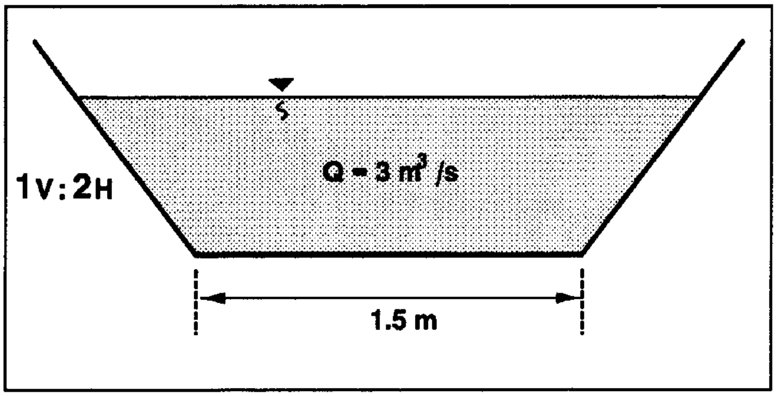
$$R = \frac{By + Zy^2}{B + 2y\sqrt{1 + Z^2}} =$$

$V =$

$Q = AV =$

Part 2: Application of Manning's equation to determine velocity and flow depth, given discharge.

Given: A concrete trapezoidal channel B = 1.5 m, sideslopes 1V:2H, n = 0.013, slope = 0.002, Q = 3 m^3/s



Find: Depth y and velocity v

Solution: Use Manning's equation with continuity

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

Relationships for A and R are

$$A = By + Zy^2 =$$

$$R = \frac{By + Zy^2}{B + 2y\sqrt{1 + Z^2}} =$$

Substituting A and R into Manning's Equation

Trial and Error Solution for y

Try $y = 0.70$;

Try $y = 0.60$;

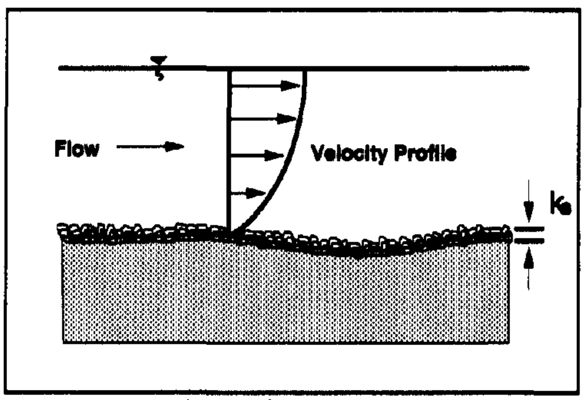
VELOCITY VARIATIONS

- Velocity in a given cross section is not uniformly distributed
- Velocity can vary horizontally, particularly in bends
- Velocity can vary vertically

4.21 Velocity variations

VIII. Velocity Distribution and Shear Stress

A. Velocity is not constant throughout the cross section, but varies horizontally and vertically. Horizontal variations are particularly prevalent in channel bends.



4.22 Velocity profile schematic

VELOCITY PROFILE

- Vertical velocity distribution:

$$\frac{v}{V_*} = 5.75 \log \left(30.2 \frac{X y}{k_s}\right)$$

$$\frac{V}{V_*} = 5.75 \log \left(12.27 \frac{X y_o}{k_s}\right)$$

4.23 Velocity profile equations

B. As a result of boundary roughness, the velocity varies vertically from some minimum value along the bed to a maximum value near the water surface.

C. There are times in the design of highway drainage facilities that knowledge of the velocity distribution in the vertical is needed (e.g., the design of riprap for scour and erosion control).

D. The equations for velocity distribution (v), mean velocity (V) can be written in the following dimensionless forms:

$$\frac{v}{V_*} = 5.75 \log \left(30.2 \frac{X\, y}{k_s}\right)$$

and

$$\frac{V}{V_*} = 5.75 \log \left(12.27 \frac{X\, y_0}{k_s}\right)$$

Note that any system of units can be used as long as y_o and k_s (and V, v and V_*) have the same units.

where
- X = Coefficient (see figure below)
- k_s = Measure of the roughness height. k_s varies from the D_{84} size for pure sand-bed channels, to 3.5 times D_{84} for graded coarse bed streams. For practical application use 3.5 times D_{84}.
- y = Depth to specified location
- v = Local mean velocity at depth y
- y_o = Depth of flow
- V = Depth-averaged velocity
- V_* = Shear velocity $(\tau_o/\rho)^{0.5}$
- τ_o = Shear stress at the boundary
- δ' = Thickness of the viscous sublayer $(11.6\ \mu/V_*)$
- μ = Dynamic viscosity of water (see HDS-4, table 8)
- ρ = Density of water

SHEAR STRESS

- ▸ **Shear stress is the force water exerts on the bed and bank of a channel**
- ▸ **Average shear stress on wetted perimeter is**

$$\tau_o = \gamma\, R\, S_o$$

4.24 Shear stress

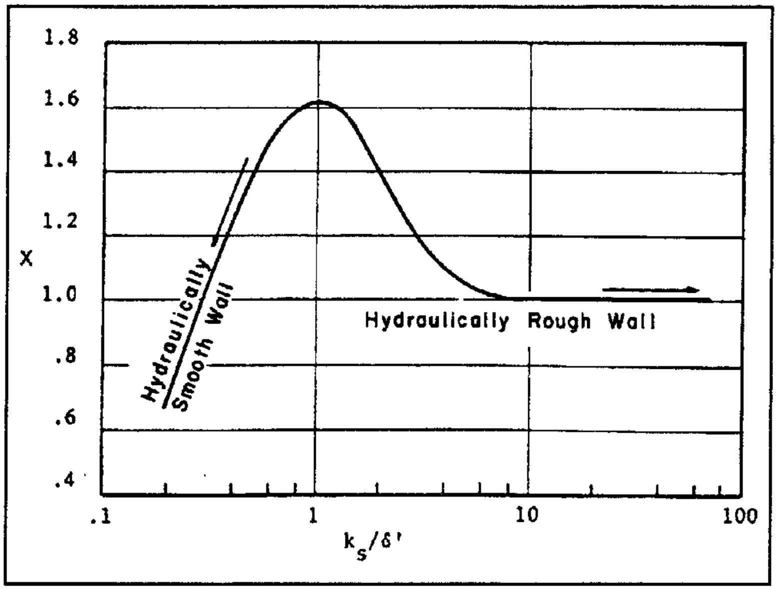


Figure for evaluation of coefficient X

D. Shear stress is the force water exerts on the bed and bank of a channel as it flows over them. The following equations can be used to determine the shear stress on the boundary of the channel that results from the force of flowing water.

$$\tau_0 = \gamma R S_0$$

where

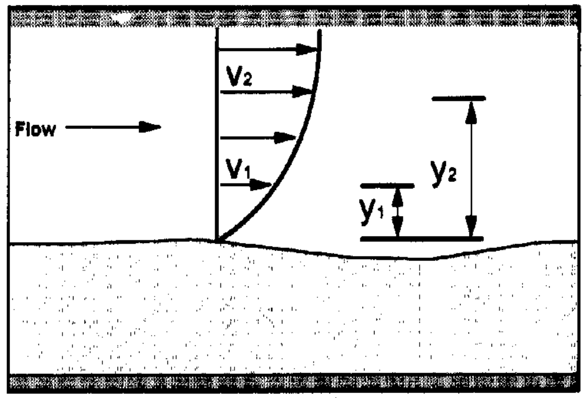
τ_0 = Average shear stress on the wetted perimeter, N/m^2
γ = The unit weight of water, N/m^3
R = Hydraulic radius, m
S_0 = Slope of the channel, m/m. In gradually varied flow the slope is of the energy grade line, $S_0 = S_f$

SHEAR STRESS

- Shear stress at a specific location based on velocity in vertical

$$\tau_o = \frac{\rho\ (v_1 - v_2)^2}{[\ 5.75\ \log(\frac{y_1}{y_2})]^2}$$

4.25 Shear stress in vertical



4.26 Velocity profile schematic

SHEAR STRESS

- Shear stress at a specific location based on mean velocity in vertical

$$\tau_o = \frac{\rho V^2}{[\ 5.75\ \log\ (12.27\ \frac{y_o}{k_s}\ )]^2}$$

4.27 Shear stress in the vertical

$$\tau_0 = \frac{\rho\ (v_1 - v_2)^2}{[5.75\ \log\ (\frac{y_1}{y_2})]^2}$$

$$\tau_0 = \frac{\rho\ V^2}{[5.75\ \log\ (12.27\ \frac{y_0}{k_s})]^2}$$

where

τ_0 = Shear stress at a point in the flow (N/m^2), v_1 and v_2 are point velocities in the vertical at y_1 and y_2, respectively;

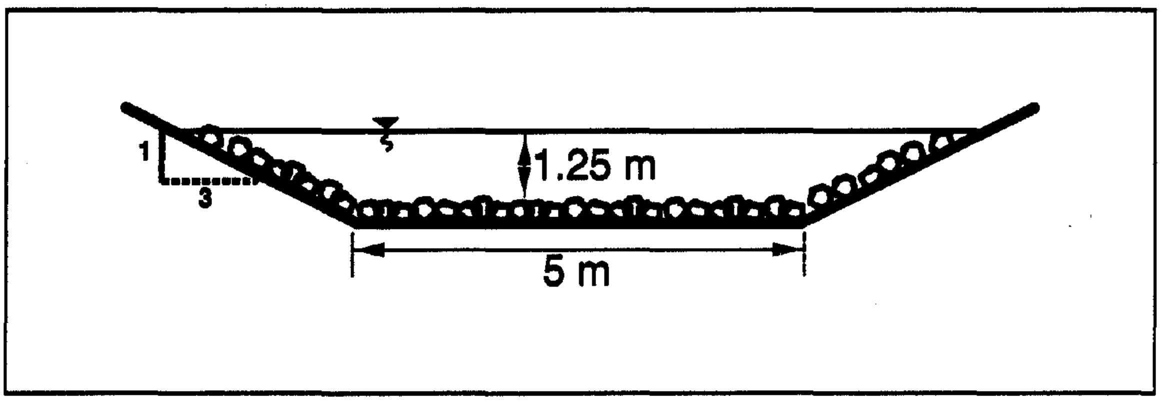
V = Mean velocity in the vertical with a depth of y_0;

The other terms have been defined previously.

E. Shear Stress Problem

Determine the shear stress along the wetted perimeter of a trapezoidal channel. Also determine the shear stress on a particle along the bottom of the same channel.

Given: Trapezoidal channel as illustrated with $S_o = 0.005$, $\gamma = 9{,}800\ N/m^3$, $V = 1.8\ m/s$, $D_{84} = 0.15\ m$



Find: (1) τ_o along wetted perimeter; and (2) τ_o along bed

Solution: (1) Shear stress along the wetted perimeter is given by

$$\tau_o = \gamma RS$$

(2) Shear stress along the bottom at a point is

$$\tau_o = \frac{\rho V^2}{[5.75 \log (12.27 \frac{y_o}{k_s})]^2}$$

The computed shear stress information can be used to design a channel lining. For example, a riprap lining would have to withstand at least ____ N/m^2 to be stable in this flow condition. The design of channel linings and stable channel conditions is detailed in HEC-15, "Design of Roadside Channels with Flexible Linings" and will be discussed in Lesson 5.

FROUDE NUMBER

- Froude number is used in many formulas. It is

$$Fr = \frac{V}{\sqrt{gy}}$$

- When Fr = 1.0, flow is critical
- When Fr < 1.0, flow is subcritical
- When Fr > 1.0, flow is supercritical

4.28 Froude Number

IX. Froude Number and Relationship to Subcritical, Critical, and Supercritical Flow

A. The Froude Number is an important dimensionless parameter in open-channel flow. It is normally expressed as

$$Fr = \frac{V}{\sqrt{gy}}$$

where

Fr = Froude Number
V = Velocity of flow, m/s
g = Acceleration of gravity, m/s^2
y = Depth of flow, m

B. V and y can be the mean velocity and depth in a channel or the velocity and depth in the vertical. If the former are used, then the Froude Number is for the average flow conditions in the channel. If the latter are used, then it is the Froude Number for that vertical at a specific location in the cross section. The Froude Number uniquely describes the flow pattern in open-channel flow.

C. When the Froude Number is 1.0, the flow is critical; values of the Froude Number greater than 1.0 indicate supercritical or rapid flow and smaller than 1.0 indicate subcritical or tranquil flow.

D. The velocity and depth at critical flow are called the critical velocity and critical depth. The channel slope which produces critical depth and critical velocity is the critical slope.

SUPERCRITICAL FLOW
(Fr > 1.0)

- **Can be difficult to control**
- **Waves and cross waves are common**
- **Wave formation can cause channel overtopping**
- **Occurs in steep channels**

4.29 Supercritical flow

E. Supercritical flow is difficult to control because abrupt changes in alignment or in cross section produce waves which travel downstream, alternating from side to side, sometimes causing the water to overtop the channel sides. Supercritical flow is common in steep flumes, channels, and mountain streams.

SUBCRITICAL FLOW
(Fr < 1.0)

- **Easy to control for Fr < 0.8**
- **Common in flatter channels**

4.30 Subcritical flow

F. Subcritical flow is relatively easy to control for flows with Froude Numbers less than 0.8. Subcritical flow is common in channels, flumes and streams located in the plains regions and valleys where slopes are relatively flat.

CRITICAL FLOW • Critical flow is a hydraulic control • Flow passes through critical between subcritical and supercritical flow, and vice-versa • Critical depth and velocity depend only on channel size and shape and are independent of slope and n value • Critical slope depends on geometry and discharge • Examples of critical flow

4.31 Critical flow

G. Critical depth is important in hydraulic analysis because it is always a hydraulic control. The flow must pass through critical depth in going from subcritical flow to supercritical or going from supercritical flow to subcritical. The change from supercritical to subcritical is often abrupt (particularly if the Fr > 2) resulting in a phenomena known as the hydraulic jump. In subcritical flow the hydraulic control is always located downstream, while in supercritical flow the hydraulic control is located upstream. Typical locations of critical depth are:

1. At abrupt changes in slope when a flat (subcritical) slope is sharply increased to a steep (supercritical) slope.

2. At channel constrictions such as a culvert entrance, flume transitions, etc. under some conditions.

3. At the unsubmerged outlet of a culvert or flume on a subcritical slope, discharging into a wide channel, steep slope channel (supercritical), or with a free fall at the outlet.

4. At the crest of an overflow dam, weir, or embankment.

5. At bridge constrictions where the bridge chokes the flow.

Critical depth and velocity for a particular discharge are dependent on channel size and shape only and are independent of channel slope and roughness. Critical slope depends upon the channel roughness, channel geometry, and discharge. For a given critical depth and velocity, the critical slope for a particular roughness can be computed by Manning's equation.

X. Demonstration 5

Purpose: To use the demonstration flume to illustrate the following concepts:

1. Steady verses unsteady flow and uniform verses non-uniform flow.
2. Manning's equation.
3. Froude number.
4. Subcritical, supercritical, and critical flow conditions

A. Demonstration 5

A steady flow condition will exist after the flume has been turned on and allowed to stabilize at a given discharge. By rapidly raising and lowering the head box sluice gate, an unsteady flow condition will be created. With the sluice gate up, the flume will be allowed to stabilize again creating a steady, uniform flow. Stop logs will then be added to create a steady, nonuniform flow condition. This condition also illustrates gradually varied flow.

NOTES:

B. Demonstration 6

A uniform flow condition will be created on a steep slope. Are the bed, water surface, and energy slopes parallel (equal)? Compare the flow depth and velocity head measurements at up- and downstream locations.

NOTES:

C. Demonstration 7

A steady uniform flow condition will be created in the flume. The class will be divided into three groups and each group will apply different measurements and procedures to estimate the velocity and discharge:

Method 1: Calculate the discharge from bucket/stop watch measurements and the velocity from the continuity equation.

Method 2: Calculate the velocity from the Manning's equation ($n=0.009$) and discharge from the continuity equation.

Method 3: Calculate the velocity from float measurements and discharge from the continuity equation.

On the flip chart paper, write out the necessary equations and your solution. Be prepared to discuss your results and compare your answers with the other groups.

NOTES:

D. Demonstration 8

To illustrate the Froude Number and supercritical and subcritical flow conditions, the broad-crested weir will be inserted into the flow. Each group shall calculate the Froude Number for the approach flow and the flow over the step. Use the flip chart paper and be prepared to discuss your results.

Using your hand, create some waves in both locations. Notice how the surface waves can travel upstream in subcritical flow, but cannot in supercritical flow.

NOTES:

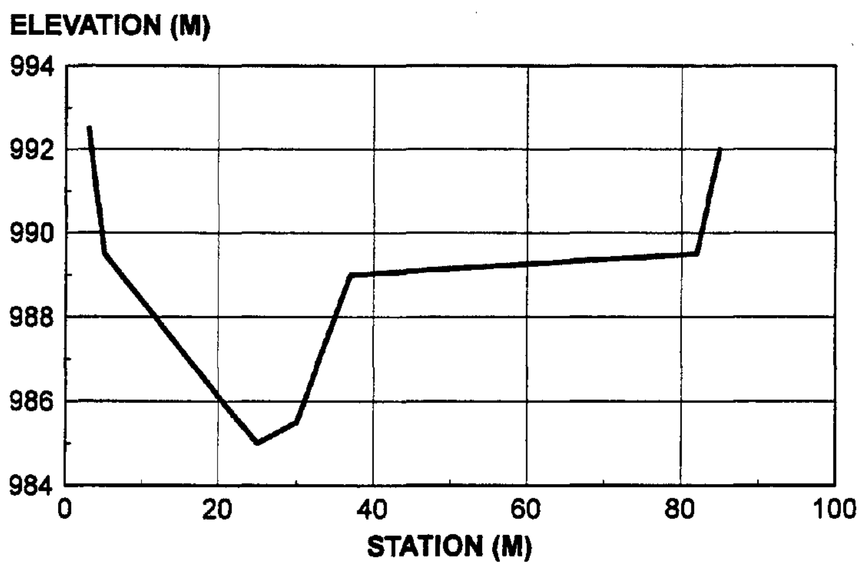
XI. Manning Equation Workshop

Problem Statement

The stage at a several different discharges must be determined for an irregular shaped natural channel adjacent to a highway. The discharge range of interest is from 100-1000 m^3/s. The channel is relatively clean and straight, with some stones and vegetation growth. Based on Table 12, HDS-4, the appropriate Manning's n value is 0.03-0.04. The slope of the channel in this reach is estimated by reference to the contours on a USGS 15 minute quadrangle as 0.07% (0.0007 m/m). Based upon a field survey, the coordinate points of the typical section best representing the conditions at this site are as follows:

Point	Station	Elevation
1	3	992.5
2	5	989.5
3	25	985.0
4	30	985.5
5	37	989.0
6	82	989.5
7	85	992.0

The plotted cross section looks like:



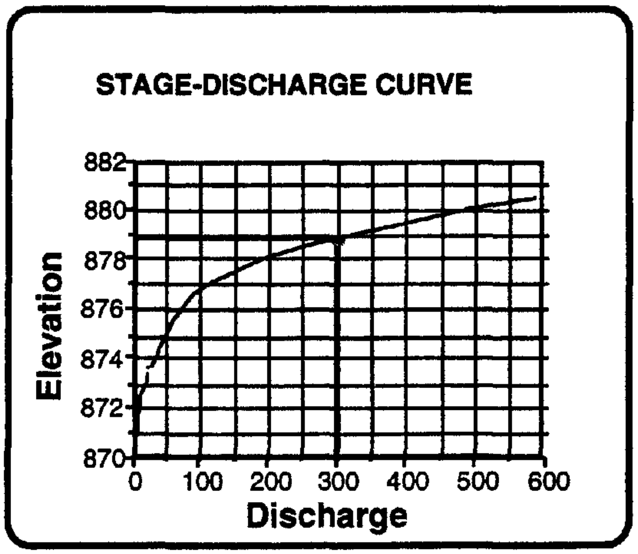
Use the Manning's Equation to develop a stage-discharge curve for this channel location. Begin with an assumed water surface elevation of 986.0 and use 0.5 m elevation increments for additional calculations. The water surface elevations for the various discharges of interest can then be evaluated from the computed stage-discharge curve.

Solution

A. Method

This method is based upon the use of the Manning's Equation, a uniform flow equation involving channel characteristics, water-surface profiles, and roughness coefficients. Manning's Equation was developed to describe uniform flow in which the water surface profile and the energy gradient are parallel to the stream bed. Furthermore, Manning's Equation assumes no change in area, hydraulic radius, nor depth throughout a given reach. Natural channels rarely, if ever, present such a set of circumstances. However, for many practical design problems uniform flow conditions can be assumed and the Manning's equation will provide a reasonable solution.

Given a discharge, Manning's Equation is indeterminate for depth of flow. An iterative solution can be used to calculate the flow depth for a given discharge. Alternatively, a series of assumed depths of flow can be used to directly calculate the corresponding discharges which are then plotted to develop a "stage vs. discharge" curve (see below).



B. Uses and Limitations

- Useful if uniform flow is assumed.
- Simple in application.
- Often used for indirect measurement calculations.
- Results sensitive to water surface slope and roughness coefficient (n-value) selection.
- Often adequate for practical designs/analyses.

C. Manning Equation Combined With Continity Equation:

MANNING'S EQUATION FOR UNIFORM FLOW

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

where: Q = discharge (m^3/s),
n = roughness coefficient (dimensionless),
A = cross-sectional area of flow (m^2),
R = hydraulic radius (A divided by wetted perimeter) (m),
S = longitudinal slope of water surface (m/m).

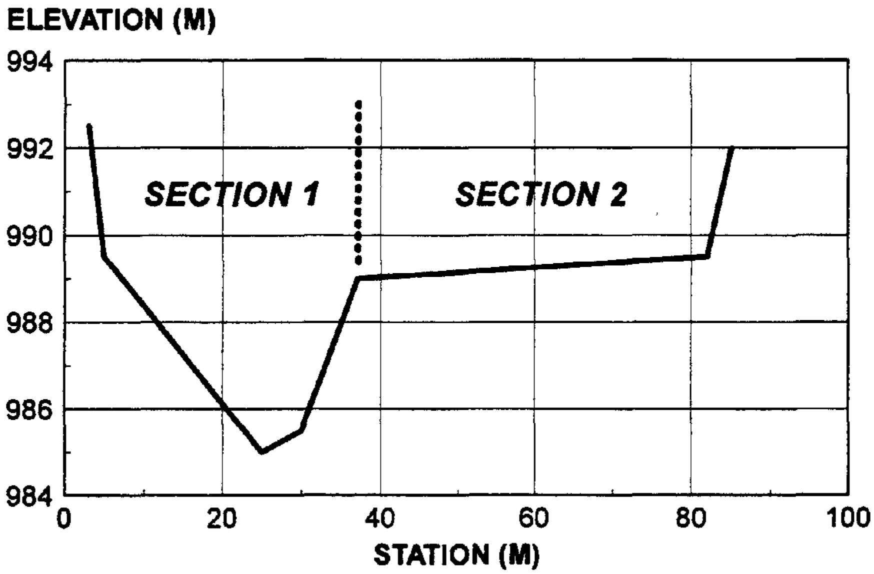
D. Procedure

- Assume a series of water surface elevations representing a range of flow conditions in the channel cross section.
- For each assumed water surface elevation:
  - Calculate the cross-sectional area of flow, wetted perimeter, and hydraulic radius.
  - Select the appropriate n-value (assume 0.04)
  - Apply Manning's Equation to calculate the discharge. When doing the calculations by hand, the use of conveyance can simplify the analysis.
- Tabulate or plot (on linear graph provided below) each pair of assumed water surface elevation/calculated discharge to form a stage-discharge relationship.

	Elevation	A	P	R	K	Q
1	986.0	6.22	10.70			
2	986.5	12.25	14.09			
3	987.0	19.89	17.49			
4	987.5	29.14	20.89			
5	988.0	40.00	24.28			
6	988.5	52.47	27.68			
7	989.0	66.56	31.07			
8	989.5	93.25	78.35			
9	990.0	131.98	79.74			
10	990.5	171.18	81.12			
11	991.0	210.85	82.50			

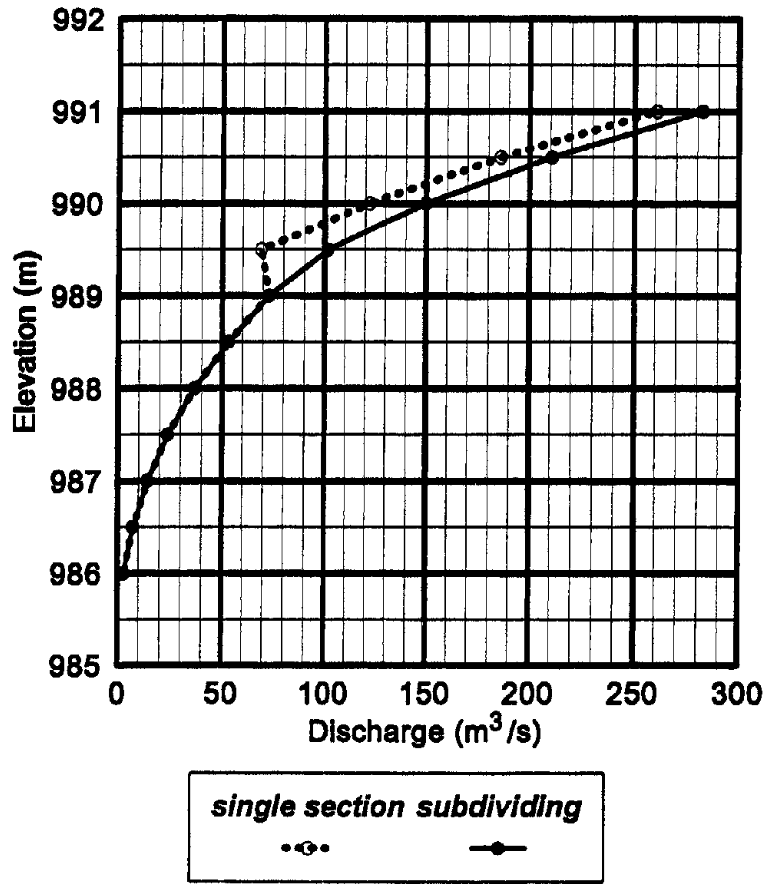
E. Discussion

- Does the resulting stage-discharge curve make sense?
- Recalculate the stage-discharge curve using a subdivided cross section:



	Elev	A(1)	P(1)	R(1)	K(1)	Q(1)	A(2)	P(2)	R(2)	K(2)	Q(2)	Q(t)
1	986.0	6.22	10.70				-	-	-	-	-	
2	986.5	12.25	14.09				-	-	-	-	-	
3	987.0	19.89	17.49				-	-	-	-	-	
4	987.5	29.14	20.89				-	-	-	-	-	
5	988.0	40.00	24.28				-	-	-	-	-	
6	988.5	52.47	27.68				-	-	-	-	-	
7	989.0	66.56	31.07				-	-	-	-	-	
8	989.5	82.00	33.35				11.25	45.00				
9	990.0	98.09	33.95				33.89	45.79				
10	990.5	114.34	34.55				56.84	46.57				
11	991.0	130.76	35.15				80.09	47.35				

F. Plotted solution for single section and subdivided cross section calculations:



LESSON 4

PART 2 - Steady Nonuniform Flow

Introduction to Highway Hydraulics

LESSON 4, Part 2

STEADY NONUNIFORM FLOW

4.32a Lesson 4, Part 2

OBJECTIVES - Part 2

- **Understand basic concepts and calculation procedures for steady nonuniform flow**
  - **rapidly varied versus gradually varied flow**
  - **specific energy and specific discharge**
  - **hydraulic jump**
- **Know how to calculate superelevation in channel bends**

4.32b Part 2 objectives

INTRODUCTION

- **Steady nonuniform flow occurs when Q is constant, but y, V or cross section changes occur**
- **Continuity equation applies at all cross sections**
- **Manning's equation can be used if changes in V are small so that effect of acceleration is small**
- **Steady rapidly varied flow versus steady gradually varied flow**

4.33 Introduction

I. Introduction

A. Steady nonuniform flow occurs when the quantity of water (discharge) remains constant, but the depth of flow, velocity, or cross section changes from section to section.

B. From the continuity equation, the relation of all cross sections will be:

$$Q = A_1 V_1 = A_2 V_2 = A_n V_n$$

C. The Manning's equation can be used if changes in velocity are small so that the effect of acceleration is small.

D. The hydraulic design engineer needs a knowledge of nonuniform flow in order to determine the behavior of the flowing water when changes in channel resistance, size, cross section, shape, or slope occur.

E. There are two basic types of steady nonuniform flow: steady rapidly varied flow and steady gradually varied flow. Steady rapidly varied flow occurs for relatively short distances (a few meters to several hundred meters) where accelerations are more important than friction. Steady gradually varied flow occurs over long distances (50 m to thousands of meters), where friction losses are more important than accelerations.

STEADY RAPIDLY VARIED FLOW

- **Occurs through short transitions where bed elevation or width changes**
- **Energy equation can be used when flow is uniform before and after**
- **If transition causes Fr to approach 1, problems can occur**
- **Analysis aided by specific energy**

4.34 Steady rapidly varied flow

II. Steady Rapidly Varied Flow - General Solution Procedures

A. Steady rapidly varied flow typically occurs through short transitions where bed elevation increases or decreases or flow width expands or contracts.

B. Steady flow through relatively short transitions where the flow is uniform before and after the transition can be analyzed using the energy equation. Energy loss due to friction may be neglected, at least as a first approximation. Refinement of the analysis can be made in a second step by including friction loss.

C. Any time a transition changes velocity and depth such that the Froude Number approaches unity, problems such as waves, blockage, or choking of the flow may occur. If the approaching flow is supercritical, a hydraulic jump may result.

SPECIFIC ENERGY EQUATION

- **The specific energy is the total energy relative to the channel bed:**

$$H = \frac{V^2}{2g} + y = \frac{q^2}{2g\,y^2} + y$$

4.35 Specific energy equation

D. The analysis or design of transitions is aided by the use of the depth of flow and velocity head terms in the energy equation. The sum of the two terms is called the specific energy or specific head, H, and defined as

$$H = \frac{V^2}{2g} + y = \frac{q^2}{2gy^2} + y$$

where

H = Specific energy, m
q = Unit discharge, defined as the discharge per unit width, $m^3/s/m$
g = Acceleration of gravity, (9.81 m/s^2)
y = Depth of flow, m

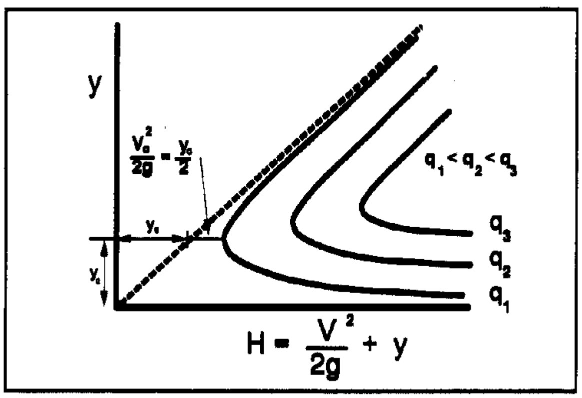
The specific energy H is the height of the total energy above the channel bed.

APPLICATION OF SPECIFIC ENERGY

- **Specific energy diagram**
  - **q constant, H and y variable**
- **Specific discharge diagram**
  - **H constant, q and y variable**

4.36 Application of specific energy

E. The relationship between the three terms in the specific energy equation, q, y, and H, is evaluated by considering q constant and determining the relationship between H and y (specific energy diagram) or considering H constant and determining the relationship between q and y (specific discharge diagram). These diagrams for a given discharge or energy are then used in the design or analysis of transitions or flow through bridges.



4.37 Specific energy diagram

III. Specific Energy Diagram and Evaluation of Critical Depth

A. For a given q, the Specific Energy equation can be solved for various values of H and y. A specific energy diagram is a plot of y verses H for a given q.

B. There are two possible depths called alternate depths for any H larger than a specific minimum. Thus, for specific energy larger than the minimum, the flow may have a large depth with small velocity or small depth with large velocity. Flow for a given unit discharge q cannot occur with specific energy less than the minimum.

C. The single depth of flow at the minimum specific energy is called the critical depth, y_c, and the corresponding velocity, the critical velocity, $V_c = q/y_c$. The relation for y_c and V_c for a given q (for a rectangular channel) is

$$y_c = \sqrt[3]{\frac{q^2}{g}} = 2\frac{V_c^2}{2g}$$

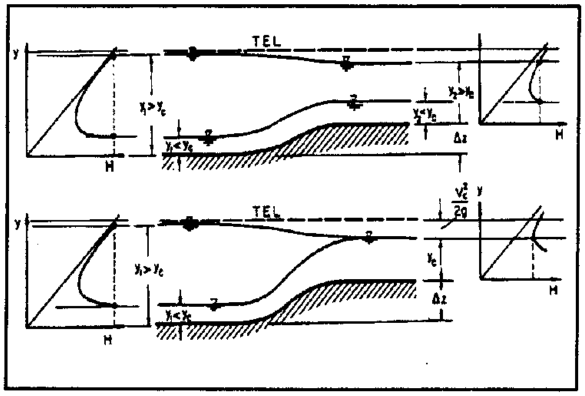
Note that for critical flow:

$$\frac{V_c}{\sqrt{g y_c}} = 1 = Fr$$

and

$$H_{min} = \frac{V_c^2}{2g} + y_c = \frac{3}{2} y_c$$

D. **The specific energy diagram can be used to understand changes in channel depth as the bed elevation decreases or increases.**



4.38 Flow over a step

E. Distinguishing between the types of flow and how the water surface reacts with changes in cross section is important in channel design; thus, the location of critical depth and the determination of critical slope for a cross section of given shape, size, and roughness becomes necessary.

CRITICAL DEPTH RELATIONSHIPS

- **Calculating critical depth and critical slope can be important in drainage analysis**
- **Basic equations for any cross section**

$$\frac{A_c^3}{T_c} = \frac{Q^2}{g} \qquad V_c = \sqrt{g\ y_c} \qquad y_c = \frac{A_c}{T_c}$$

4.39 Critical depth relationships

F. For any channel section, regular or irregular, critical depth may be found by a trial-and-error solution of the following equation:

$$\frac{A_c^3}{T_c} = \frac{Q^2}{g}$$

An expression for the critical velocity (V_c) of any cross section at critical flow conditions is:

$$V_c = \sqrt{g\ y_c}$$

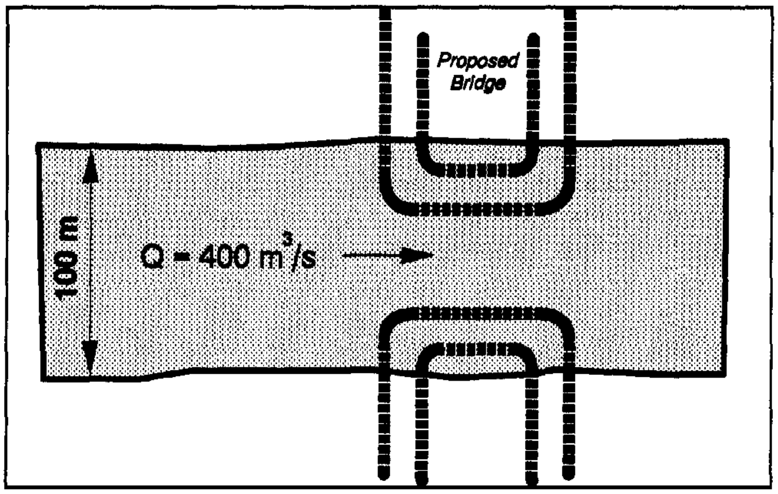
where $y_c = \frac{A_c}{T_c}$

G. Uniform flow within about 10 percent of the critical depth is unstable and should be avoided in design. The reason for unstable flow can be seen by referring to the specific head diagram. As the flow approaches the critical depth from either limb of the curve, a very small change in energy is required for the depth to abruptly change to the alternate depth on the opposite limb of the specific head curve. If the unstable flow region cannot be avoided in design, the least favorable type of flow should be assumed for the design.

NOTES:

H. Specific Energy Problem

Given: A proposed bridge with 1V:1H abutments is to cross a river with trapezoidal cross section with sideslopes of 1V:3H. Since the length of bridge deck is the most costly factor in the design, determine the minimum width of span so as not to cause an increase in depth upstream of the bridge. The channel approaching the bridge carries 400 m^3/s, and is 3 m deep, with a 100 m bottom width.



Find: Minimum channel width at bridge

Solution:

a. As discussed above, the maximum possible contraction occurs at critical flow when the Froude Number is 1.0 and

$$y_c = \frac{2}{3} H$$

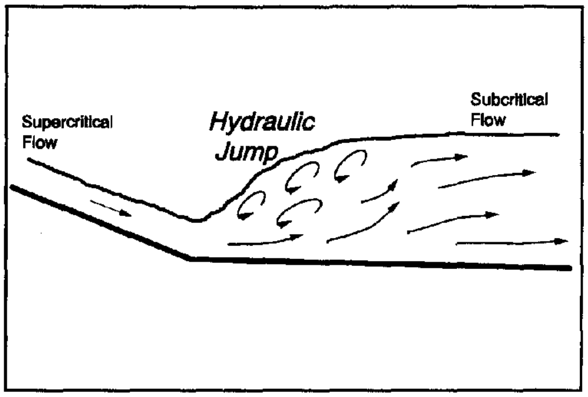
The specific energy immediately upstream of the bridge site is

$$H = y + \frac{Q^2}{2gA^2} =$$

Assuming the specific energy in the contraction caused by the bridge will also be _______ (no energy losses and no bed elevation change), the critical depth will be

y_c =

Substitute y_c back into the energy equation to get minimum width



4.40 Sketch of hydraulic jump

HYDRAULIC JUMP

- **A hydraulic jump occurs when supercritical flow is changed to subcritical flow (sequent depth)**
- **For larger Froude numbers, significant energy dissipation can occur**
  - **Fr < 2 results in an undulating jump**
  - **Fr < 3 results in minimal energy loss**
- **The existence of a jump assumes that adequate tailwater exists**

4.41 Hydraulic jump

IV. Hydraulic Jump

A. A hydraulic jump will occur when the flow velocity V_1 is rapid or supercritical and the slope is decreased to a slope for subcritical flow, or an obstruction such as an energy dissipater is placed in the flow. The supercritical depth is changed to a subcritical depth, called the sequent depth.

B. Depending on the magnitude of the Froude Number, a considerable amount of energy is changed to heat. The larger the Froude Number, the more energy that is lost. The existence of a jump assumes adequate tailwater conditions exist.

C. Many engineers/designers assume that a jump will <u>always</u> occur when a change from a steep grade to a flat grade is encountered, such as near the outlet end of a culvert (i.e., a broken-back culvert). The jump will only occur with adequate downstream tailwater to maintain the sequent depth just below the culvert grade break. Without adequate tailwater, the jump will be swept downstream out of the culvert, causing a potentially large scour hole at the culvert outlet.

D. Video on hydraulic jump.

HYDRAULIC JUMP EQUATIONS

- **For rectangular, flat channels**

$$\frac{y_2}{y_1} = \frac{1}{2} \{(1 + 8\ Fr_1^2)^{1/2} - 1\}$$

- **Corresponding energy loss is**

$$h_L = \frac{(y_2 - y_1)^3}{4\ y_1\ y_2}$$

4.42 Hydraulic jump equations

E. The relation between the supercritical depth and the sequent depth for a rectangular flat channel is

$$\frac{y_2}{y_1} = \frac{1}{2} \{(1 + 8\ Fr_1^2)^{1/2} - 1\}$$

F. The corresponding energy loss in a hydraulic jump is the difference between the two specific energies. It can be shown that this headloss is

$$h_L = \frac{(y_2 - y_1)^3}{4\ y_1\ y_2}$$

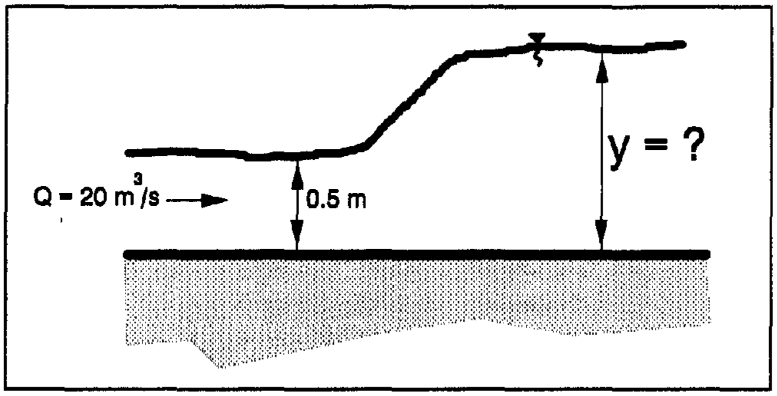
G. These equations have been experimentally verified along with the dependence of the jump length L_j and energy dissipation (headloss h_L) on the Froude Number of the approaching flow (Fr_1).

H. When the Froude Number for rapid flow is less than 2.0, an undulating jump with large surface waves is produced. The waves are propagated for a considerable distance downstream.

I. When the Froude Number of the approaching flow is less than 3.0, the energy dissipation of the jump is not large and jets of high velocity flow can exist for some distance downstream of the jump. These waves and jets can cause erosion a considerable distance downstream of the jump. For larger values of the Froude Number, the rate of energy dissipation in the jump is very large.

J. Hydraulic Jump Problem

Given: A hydraulic jump occurs in a 5m wide rectangular channel at a flow depth of 0.5 m. Determine the downstream water surface elevation needed to cause the jump. Also calculate the headloss due to the jump. Given Q = 20 m^3/s.



Find: (1) Determine the required downstream WSEL to initiate a jump
(2) Determine the headloss across the jump

Solution: (1) Using the formula for a hydraulic jump, find y_2.

$$y_2 = \frac{y_1}{2}\left(\sqrt{1 + 8Fr_1^2} - 1\right) \qquad Fr_1 = \frac{V}{\sqrt{gy_1}} =$$

(2) Find the headloss, h_L, across the jump

$$h_L = \frac{(y_2 - y_1)^3}{4(y_2)(y_1)}$$

SUBCRITICAL FLOW IN BENDS

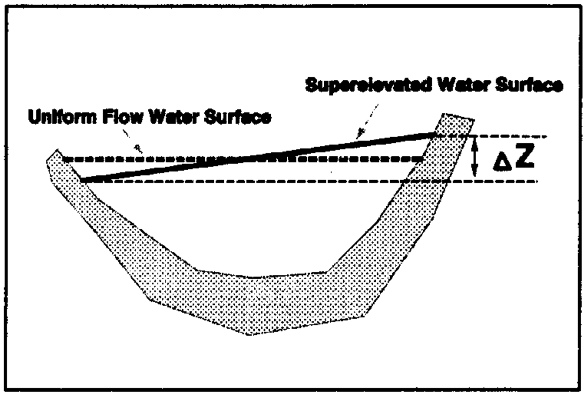
- Superelevation of water surface occurs around a bend
- For natural channels

$$\Delta Z = Z_0 - Z_i = \frac{V^2}{g\, r_c}(r_0 - r_i)$$

- For lined channels use

$$\Delta Z = \frac{V_{max}^2}{2g}\left\{2 - \left(\frac{r_i}{r_c}\right)^2 - \left(\frac{r_c}{r_0}\right)^2\right\}$$

4.43 Subcritical flow in bends



4.44 Superelevation schematic

V. Subcritical Flow in Bends

A. When subcritical flow goes around a bend, the water surface is elevated on the outside of the bend and lowered on the inside of the bend. For a natural channel the approximate difference in elevation (ΔZ) between the water surface along the sides of the curved channel can be found from:

$$\Delta Z = Z_0 - Z_i = \frac{V^2}{g\, r_c}(r_0 - r_i)$$

where
Z = Elevation of the water surface, m
V = Average velocity in the channel, m/s
g = Acceleration of gravity, (9.81 m/s^2)
r_c = Radius of curvature to the centerline of the channel, m
r_0 = Radius of curvature to the outside flow line around the bend, m
r_i = Radius of curvature to the inside flow line around the bend, m

B. For lined canals with strong curvature and large velocities, superelevation should be computed using

$$\Delta Z = \frac{V_{max}^2}{2g} \{ 2-(\frac{r_i}{r_c})^2-(\frac{r_c}{r_0})^2 \}$$

C. Superelevation is accounted for in design by adding one-half of the superelevation to the normal depth.

D. Other problems introduced by curved alignment of channel in subcritical flow include spiral flow, changes in velocity distribution, and increased friction losses within the curved channel as contrasted with the straight channel.

SUPERCRITICAL FLOW IN BENDS

- **In supercritical flow changes in alignment are difficult**
- **Water can "climb out" of bends**
- **Waves disturbances can continue for long distances downstream**
- **If possible, changes in alignment should be made before supercritical flow develops**
- **Model testing may be necessary**

4.45 Superelevation in supercritical flow

VI. Supercritical Flow in Bends

A. Changes in alignment of supercritical flow are difficult to make. The water traveling at supercritical velocities around bends builds up waves which may "climb out" of the channel. The waves that are set up may continue downstream for a long distance. Also, sharp changes in alignment may set up a hydraulic jump with the flow overtopping the banks.

B. Changes in alignment, whenever possible, should be made near the upper end of the section before the supercritical velocity has developed. If a change in alignment is necessary in a channel carrying supercritical flow, the channel should be rectangular in cross section, and preferably enclosed.

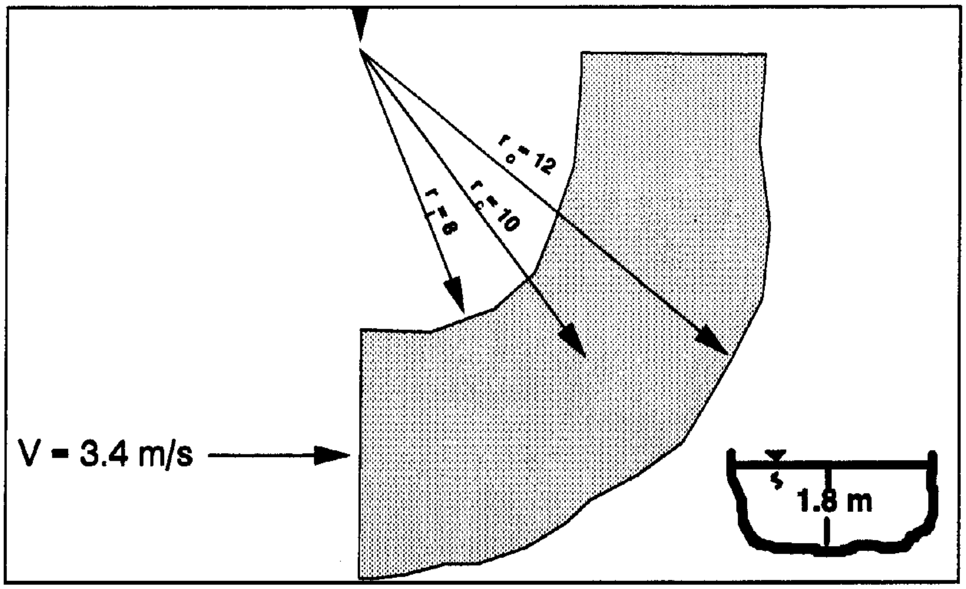
C. On small chutes the angular variation (α) of rectangular flow boundaries (expansion) should not exceed that produced by the equation:

$$\tan\alpha = \frac{1}{3Fr}$$

D. Changes in alignment of open channels can and should be designed to reduce the wave action, resulting from the change in direction in flow. Often designs involving supercritical flow should be model tested to develop the best design, or even a design that will work.

E. Superelevation Problem

Given: During high runoff, a 2.0 m deep mountain stream flows near bank full with a normal depth and velocity of 1.8 m and 3.4 m/s, respectively. At a sharp bend $r_o = 12$, $r_c = 10$, $r_i = 8$. Will flow overtop the bend?



Find: ΔZ

Solution: Use the superelevation formula

$$\Delta Z = \frac{V^2}{g r_c}(r_o - r_1) =$$

The water surface raises approximately (________) m on the outside of the bend and lowers by that same amount on the inside of the bend. The maximum flow depth in the bend will be

$y_{outside}$ =

GRADUALLY VARIED FLOW

- Changes in depth and velocity occur slowly over large distances
- Analysis involves
  - sketching the water surface
  - calculating the water surface elevation
- The flow depth will be either larger or smaller than the normal depth, and larger or smaller than critical depth

4.46 Gradually varied flow

VII. Gradually Varied Flow

A. The second basic type of steady nonuniform flow is gradually varied flow. In gradually varied flow, changes in depth and velocity take place slowly over large distances, resistance to flow dominates, and acceleration forces are neglected.

B. Analysis of gradually varied flow involves: (1) the determination of the general characteristics of the water surface; and (2) the elevation of the water surface or depth of flow.

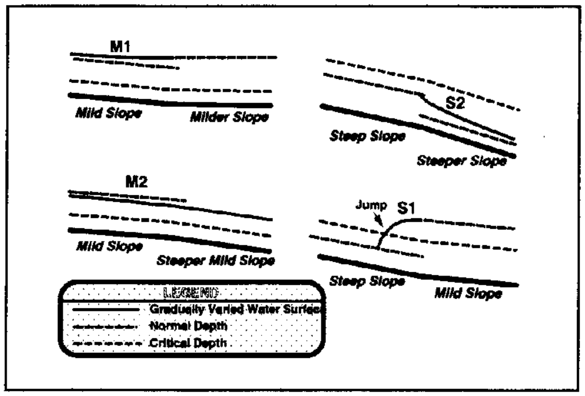
C. In gradually varied flow, the actual flow depth, y, is either larger or smaller than the normal depth, y_o, and either larger or smaller than the critical depth, y_c. The water surface profiles, which are often called backwater curves, depend on the magnitude of the actual depth of flow, y, in relation to the normal depth, y_o, and the critical depth, y_c.

D. In working with gradually varied flow, the first step is to determine what type of water surface profile would exist. The second step is to perform the numerical computations.

GRADUALLY VARIED FLOW
Types of Profiles

- **12 basic types of profiles grouped into 5 classes**
- **Sketching the profile requires locating control points**
- **In supercritical flow the control is upstream; in subcritical flow the control is downstream**

4.47 Types of profiles



4.48 Examples of profiles

VIII. Gradually Varied Flow - Types of Water Surface Profiles

A. There are 12 types of water surface profiles. The 12 types are subdivided into 5 classes which depend on the bed slope. Slide 4.48 illustrates several of the more common profiles.

B. When there is a change in cross-section or slope or an obstruction to the flow, the qualitative analysis of the flow profile depends on locating the control points, determining the type of water surface profile upstream and downstream of the control points, and then sketching these profiles.

C. It must be remembered that when flow is supercritical (Fr > 1), the control depth is upstream and the water surface profile analysis proceeds in the downstream direction. When flow is subcritical (Fr < 1), the control depth is downstream and the computations must proceed upstream (a control is a location where the discharge and stage are uniquely defined).

SKETCHING A PROFILE

- **Supercritical flow can become subcritical only with a hydraulic jump**
- **Location of jump depends on relationship of normal and sequent depth**
- **Subcritical flow must pass through critical depth to become supercritical**

4.49 Sketching a profile

D. In general, when the flow is rapid (Fr > 1), the flow cannot become tranquil or subcritical without a hydraulic jump occurring. In contrast, subcritical flow can become rapid, or supercritical, (cross the critical depth line).

E. The location of the hydraulic jump is determined by evaluating the normal depth and sequent depth conditions (see HDS-4, Example 4.9).

GRADUALLY VARIED FLOW
Calculating the Profile

- **Standard step method**
- **Computer programs used include:**
  - **WSPRO**
  - **HY8**
  - **HEC-2**
- **Qualitative sketching is useful, particularly for complex profiles**

4.50 Calculating a profile

IX. Gradually Varied Flow - Overview of Calculation Procedure

A. **The standard step method** is a simple computational procedure to determine the water surface profile in gradually varied flow. Prior knowledge of the type of water surface profile as determined in the preceding section would be useful to determine whether the analysis should proceed upstream or downstream.

B. The standard step method is derived from the energy equation.

C. Although computer programs such as WSPRO and HY8 are now used to compute water surface profiles, it is recommended that a qualitative sketch of the water surface profiles be made using the information given in the preceding section. This is particularly useful in complicated profiles where the channel slopes change from steep to mild or mild to steep.

X. Demonstration

Purpose: To use the demonstration flume to illustrate the following concepts:

1. Specific energy.

2. Flow over a step and through a contraction.

3. Hydraulic jump.

A. Demonstration 9

A specific energy curve will be graphically constructed by varying the flume slope for a given discharge setting to create flows ranging from subcritical to supercritical. The specific energy will be defined by the water rise in a pitot tube relative to the channel bottom. Using graph paper, a plot the depth of flow on the y axis, and the pitot tube height on the x axis will be created for each different slope setting. The resulting curve defines the specific energy curve for the given unit discharge.

Notice at the point of critical flow, the velocity head is one-half the flow depth, and the minimum specific energy is 1.5 times the critical depth.

For a given subcritical flow setting, a step will be placed on the flume bottom. Notice how the water depth over the step decreases. The flow will then be changed to supercritical by increasing the slope. The water depth over the step now increases. Both conditions can be explained by the specific energy concept.

Now, with the step removed a contraction in width will be inserted into the flume. For supercritical flow conditions, the flow depth in the contraction increases, while for subcritical flow the flow depth decreases. Both conditions can be explained by the specific discharge concept.

NOTES:

B. Demonstration 10

A highly supercritical flow condition will be created with a sluice gate at the upstream end of the flume. By adjusting the tailwater conditions, both strong and weak hydraulic jumps will be created. For each condition, the Froude number before and after the jump will be calculated based on measured data. Note the differences between the upstream and downstream Froude numbers for each jump condition.

NOTES:

LESSON 5

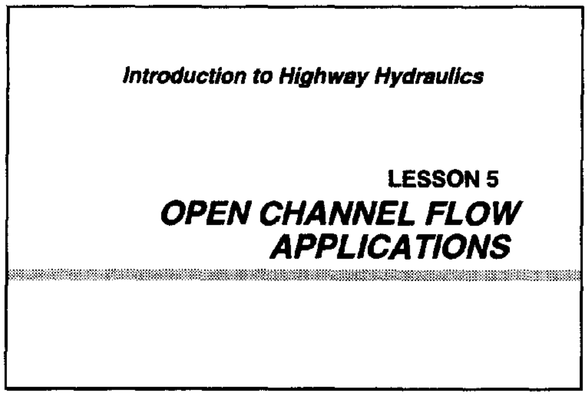
OPEN-CHANNEL FLOW APPLICATIONS

OVERVIEW: Method of Instruction: Lecture and Workshop

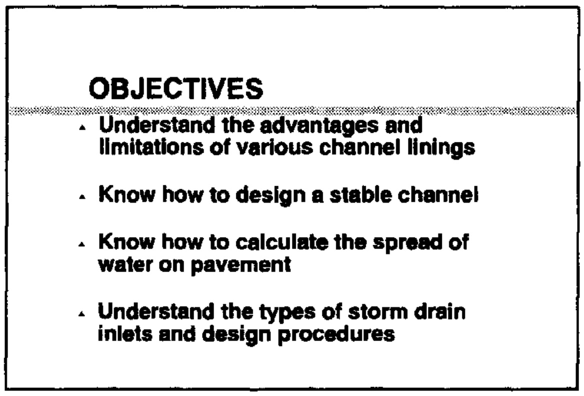
Lesson Length:	Stable Channel Design	-	60 minutes
	Pavement Drainage Design	-	60 minutes
	Application Problem	-	30 minutes

Resources:

Lesson Outline
Slides
HDS-4, Chapter 5



5.0 Title



5.1 Objectives

OBJECTIVES: At the conclusion of this lesson, the Participant should:

1. Understand the advantages and limitations of various channel linings.

2. Know how to design a stable open channel.

3. Know how to calculate the spread of water on pavement.

4. Understand the types of storm drain inlets and design procedures.

LESSON 5

OPEN-CHANNEL FLOW APPLICATIONS

Part 1: Stable Channel Design

OPEN CHANNEL FLOW APPLICATIONS

- **Part 1 : Stable Channel Design**
- Part 2 : Pavement Drainage Design
- Part 3 : Design Problem

5.2 Open-channel flow applications

GENERAL DESIGN CONCEPTS

- **Channel capacity depends on shape, size, slope, and roughness**
- **Most efficient shape is a semi-circle; other factors include:**
  - construction cost
  - maintenance requirements
  - safety
  - channel outlet conditions
- **Preferred cross section**

5.3 General design concepts

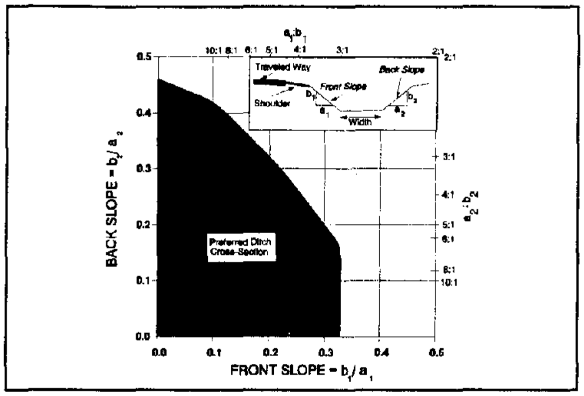
I. General Design Concepts

A. The capacity of a drainage channel depends upon its shape, size, slope, and roughness.

B. The most efficiently shaped channel is a semi-circle flowing full; however, other factors often control geometric shape:

-construction cost
-maintenance requirements (e.g. steeper than 1V:3H difficult to mow)
-safety when vehicles accidentally leave the roadway
-discharge of collected water without damage to the adjacent property

Most of these additional requirements for drainage channels reduce the hydraulic capacity of the channel. Thus, the best design for a particular section of highway is a compromise among the various requirements.



5.4 Preferred geometric cross section

GENERAL DESIGN CONCEPTS (cont)

- **Within constraints of the ROW, avoid sharp changes in alignment/grade**
- **Standardized design for any distance is not necessary**
- **Hydraulic performance based on Lesson 4 concepts**
- **Maintenance is essential**

5.5 General design concepts (cont)

C. The width of the right-of-way usually allows little choice in the alignment or in the grade of the channel, but as far as practicable, abrupt changes in alignment or in grade should be avoided. Sharp changes in alignment present a point of attack for the flowing water, and abrupt changes in grade cause deposition of transported material when the grade is flattened or scour when grade is steepened.

D. It is unnecessary to standardize the design of roadway drainage channels for any length of the highway. Channel width and depth can change to create the most cost efficient channel.

E. Evaluation of the hydraulic conditions in the channel is based on the concepts discussed in Lesson 4.

F. Systematic maintenance is essential to any drainage channel. Maintenance methods should be considered in the design of drainage channels so that the channel sections will be suitable for the methods and equipment that will be used for their maintenance.

STABLE CHANNEL DESIGN CONCEPTS

- **Channel grade typically parallels highway grade, which may create erosive conditions**
- **Channel lining often necessary**
- **Type of lining should be consistent with protection required, cost, safety, habitat enhancement and aesthetic considerations**

5.6 Stable channel design concepts

II. Stable Channel Design Concepts

A. The gradient of roadside channels typically parallels the grade of the highway. Even at relatively mild highway grades, highly erosive hydraulic conditions can exist in adjacent roadside channels. Consequently, designing a stable conveyance becomes a critical component in the design of roadside channels.

B. A channel lining is often necessary to create a stable roadside channel. Lining as applied to drainage channels includes vegetative coverings.

C. The type of lining should be consistent with the degree of protection required, overall cost, safety requirements, and esthetic considerations.

LINING MATERIALS

- **Can be flexible or rigid**
- **Typical flexible linings include grass and riprap; a typical rigid lining is concrete or soil cement**
- **Flexible linings can be either temporary or permanent**
- **Rigid linings are generally permanent**

5.7 Lining materials

III. Lining Materials

A. Lining materials may be classified as flexible or rigid.

B. Typical flexible linings include grass and riprap; a typical rigid lining is concrete or soil cement.

C. Flexible linings can be either permanent or temporary. Temporary flexible linings are commonly used to provide erosion protection until a permanent lining, such as grass, is established. Typical temporary linings include straw mat products and woven geotextiles. Rigid linings are generally considered permanent. Temporary linings are typically designed for a lower return period than used for permanent linings.

FLEXIBLE LININGS-
Advantages

- Less expensive to install and maintain
- Allows infiltration and exfiltration
- Natural appearance
- Provides a safer roadside
- Self-healing
- Provides filtration media for pollutants

5.8 Flexible lining advantages

FLEXIBLE LININGS-
Disadvantages

- Low capacity for a given area
- Flow depth limitation
- Risk of damage during vegetation establishment

5.9 Flexible lining disadvantages

D. Advantages of flexible linings include:

1. Generally less expensive to install and maintain.

2. Allows infiltration and exfiltration.

3. Natural appearance.

4. Provides a safer roadside.

5. Self-healing.

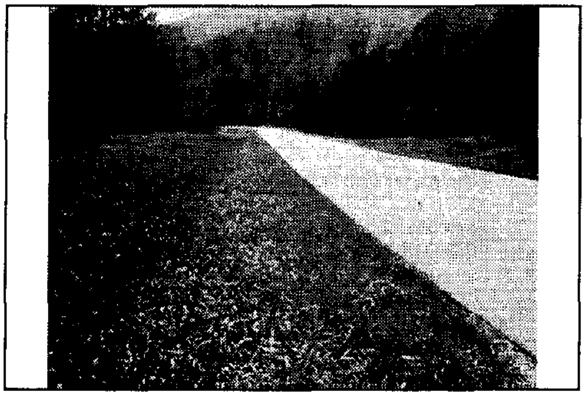
6. Provides a filtering media for runoff contaminants.

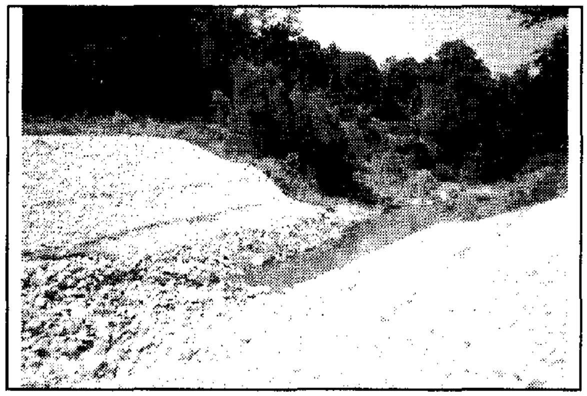
E. Disadvantages of flexible linings include:

1. Low capacity for a given cross sectional area.

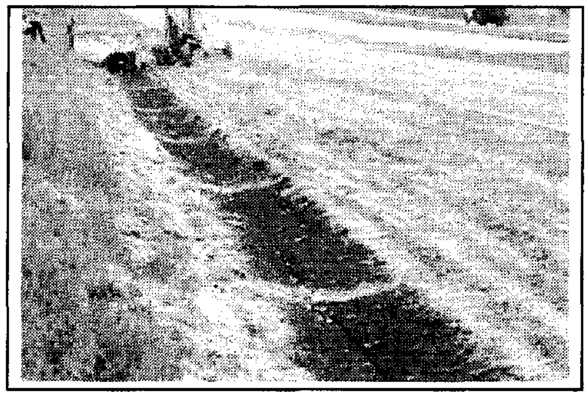
2. Flow depth limitation.

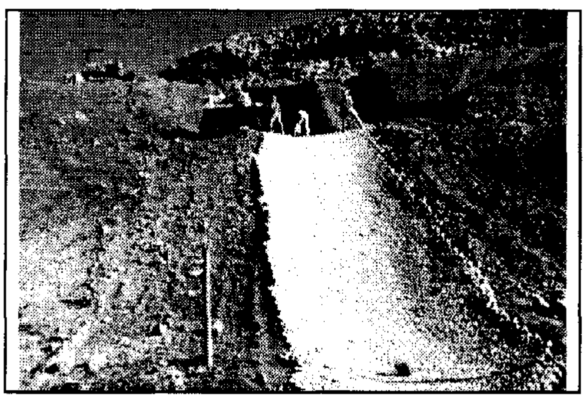
3. Risk of damage during vegetation establishment.


5.10a Vegetative channel lining


5.10b Riprap channel lining


5.10c Straw blanket temporary lining


5.10d Synthetic temporary lining


5.10e Interlocking block lining

RIGID LININGS-
Advantages

- High capacity for a given area
- May construct deep, narrow channels
- Immediate erosion protection

5.11 Rigid lining advantages

RIGID LININGS-
Disadvantages

- Expensive to build and maintain
- High erosive velocities at outlet
- No infiltration or exfiltration
- Unnatural appearance
- Steep walls a safety hazard
- Not self-healing

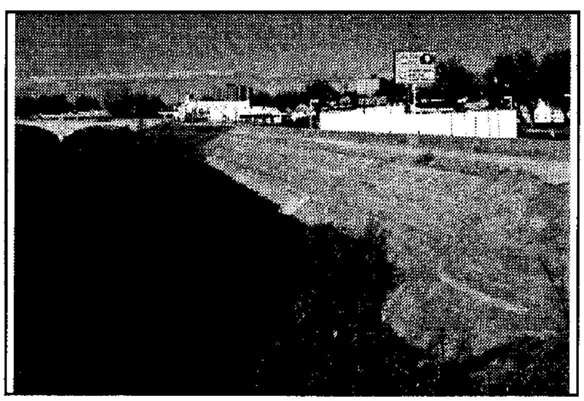
5.12 Rigid lining disadvantages

F. Advantages of rigid linings include:

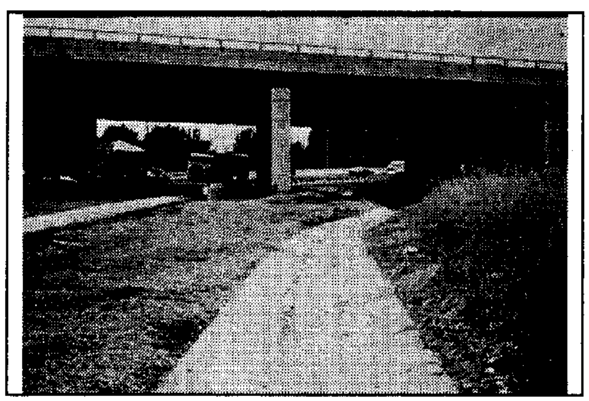
1. High capacity for a given cross sectional area.
2. May construct deep, narrow channels with steep sidewalls.
3. Immediate erosion protection.

G. Disadvantages of rigid linings include:

1. Expensive to build and maintain.
2. High erosive velocities at outlet.
3. No infiltration or exfiltration.
4. Unnatural appearance.
5. Steep walls a safety hazard.
6. Not self-healing.



5.13 Soil cement bank lining



5.14 Concrete-lined swale

SELECTING A LINING MATERIAL

- Select the lowest cost lining that provides satisfactory protection and meets design objectives
- Combining lining types is feasible

5.15 Lining materials

G. In general, when a lining is needed, the lowest cost lining that affords satisfactory protection should be used. Note that the lowest cost lining might involve using different lining types within one project.

NOTES:

EQUILIBRIUM CONCEPTS IN STABLE CHANNEL DESIGN

- **Design based on static or dynamic equilibrium**
- **Static equilibrium exists when boundaries do not move**
- **Dynamic equilibrium exists when boundaries are movable**
- **Most highway channels based on static equilibrium**

5.16 Equilibrium concepts in stable channel design

IV. Equilibrium Concepts in Stable Channel Design

A. Stable channel design can be based on the concepts of static or dynamic equilibrium.

B. Static equilibrium exists when the channel boundaries are essentially rigid and the material forming the channel boundary effectively resists the erosive forces of the flow. Under such conditions the channel remains essentially unchanged during the design flow and the principles of rigid boundary hydraulics can be applied.

C. Dynamic equilibrium exists when the channel boundary is moveable and some change in the channel bed and/or banks occurs. A dynamic system is considered stable as long as the net change does not exceed acceptable levels. Designing a stable channel under dynamic equilibrium conditions must be based on the concepts of sediment transport.

D. For most highway drainage channels bed and bank instability and/or possible lateral migration cannot be tolerated and stable channel design must be based on the concepts of static equilibrium, including the use of a lining material if necessary to achieve a rigid boundary condition.

DESIGNING A STATIC EQUILIBRIUM CHANNEL

- Permissible velocity approach based on maximum allowable velocity
- Permissible tractive force approach based on maximum allowable shear stress
- Tractive force better represents physical processes and was used in HEC-15

5.17 Designing static equilibrium channel

V. Methods for Designing a Static Equilibrium Channel

A. Two available methods are the permissible velocity approach and the permissible tractive force (shear stress) approach.

B. Under the permissible velocity approach the channel is assumed stable if the adopted mean velocity is lower than the maximum permissible velocity for the given channel boundary condition.

C. Similarly, the tractive force approach requires that the shear stresses on the channel bed and banks do not exceed the allowable amounts for the given channel boundary.

D. The tractive force procedure better represents the physical processes occurring and is the method used in HEC-15.

TRACTIVE FORCE PROCEDURE

- **Shear stress on channel boundary is:**

  $\tau_o = \gamma R S$

- **Permissible shear stress defined by HDS-4, Table 13**

5.18 Tractive force procedure

VI. The Tractive Force Procedure

A. The tractive force, or shear stress, is the force water exerts on the bed and bank of a channel as it flows over them. The following equation can be used to determine the average shear stress on the boundary of the channel that results from the force of flowing water.

$$\tau_o = \gamma R S$$

where

τ_o = Average shear stress on the wetted perimeter, Pa

γ = The unit weight of water, N/m^3

R = Hydraulic radius, m

S = Slope of the channel, m/m; in gradually varied flow, the energy slope should be used

The maximum shear stress along the channel bottom may be estimated by substituting the flow depth, y, for the hydraulic radius, R.

TYPICAL PERMISSIBLE SHEAR STRESS VALUES

Temporary	**Permissible Shear Stress (Pa)**
Jute Net	21.6
Straw with Net	69.4
Vegetative	
Long Bermuda Grass	100.5
Short Bermuda Gras	16.8
Riprap	
Gravel (25 mm)	15.8
Rock (300 mm)	191.5

5.19 Permissible tractive force

B. The permissible tractive force for a variety of lining materials is provided by HDS-4, table 13.

C. If the computed maximum tractive force is greater than the permissible tractive force for a given lining material, the channel will not be stable. Calculation of hydraulic geometry conditions can be based on normal flow depth conditions.

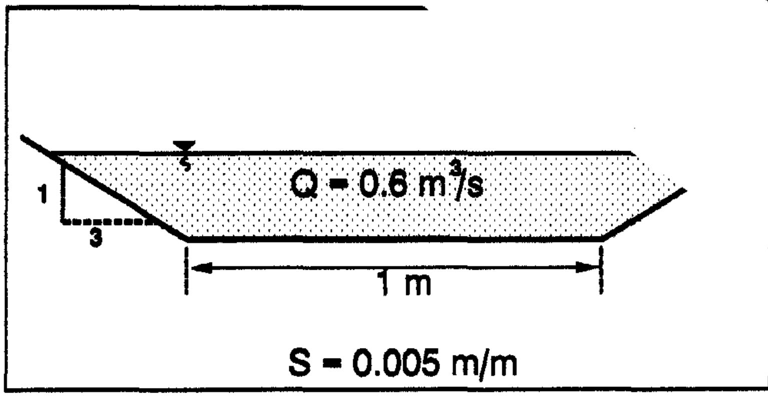
D. HEC-15 details the tractive force stable channel design procedure, including special considerations for steep-slope riprap design and design of composite linings.

E. The HYCHL module of HYDRAIN can be used to analyze channel lining conditions.

VII. Design Problem

Purpose: Determine whether it is feasible to us

Given: $Q = 0.6\ m^3/s$
$S = 0.005$ m/m
Trapezoidal channel with a bott



Find: Depth of flow in the channel and the adequacy of the jute net lining.

Solution:

(1) From table 13, the permissible shear stress is _______ and from table 14, the Manning's n value is _______ (assuming a flow depth between 0.15-0.60 m).

(2) Solving Manning's equation for $S = 0.005$, $Q = 0.6\ m^3/s$, and $B = 1.0$, the flow depth y_s is _______.

(3) The maximum shear stress on the channel bed is:

$$\tau_o = \gamma y S =$$

(4) Comparing the shear stress, _______, to the permissible shear stress, __________, shows that jute net is an acceptable channel lining.

,missible Shear Stresses for Lining Materials (HDS-4, Table 13).[26]

Category	Lining Type	Permissible Unit Shear Stress (lb/ft²)	(Pa)
Temporary*	Woven Pater Net	0.15	7.2
	Jute Net	0.45	21.6
	Fiberglass Roving: Single	0.60	28.7
	Double	0.85	40.7
	Straw with Net	1.45	69.4
	Curled Wood Mat	1.55	74.2
	Synthetic Mat	2.00	95.8
Vegetative**	Class A	3.70	177.2
	Class B	2.10	100.5
	Class C	1.00	47.9
	Class D	0.60	28.7
	Class E	0.35	16.8
Gravel Riprap	25 mm	0.33	15.8
	50 mm	0.67	32.1
Rock Riprap	150 mm	2.00	95.8
	300	4.00	191.5
Bare Soil	Noncohesive	See "Hydraulic Engineering Circular No. 15"[26]	
	Cohesive		

*Some "temporary" linings become permanent when buried.

**A-E refers to retardance class, with Class A vegetation having high retardance and Class E having low retardance. Typical examples include (HEC-15, Table 1):

Retardance Class	Cover	Condition
A	Weeping lovegrass	Excellent stand, tall (76 cm)
B	Weeping lovegrass	Good stand, tall (61 cm)
C	Bermuda grass	Good stand, mowed (15 cm)
D	Bermuda grass	Good stand, cut (6 cm)
E	Bermuda grass	Good stand, cut (4 cm)

Manning's Roughness Coefficients (HDS-4, Table 14).[26]				
Lining Category	Lining Type	n - value		
		Depth Ranges		
		0-0.15 m	0.15 - 0.60 m	>0.60 m
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.021	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	25 mm D_{50}	0.044	0.033	0.030
	50 mm D_{50}	0.066	0.041	0.034
Rock Riprap	150 mm D_{50}	0.104	0.069	0.035
	300 mm D_{50}	--	0.078	0.040

*Some "temporary" linings become permanent when buried.

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

NOTES:

LESSON 5

OPEN-CHANNEL FLOW APPLICATIONS

Part 2: Pavement Drainage Design

OPEN CHANNEL FLOW APPLICATIONS

- Part 1 : Stable Channel Design
- **Part 2 : Pavement Drainage Design**
- Part 3 : Design Problem

5.20 Open-channel flow, Part 2

BASIC CONCEPTS OF PAVEMENT DRAINAGE

- **Water on pavement causes hydroplaning, splash and spray**
- **Pavement drainage design effectively removes water from roadway surface**
- **Curb/gutter flow is a special case of open channel flow**
- **Design based on discharge and spread, according to HEC-22**

5.21 Basic concepts of pavement drainage

I. Basic Concepts of Pavement Drainage

A. Water on the pavement will slow traffic and contribute to accidents from hydroplaning and loss of visibility from splash and spray. Pavement drainage design provides for effective removal of water from the roadway surface.

B. Water flowing on a pavement with a curb is essentially a special case of open channel flow in a shallow, triangular shaped cross section. Pavement drainage to a gutter or swale adjacent to the roadway surface is another case of open channel flow, again typically occurring in a wide shallow cross section.

C. Pavement drainage design is typically based on a design discharge and an allowable spread of water across the pavement.

D. The primary design reference for highway pavement drainage design is HEC-22 and detailed instruction on pavement drainage is available through the NHI Urban Drainage Design Course (NHI Course 13027).

FACTORS INFLUENCING SPREAD

- **Longitudinal grade**
  - **minimum slope for curbed pavements is 0.3%**
  - **rolling profiles necessary in flat terrain**
- **Cross slope**
  - **compromise between driver comfort and drainage considerations**
  - **cross slope is a countermeasure for hydroplaning**

5.22 Factors influencing spread

SPREAD CRITERIA EXAMPLE

ROADWAY CLASSIFICATION	ALLOWABLE SPREAD
LOCAL STREET	*1/2 driving lane*
COLLECTOR < 70 km/h > 70 km/h	 *1/2 driving lane* *Shoulder*
HIGH VOLUME < 70 km/h > 70 km/h	 *Shoulder + 1 m* *Shoulder*

5.23 Typical spread criteria

II. Factors Influencing Pavement Spread

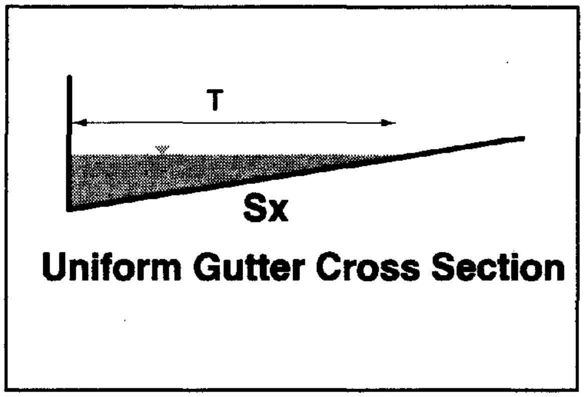
A. Longitudinal grade influences the spread of water onto the pavement. Curbed pavements typically require a minimum slope of 0.3 percent to promote drainage. Minimum grades can be maintained in very flat terrain by use of rolling profiles or by warping the cross slope to achieve a rolling gutter profile.

B. Pavement cross slope is often a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort. Adequate cross slope will reduce water depth on the pavement and, therefore, is an important countermeasure against hydroplaning.

FLOW IN GUTTERS AND SWALES

- Triangular shaped area adjacent to curb defines an open channel section
- Gutters can have a steeper cross slope than pavement

5.24a Flow in gutters and swales



5.24b Gutter flow schematic

III. Flow in Gutters and Swales

A. The triangular shaped area defined by the curb, gutter and the spread onto the pavement creates an open channel flow section for conveying runoff.

MODIFIED MANNING'S EQUATION

- In shallow gutter or swale flow the hydraulic radius does not adequately describe cross section
- Modified Manning's equation:

$$Q = \frac{0.377}{n} Sx^{5/3} S^{1/2} T^{8/3}$$

- Nomographs available for solution

5.25 Modified Manning's equation

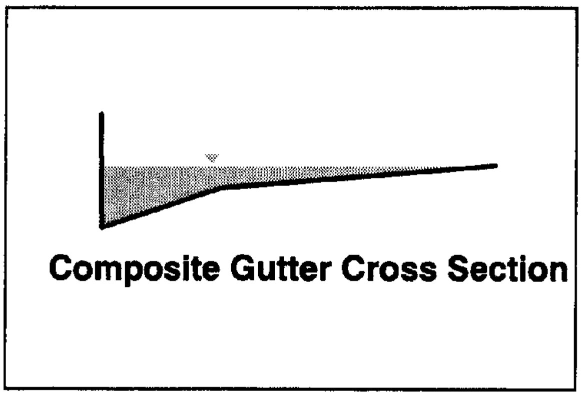
B. Modification of Manning's equation is necessary for use in computing flow in triangular channels because the hydraulic radius in the equation does not adequately describe the gutter cross section, particularly where the top width of the water surface may be more than 40 times the depth at the curb.

$$Q = \frac{0.377}{n} S_x^{5/3} S^{1/2} T^{8/3}$$

where

Q = Discharge, m^3/s
S_x = Cross slope, m/m
S = Longitudinal slope, m/m
T = Spread, m

C. Nomographs for solution of this equation, for both uniform and for the more complex geometry created by composite cross slopes, are provided in HEC-22. HEC-22 also provides nomograph solution of Manning's equation for shallow swale sections. Tabulated flow capacity for standard highway cross sections may also be available from local and regional design guides.



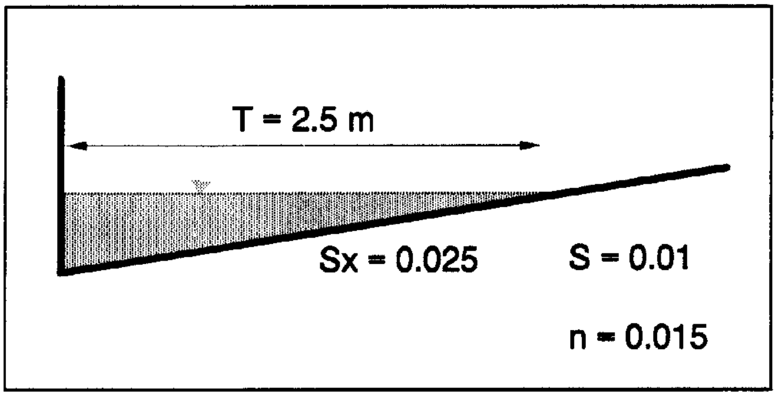
5.26 Composite gutter cross slope

D. Increased capacity is possible with composite cross sections. Gutters adjacent to the curb can have a steeper cross slope from the pavement. This steeper gutter section can be an effective countermeasure for reducing spread on the pavement.

E. Example Problem

Given: A gutter with the following dimensions and conditions

$T = 2.5$ m
$S_x = 0.025$
$S = 0.01$
$n = 0.015$



Find: Flow in gutter at design spread

Solution: Use the modified Manning's equation

$$Q = \frac{0.377}{n} S_x^{5/3} S^{1/2} T^{8/3}$$

PAVEMENT DRAINAGE INLETS

- When spread is too large, runoff must be diverted from the roadway
- All or part of the runoff may be intercepted by inlets connected to a storm drain system

5.27 Pavement drainage inlets

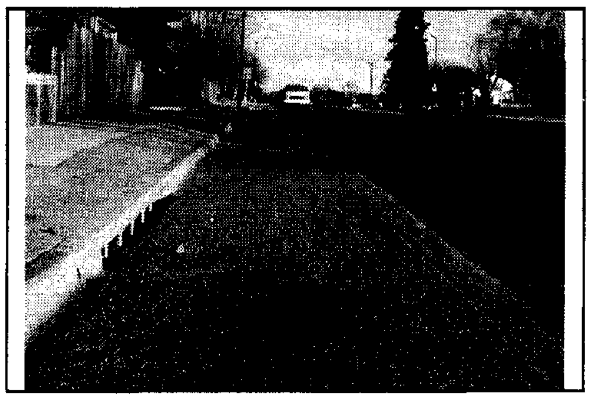
IV. Pavement Drainage Inlets

A. When the capacity of the curb/gutter/pavement section has been exceeded, typically as a result of spread considerations, runoff must be diverted from the roadway surface. A common solution is often interception of all or a portion of runoff by drainage inlets that are connected to a storm drain pipe.

TYPES OF INLETS

- Curb-opening inlets

5.28 Types of inlets



5.29 Curb-opening inlet

B. Inlets used for intercepting runoff from highway surfaces can be divided into four major classes: (1) curb-opening inlets, (2) grate inlets, (3) slotted drains, and (4) combination inlets. Each class has many variations in design and may be installed with or without a depression of the gutter.

TYPES OF INLETS

- Curb-opening inlets
- Grate inlets

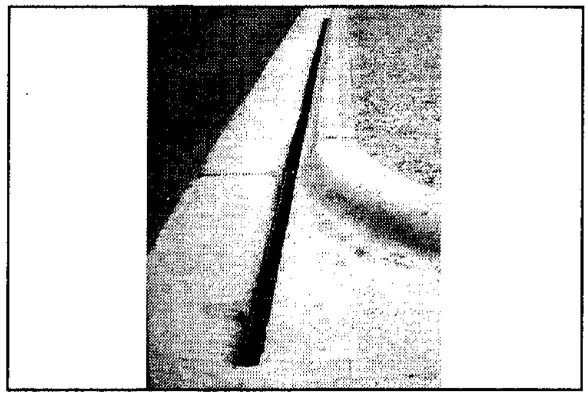
5.30 Types of inlets

5.31 Grate inlet

TYPES OF INLETS

- Curb-opening inlets
- Grate inlets
- Slotted drains

5.32 Types of inlets

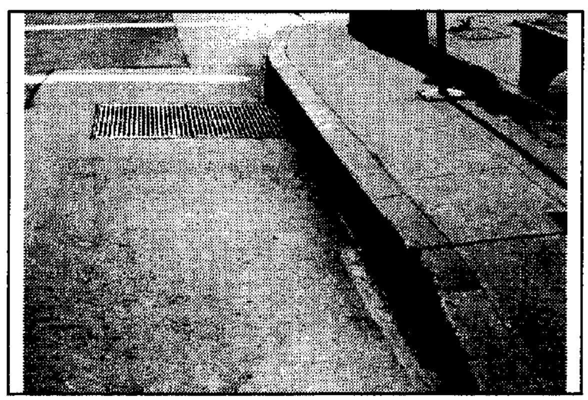


5.33 Slotted drain

TYPES OF INLETS

- Curb-opening inlets
- Grate inlets
- Slotted drains
- Combination inlets

5.34 Types of inlets



5.35 Combination inlet

INLET DESIGN

- Inlet capacity function of
  - type of inlet
  - grate design
  - location (on-grade or sag)
  - gutter design
  - debris clogging
- Weir flow, orifice flow, or combination
- Sag locations vulnerable to debris
- Nomographs or HYDRAIN for design

5.36 Inlet design

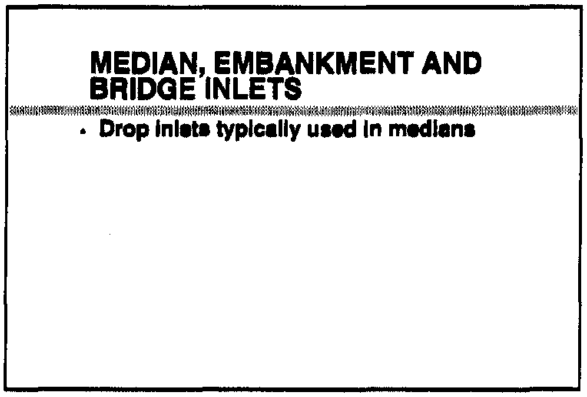
C. Inlet capacity is a function of a variety of factors including type of inlet, grate design, location (on grade or sag location), gutter design, debris clogging, etc.

D. Inlets on continuous grade operate as weir flow, while inlets in sag locations will initially operate as weir flow, but will transition to orifice flow as depth increases.

E. The efficiency of inlets in passing debris is critical in sag locations since all runoff that enters the sag must pass through the inlet, otherwise hazardous ponding condition can result. Grate inlets alone are not recommended in sag locations because of potential clogging.

F. Inlet capacity is typically defined by design charts developed for standard inlet configurations from laboratory testing. HEC-22 provides design charts for a wide range of inlet and grate types typically used in highway engineering.[6]

G. The HYDRA module of HYDRAIN can also be used to analyze pavement drains for grates, curb openings, combinations, and slotted drains.

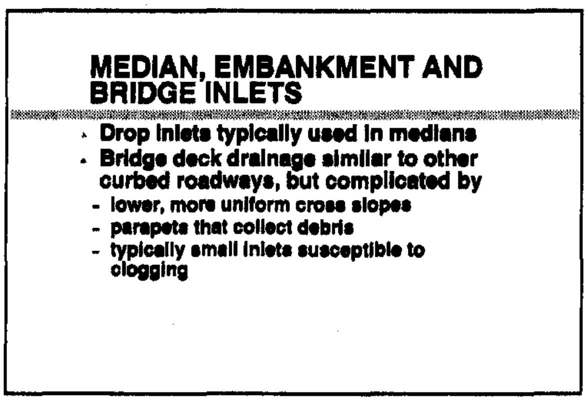


5.37 Median, embankment and bridge inlets

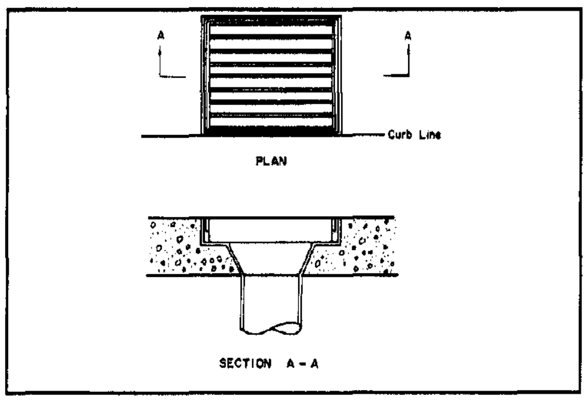
5.38 Median inlet

V. Median, Embankment, and Bridge Inlets

A. Based on capacity or erosion considerations, it is sometimes necessary to place inlets in medians to remove some or all the runoff that has been collected. Medians may be drained by drop (grate) inlets similar to those used for pavement drainage.



5.39 Median, embankment and bridge inlets



5.40 Bridge deck inlet

B. Bridge deck drainage is accomplished in the same manner as any other curbed roadway section, although bridge decks are often less effectively drained because of lower cross slopes, uniform cross slopes for traffic lanes and shoulders, parapets that collect debris, and drainage inlets that are relatively small and susceptible to clogging. Because of the limitations of bridge deck drainage, roadway drainage should be intercepted where practical before it reaches a bridge.

MEDIAN, EMBANKMENT AND BRIDGE INLETS

- **Drop Inlets typically used in medians**
- **Bridge deck drainage similar to other curbed roadways, but complicated by**
  - **lower, more uniform cross slope**
  - **parapets that collect debris**
  - **typically small inlets susceptible to clogging**
- **Inlets upstream of bridges and embankments typically require near total interception**

5.41 Median, embankment and bridge inlets

C. Drainage inlets used to intercept runoff upgrade or downgrade of bridges, or runoff that might endanger an embankment fill slope, differ from other pavement drainage inlets because the economies achieved by system design are not possible because a series of inlets are not used. Also, total or near total interception is necessary and a closed storm drain system is often not available to dispose of the intercepted flow. Intercepted flow is usually discharged into open chutes or pipe downdrains terminating at the toe of the fill slope.

D. Design Problem

Design the length of curb opening inlet required to intercept 0.15 m^3/s flowing along a street with a cross slope of 0.03, and a longitudinal slope of 0.035. Assume an n value of 0.016. Also determine the discharge intercepted if a 3.0 m curb inlet is utilized, and the amount of bypass flow to the next inlet.

Given: $S_x = 0.03$

where
$S = 0.035$
$Q = 0.15\ m^3/s$
$n = 0.016$

Find: (1) Required length for total interception by a curb-opening inlet.

(2) Discharge intercepted by a 3.0 m curb-opening inlet, and the amount of bypass flow to the next inlet.

Solution:

(1) From HDS-4, figure 35, for the given conditions a curb opening inlet ________ long would intercept the total design flow of 0.15 m^3/s.

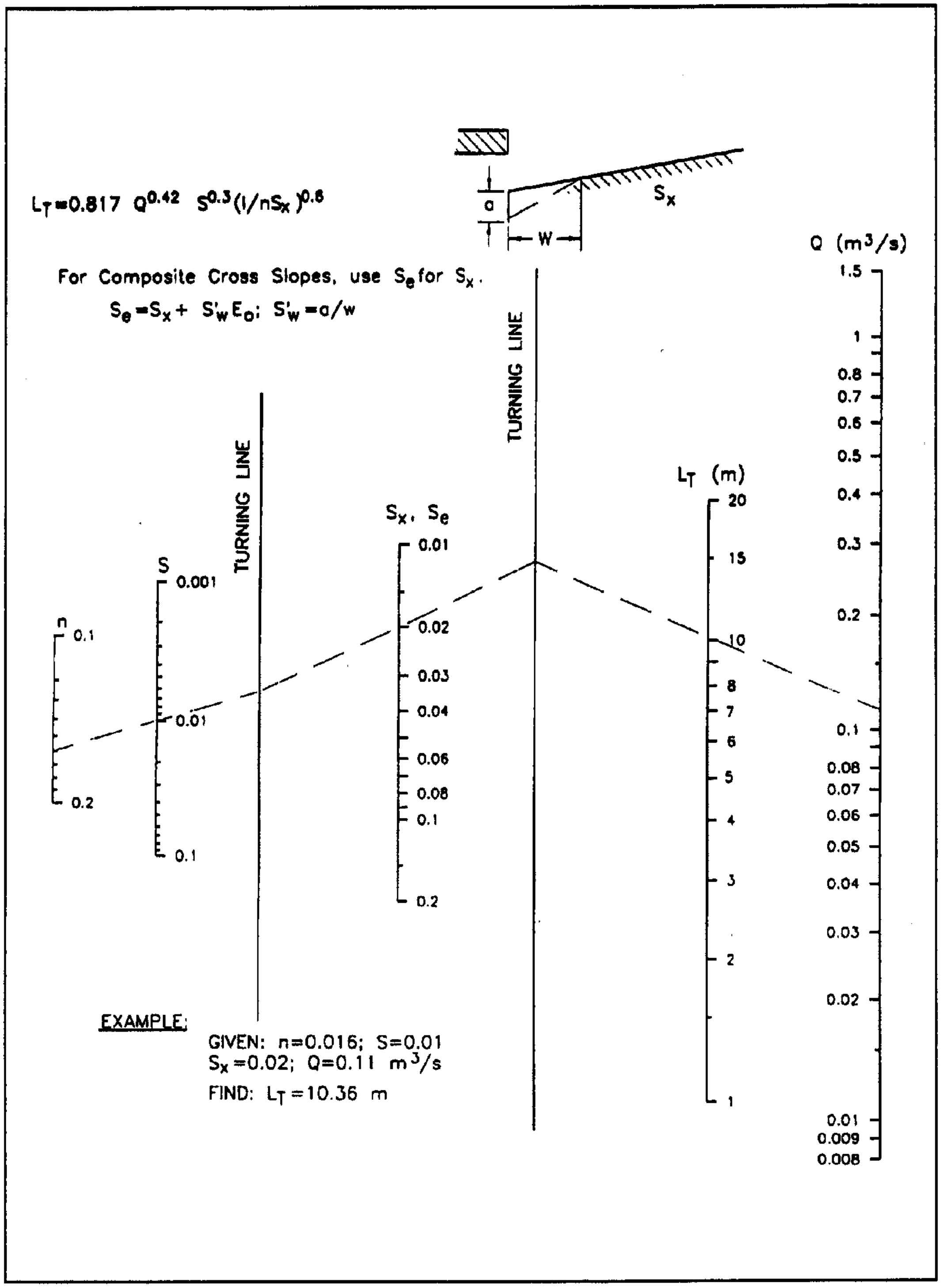
(2) If a 3.0 m curb-opening inlet is used, only a portion of the design flow will be intercepted. HDS-4, figure 36 defines the interception efficiency for curb-opening inlets based on the length for total interception (L_t).

Therefore, given a 13.2 m length for total interception from item (1), and an a curb-opening length of only 3.0 m, the ratio of L/L_t is

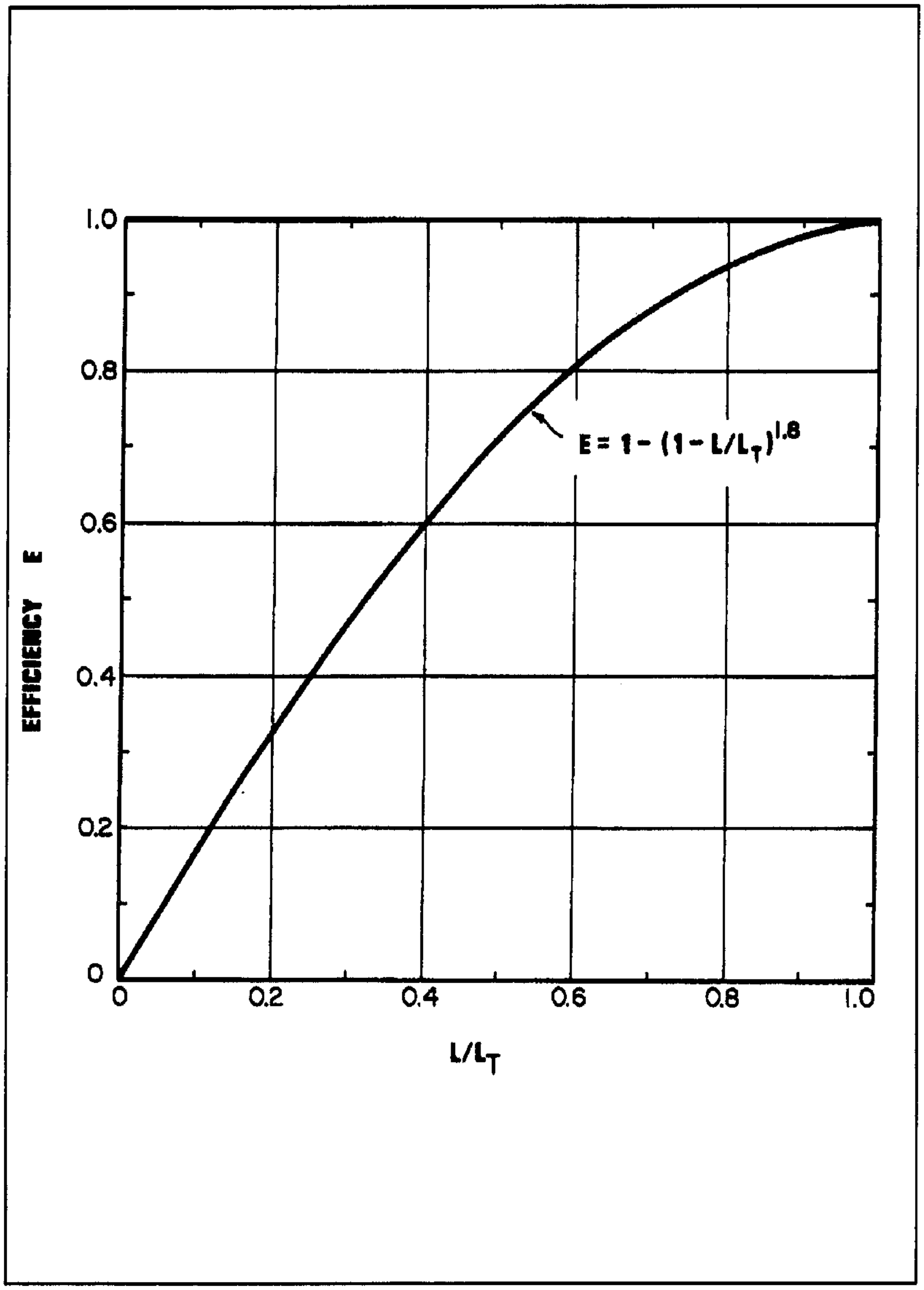
From HDS-4, figure 36 the efficiency, E, is then ______, and the discharge intercepted by a 3.0 m curb-opening inlet is

Qi = EQ =

The amount of bypass flow to the next inlet is



Curb-opening and slotted-drain nomograph (HDS-4, figure 35)



Curb-opening and slotted drain efficiency (HDS-4, figure 36)

NOTES:

LESSON 5

OPEN-CHANNEL FLOW APPLICATIONS

Part 3: Design Problem

OPEN CHANNEL FLOW APPLICATIONS

- Part 1 : Stable Channel Design
- Part 2 : Pavement Drainage Design
- **Part 3 : Design Problem**

5.42 Open-channel flow, Part 3

I. Objective of the Design Problem

The objective of the design problem is to design a permanent channel lining for an intercepting channel above a cut slope on an interstate project. The problem will involve a Rational method calculation to define the 10-yr discharge, sizing the channel, and selecting a channel lining to protect the channel. You will begin working on the problem during the remaining time in class, and complete the problem as a homework assignment.

A secondary objective is to begin getting familiar with the comprehensive example in Lesson 9. The comprehensive example in Lesson 9 will be completed in class on the last day of the course. The intercepting channel being considered today is channel 9, shown on the plan view map for the comprehensive problem. The design of channel 9 will not be part of Lesson 9.

Before you begin designing Channel 9, spend some time reviewing the plan view map, and in particular, the different drainage channels and facilities required for the described roadway project. Note that the topography and conditions shown on the plan view map are illustrated by the schematic drawing shown in figure 1 in HDS-4. If you have not worked with topographic mapping before, a comparison of figure 1 with the plan view map should be beneficial. Developing the ability to read a topographic map and visualize the terrain is necessary for engineering design work.

This homework problem and the comprehensive example will require use of an engineering scale to measure distance from the topographic map. The topographic map is a metric map, and requires the use of a metric scale. For your convenience, a metric scale has been reproduced on paper and is provided as the last page of Lesson 9. Using an engineering scale is another fundamental skill that is necessary for engineering design work.

II. Assumptions and Solution Approach

The intercepting channel number 9 at the top of the cut prevents water from running down the cut slope. The collected water is disposed by distributing it over the hillside, which is preferable whenever the quantity of water is small and the topography and land use permit. A large channel will not be required and the size will probably be governed by construction techniques and equipment. Therefore, for purposes of this design assume a V-bottom ditch with 1V:3H sideslopes.

Calculate the 10-yr design discharge using the Rational method. The tributary area at the outlet of the channel to hillslope can be scaled from the topographic map. Note that the tributary is delineated by the short dashed line. Scale the distances off the map and use a triangular approximation for the area (1/2 base x height). You should get an area of 0.18 ha. The time of concentration should be based on the sum of the overland flow time for water to reach the channel, and the travel time in the channel from the beginning of the channel to the hillslope outlet. Since the channel size will probably be large for the quantity of runoff, shallow flow conditions will exist in the ditch. Therefore, use the shallow flow nomograph (HDS-4, figure 4) to estimate the velocity in the channelized section. Use the IDF curve to define the intensity for the total time of concentration, and a C value based on tabulated values assuming steep turf areas (Note: for your convenience, all required graphs and tables are provided at the end of this example).

After calculating the channel hydraulics from the Manning's equation (given the discharge and design slope), select a lining material based on the tractive force procedure.

III. Solution

A. Rational Method

Given:

A = 0.18 ha (area scaled from map)
S = 0.13 (overland flow slope, scaled from map)

Calculate overland flow t;

$$t = 6.99 \frac{n^{0.6} L^{0.6}}{i^{0.4} S^{0.3}} =$$

Next, calculate the t for the channel, using the shallow concentrated flow nomograph

The total t_c (the sum of the travel times) is:

From the IDF curve, determine the intensity

Apply the Rational formula

B. Manning's Equation

Next, determine the depth of flow in the channel assuming $n = 0.025$.

The Manning's equation is

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

The hydraulic radius is

$$R = \frac{A}{P} =$$

From continuity, the velocity is

C. Calculate Shear Stress

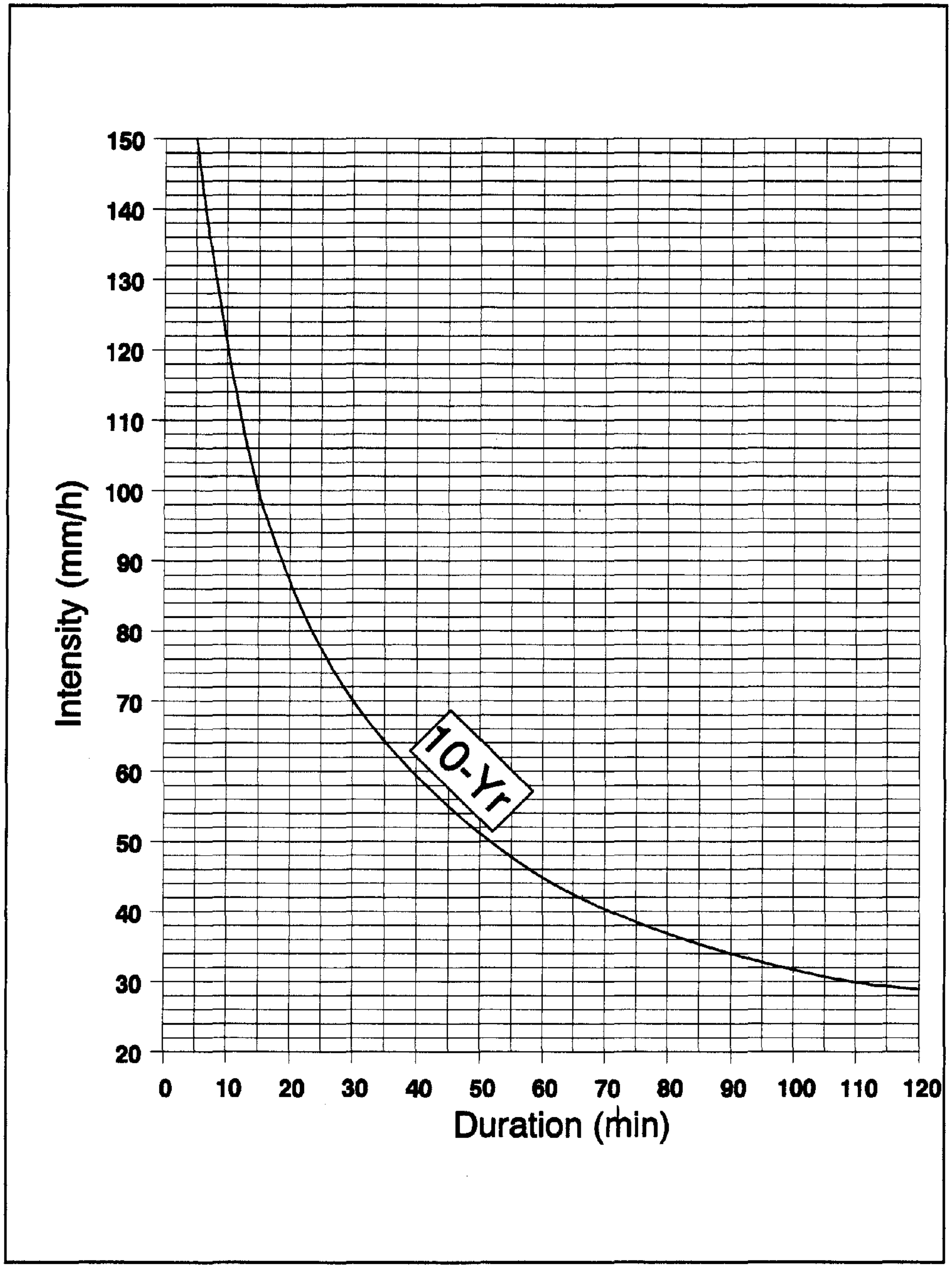
The maximum shear stress is

$$\tau_o = \gamma y S \quad \text{where} \quad \gamma = 9800 \frac{N}{m^3}$$

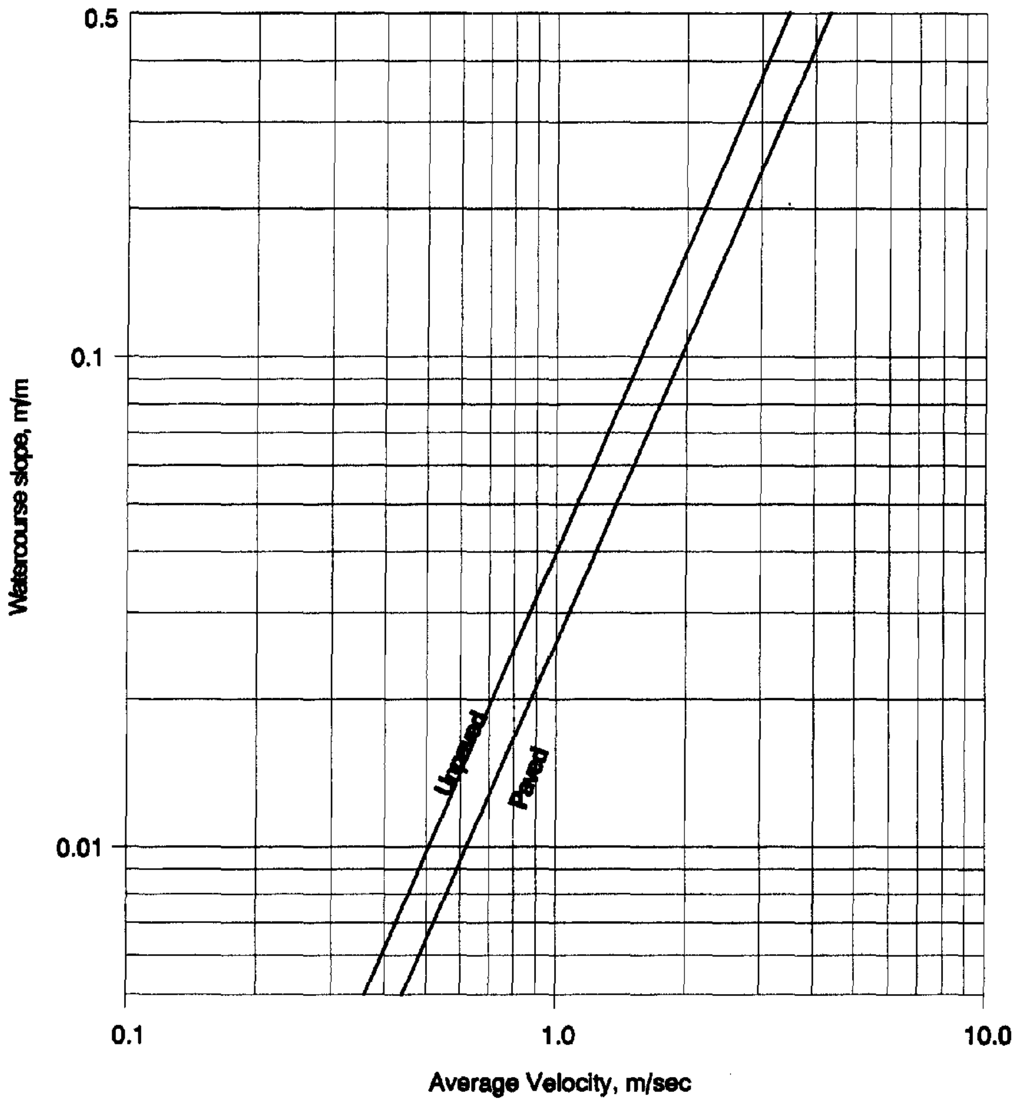
D. Select a lining

Based on the permissible shear stress for various lining materials (HDS-4, table 13), 150 mm rock riprap would be acceptable (τ = 95.8 Pa). However, it is necessary to iterate since the assumed n value (0.025) is not correct for this lining material. Revise the calculation with n=0.104 for 150 mm rock riprap and 0.00 m<y < 0.15 m.

Results of this calculation indicate that 150 mm riprap would not be large enough. Therefore, 300 mm riprap will be required.



Intensity-duration curve



Note: original graph from SCS TR55, 1986; metric conversion completed for HDS-4.

Shallow concentrated Flow Nomograph (HDS-4, figure 4)

Values of Runoff Coefficients, C, for Use in the Rational Method (HDS-4, Table 11).	
Type of Surface	Runoff Coefficient (C^1)
Rural Areas	
Concrete or sheet asphalt pavement	0.8 - 0.9
Asphalt macadam pavement	0.6 - 0.8
Gravel roadways or shoulders	0.4 - 0.6
Bare earth	0.2 - 0.9
Steep grassed areas (2:1)	0.5 - 0.7
Turf meadows	0.1 - 0.4
Forested areas	0.1 - 0.3
Cultivated fields	0.2 - 0.4
Urban Areas	
Flat residential, with about 30 percent of area impervious	0.40
Flat residential, with about 60 percent of area impervious	0.55
Moderately steep residential, with about 50 percent of area impervious	0.65
Moderately steep built up area, with about 70 percent of area impervious	0.80
Flat commercial, with about 90 percent of area impervious	0.80
[1]For flat slopes and permeable soil, use the lower values. For steep slopes and impermeable soil, use the higher values.	

Permissible Shear Stresses for Lining Materials (HDS-4, Table 13).[26]			
Lining Category	Lining Type	Permissible Unit Shear Stress	
		(lb/ft^2)	(Pa)
Temporary*	Woven Pater Net	0.15	7.2
	Jute Net	0.45	21.6
	Fiberglass Roving: Single Double	 0.60 0.85	 28.7 40.7
	Straw with Net	1.45	69.4
	Curled Wood Mat	1.55	74.2
	Synthetic Mat	2.00	95.8
Vegetative**	Class A	3.70	177.2
	Class B	2.10	100.5
	Class C	1.00	47.9
	Class D	0.60	28.7
	Class E	0.35	16.8
Gravel Riprap	25 mm	0.33	15.8
	50 mm	0.67	32.1
Rock Riprap	150 mm	2.00	95.8
	300	4.00	191.5
Bare Soil	Noncohesive	See "Hydraulic Engineering Circular No. 15"[26]	
	Cohesive		

*Some "temporary" linings become permanent when buried.

**A-E refers to retardance class, with Class A vegetation having high retardance and Class E having low retardance. Typical examples include (HEC-15, Table 1):

Retardance Class	Cover	Condition
A	Weeping lovegrass	Excellent stand, tall (76 cm)
B	Weeping lovegrass	Good stand, tall (61 cm)
C	Bermuda grass	Good stand, mowed (15 cm)
D	Bermuda grass	Good stand, cut (6 cm)
E	Bermuda grass	Good stand, cut (4 cm)

Manning's Roughness Coefficients (HDS-4, Table 14).[26]				
Lining Category	Lining Type	n - value		
		Depth Ranges		
		0-0.15 m	0.15 - 0.60 m	>0.60 m
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.021	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	25 mm D_{50}	0.044	0.033	0.030
	50 mm D_{50}	0.066	0.041	0.034
Rock Riprap	150 mm D_{50}	0.104	0.069	0.035
	300 mm D_{50}	--	0.078	0.040

*Some "temporary" linings become permanent when buried.

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

LESSON 6

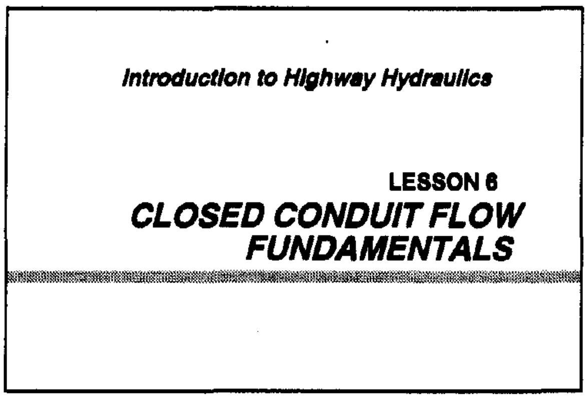
CLOSED-CONDUIT FLOW FUNDAMENTALS

OVERVIEW: Method of Instruction: Lecture

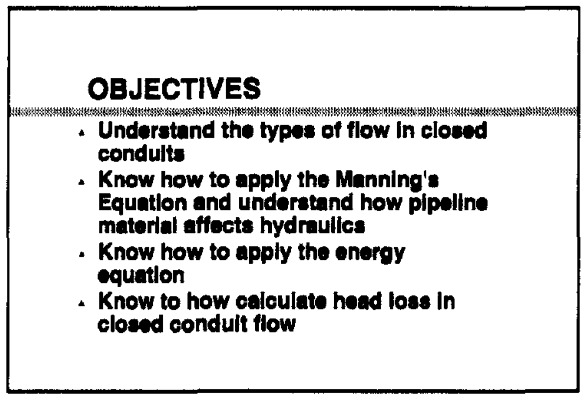
Lesson Length: 45 minutes

Resources:

Lesson Outline
Slides
HDS-4, Chapter 6



6.0 Title



6.1 Objectives

OBJECTIVES: At the conclusion of this lesson, the Participant should:

1. Understand the types of flow in closed conduits
2. Know how to apply the Manning's equation and understand how pipeline material affects hydraulics.
3. Know how to apply the energy equation.
4. Know how to calculate headlosses in closed-conduit flow.

LESSON 6

CLOSED-CONDUIT FLOW FUNDAMENTALS

TYPES OF FLOW IN CLOSED CONDUITS

- **Open channel flow**
- **Gravity full flow**
- **Pressure Flow**

6.2 Type of flow

I. Types of Flow in Closed Conduits

A. Flow conditions in a closed conduit can occur as open-channel flow, gravity full flow or pressure flow.

B. In open-channel flow the water surface is exposed to the atmosphere, which can occur in either an open conduit or a partially full closed conduit.

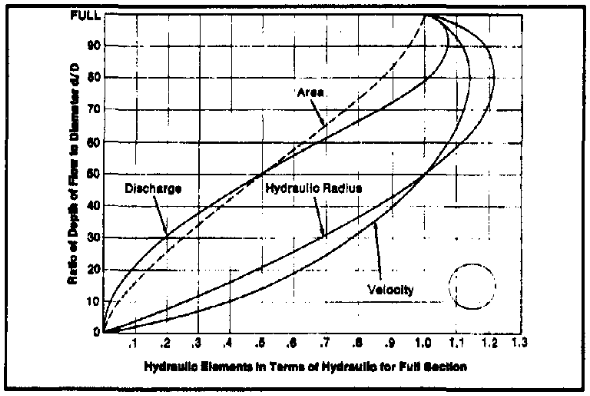
C. Gravity full flow occurs at that condition where the conduit is flowing full, but not yet under any pressure.

D. Pressure flow occurs when the conduit is flowing full and under pressure.

MANNING'S EQUATION FOR PIPE FLOWING FULL

- A partially-full pipe can carry more flow than a gravity full pipe
- Gravity full flow is used for storm drain design

6.3 Manning's equation



6.4 Part-full flow relationships

II. Manning's Equation for Pipe Flowing Full

A. Due to the additional wetted perimeter and increased friction that occurs in a gravity full pipe, a partially full pipe will actually carry greater flow. Gravity full flow condition is usually assumed to simplify hydraulic calculations.

MANNING'S EQUATION FOR PIPE FLOWING FULL

- A partially-full pipe can carry more flow than a gravity full pipe
- Gravity full flow is used for storm drain design
- **Manning's Equation**

$$Q = \frac{0.312}{n} D^{8/3} S^{1/2}$$

6.5 Manning's equation

B. The Manning's equation combined with the continuity equation for circular section flowing full can be rewritten as:

$$Q = \frac{0.312}{n} D^{\frac{8}{3}} S^{\frac{1}{2}}$$

where

Q = Discharge, m^3/s

n = Manning's coefficient

D = Pipe diameter, m

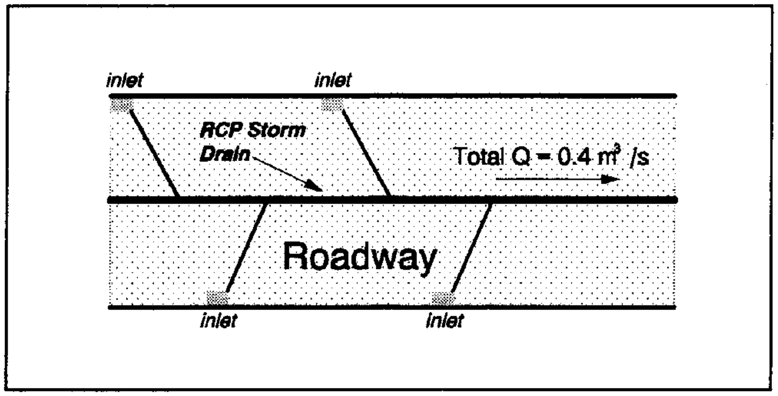
S = Slope, m/m

C. This equation allows for a direct computation of the required pipe diameter. Note that the computed diameter must be increased in size to a larger nominal dimension in order to carry the design discharge without creating pressure flow.

D. The standard SI nominal sizes based on current English unit nominal sizes are given in HDS-4, table 2.

E. Example Problem

Given: Pavement runoff is collected by a series of combination inlets. During the design event, the total discharge intercepted by all inlets is 0.4 m^3/s. A concrete storm drain pipe (n = 0.013) is to be placed on a grade parallel to the roadway grade, which is 0.005 m/m.



Find: The required storm drain pipe diameter and the full flow velocity.

1. Use the full flow equation (which gives pipe diameter in m)

$$Q = \frac{0.312}{n} D^{8/3} S^{1/2}$$

2. Based on HDS-4, table 2, the next larger nominal pipe size is _______.

3. Use the continuity equation (Q = VA) to calculate the full flow velocity.

$Q = VA$; therefore, $V = Q/A$

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

$$V =$$

4. Note that this is the full-flow velocity; however, under our design conditions, the selected nominal pipe size would be flowing slightly less than full. Based on the part-full flow relationships (slide 6.4), the velocity does not change significantly from half-full to full. However, for t_c calculation, the part-full velocity should be used.

   To calculate the part-full velocity, nomographs or trial-and-error solution can be used. Alternatively, the part-full flow relationships can be used. The full-flow discharge and velocity of a 600-mm concrete pipe is

$$Q = \frac{0.312}{0.013} (0.60)^{8/3} (0.005)^{1/2} = 0.43 \text{ m}^3/\text{s}$$

$$V = \frac{Q}{A} = \frac{0.43}{(\pi\ (0.6)^2)/4} = 1.52 \text{ m/s}$$

   The ratio of part-full to full-flow discharge is

$$\frac{Q}{Q_f} = \frac{0.40}{0.43} = 0.93$$

   and from the part-full flow relationships (Slide 6.4), the corresponding velocity ratio is 1.13. Therefore

$$\frac{V}{V_f} = 1.13 \qquad \text{and} \quad V = 1.13\ (1.52) = 1.72 \text{ m/s}$$

NOTES:

Nominal Pipe Sizes (HDS-4, Table 2).		
Nominal Size as Manufactured in English Units		Nominal Size Converted to SI Metric Units
Pipe Diameter		Pipe Diameter (mm)
Inches	Feet	
18	1.5	450
24	2.0	600
30	2.5	750
36	3.0	900
42	3.5	1,050
48	4.0	1,200
54	4.5	1,350
60	5.0	1,500
66	5.5	1,650
72	6.0	1,800
78	6.5	1,950
84	7.0	2,100
90	7.5	2,250
96	8.0	2,400
102	8.5	2,550
108	9.0	2,700
114	9.5	2,850
120	10.0	3,000
126	10.5	3,150
132	11.0	3,300
138	11.5	3,450
144	12.0	3,600

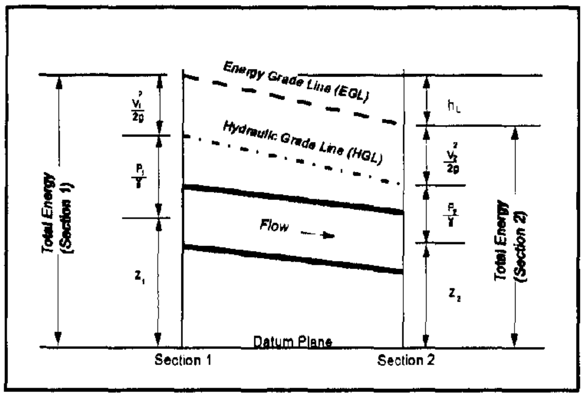
ENERGY EQUATION

- The energy equation is:

$$\alpha_1 \frac{V_1^2}{2g} + \frac{P_1}{\gamma} + Z_1 = \alpha_2 \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + Z_2 + h_L$$

- Energy head at 1 = energy head at 2, plus intervening losses

6.6 Energy equation



6.7 Energy equation in pipe flow

III. Energy Equation

A. The energy equation, as discussed in Lesson 3, is one of the most widely used equations in hydraulics.

B. The energy equation is:

$$\alpha_1 \frac{V_1^2}{2g} + \frac{p_1}{\gamma} + Z_1 = \alpha_2 \frac{V_2^2}{2g} + \frac{p_2}{\gamma} + Z_2 + h_L$$

where
- α = Kinetic energy correction factor, normally assumed to be 1
- V = Average velocity in the cross section, m/s
- g = Acceleration of gravity, 9.81 m/s^2
- p = Pressure, N/m^2 or Pa
- γ = Unit weight of water, 9800 N/m^3 at 15°C
- Z = Elevation above a horizontal datum, m
- h_L = Headloss due to friction and form losses, m

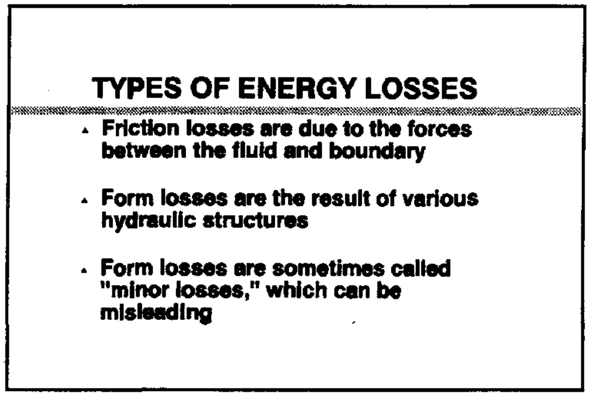
C. In very simple terms the equation states that the energy head at any cross section must equal that in any other downstream section plus the intervening losses.

D. The energy head is divided into three components: the velocity head, the pressure head and the elevation head. The energy grade line (EGL) represents the total energy at any given cross section. The energy losses are classified as friction losses and form losses.

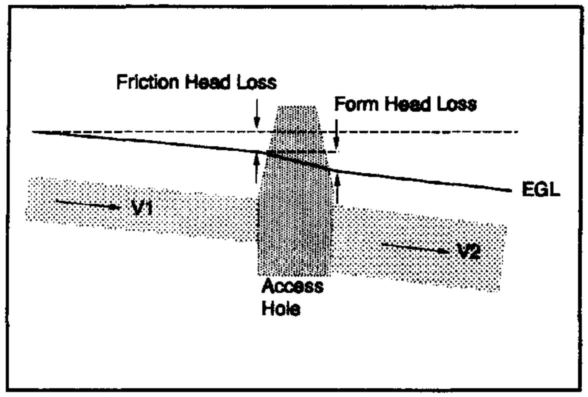
E. The hydraulic grade line (HGL) is below the EGL by the amount of the velocity head.

1. In open-channel flow the HGL is equal to the water surface elevation in the channel.

2. In pressure flow the HGL represents the elevation water would rise to in a stand pipe connected to the conduit.



6.8 Types of energy losses



6.9 Schematic of friction/form losses

IV. Two Types of Energy Losses

A. When using the energy equation all energy losses should be accounted for.

B. Energy losses can be classified as friction losses or form losses.

1. Friction losses are due to forces between the fluid and boundary material.

2. Form losses are the result of various hydraulic structures along the closed conduit (entrance/exit structures, access holes, junctions, etc). Form losses are often called "minor losses," which is misleading since these losses can be large relative to friction losses.

CALCULATING FRICTION LOSSES

- Friction losses are calculated as

$$h_f = L \times S_f$$

- Friction slope based on Manning's equation

$$S_f = \left[\frac{Q\ n}{A\ R^{2/3}}\right]^2$$

6.10 Calculating friction loss

V. Calculating Friction Losses

A. Friction losses are calculated as

$$h_f = LS_f$$

where
L = Length of the conduit
S_f = Friction slope (energy grade line slope)

B. Uniform flow conditions are typically assumed so that the friction slope can be calculated from either Manning's equation, or the Darcy-Weisbach equation. Manning's equation is more frequently used in design applications.

C. Rewriting Manning's equation for S_f:

$$S_f = \left(\frac{Qn}{A\ R^{2/3}}\right)^2$$

TYPICAL "n" VALUES FOR CLOSED CONDUITS

Concrete pipe	0.011-0.013
Concrete Box	0.012-0.015
Corrugated Metal Pipe	0.012-0.037
Plastic pipe	0.012

6.11 Typical n values

D. Typical Manning's n values for closed-conduit flow are given in HDS-4, table 15. Selection of Manning's n values requires judgement. Roughness coefficients are primarily defined by the type of pipe material; however, many other factors can modify the value based on pipe material. Other important factors include the type of joint used, poor alignment and grade due to settlement or lateral soil movement, sediment deposits and flow from laterals disturbing flow in the mainline.

CALCULATING FORM LOSSES

- Simplest method

$$h_L = K \frac{V^2}{2g}$$

- Other methods based on momentum concepts

6.12 Calculating form losses

HYDRAIN METHOD FOR FORM LOSSES

- Bend losses based on simple equation with K defined by angle and radius of bend
- Access hole losses based on simple equation with K based on a series of equations
- Junction losses based on a form of the momentum equation

6.13 HYDRAIN method for form losses

VI. Calculating Form Losses

A. Form losses may be evaluated by several methods. The simplest method is based on a coefficient times the velocity head, with different coefficients tabulated for access holes, bends, inlets, etc. The general form of the equation is:

$$h_L = K \frac{V^2}{2g}$$

For a sudden expansion (e.g., pipe diameter change or a storm drain outlet to a stream channel), K is 1.0.

B. The HYDRA module of HYDRAIN uses several methods to calculate form losses. Bend losses are based on a coefficient times velocity head, with the coefficient (K) being a function of the angle and radius of the bend.

ACCESS HOLE LOSSES

- Basic Equation for K

$K = K_o \times C_D \times C_d \times C_Q \times C_p \times C_B$

- K_o is the initial *head loss coefficient*
- C_D is the correction for *pipe diameter*
- C_d is the correction for *flow depth*
- C_Q is the correction for *relative flow*
- C_p is the correction for *plunging flow*
- C_B is the correction for *benching*

6.14 Access hole losses

6.15 Precast concrete access hole components prior to installation

C. In HYDRAIN (version 5.0), HYDRA access hole losses are calculated in a similar fashion with K defined as

$$K = K_o \times C_D \times C_d \times C_Q \times C_p \times C_B$$

where

K = Adjusted headloss coefficient
K_o = Initial headloss coefficient based on relative access hole size
C_D = Correction factor for pipe diameter
C_d = Correction factor for flow depth
C_Q = Correction factor for relative flow
C_p = Correction factor for plunging flow
C_B = Correction factor for benching

The equations for adjusted headloss and the various correction factors are given in HDS-4, appendix B.

ACCESS HOLE LOSSES (cont)

- Initial head loss coefficient

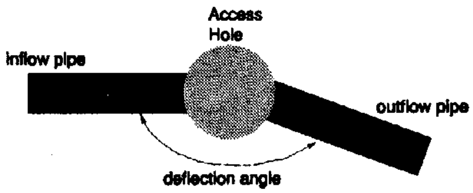
$$K_o = 0.1 \times \frac{b}{D_o}(1 - \sin\theta) + 1.4 \times \left[\frac{b}{D_o}\right]^{0.15} \times \sin\theta$$

6.16 Access hole losses (cont)

$$K_o = 0.1 \times [\frac{b}{D_o}] \times [1 - \sin\theta] + 1.4 \times [\frac{b}{D_o}]^{0.15} \times \sin\theta$$

where

K_o = Initial headloss coefficient based on relative access hole size
θ = Angle between the inflow and outflow pipes (see figure)
b = Access hole diameter
D_o = Outlet pipe diameter

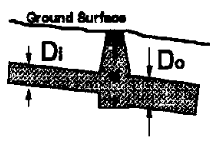


6.17 Angle of deflection

ACCESS HOLE LOSSES (cont)

- Correction factor for pipe diameter change (pressure flow)

$$C_D = \left[\frac{D_o}{D_i}\right]^3$$



only good when $\frac{d}{D_o} > 3.2$

6.18 Access hole losses (cont)

$$C_D = \left[\frac{D_o}{D_i}\right]^3$$

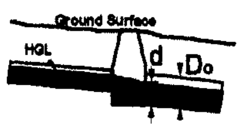
where

C_D = Correction factor for variation in pipe diameter
D_i = Incoming pipe diameter
D_o = Outgoing pipe diameter

ACCESS HOLE LOSSES (cont)

- Correction factor for flow depth (free surface or low pressure)

$$C_d = 0.5 \left[\frac{d}{D_o}\right]^{3/5}$$



only good when $\frac{d}{D_o} < 3.2$

6.19 Access hole losses (cont)

$$C_d = 0.5 \times \left[\frac{d}{D_o}\right]^{3/5}$$

where

C_d = Correction factor for flow depth
d = Water depth in access hole above outlet pipe invert
D_o = Outlet pipe diameter

$$C_p = 1 + 0.2 \times \left[\frac{h}{D_o}\right] \times \left[\frac{h - d}{D_o}\right]$$

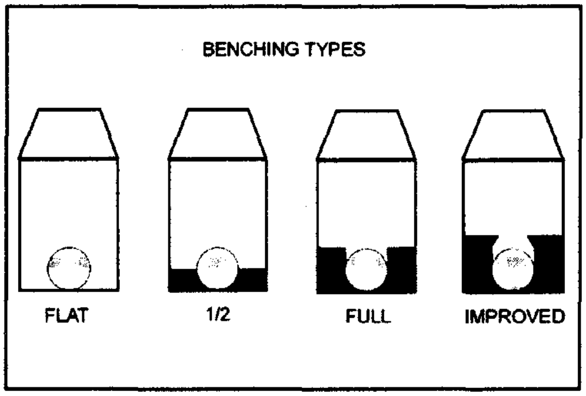
where

C_p = Correction for plunging flow

h = Vertical distance of plunging flow from invert of the plunging pipe to the center of the outlet pipe

D_o = Outlet pipe diameter

d = Water depth in the access hole



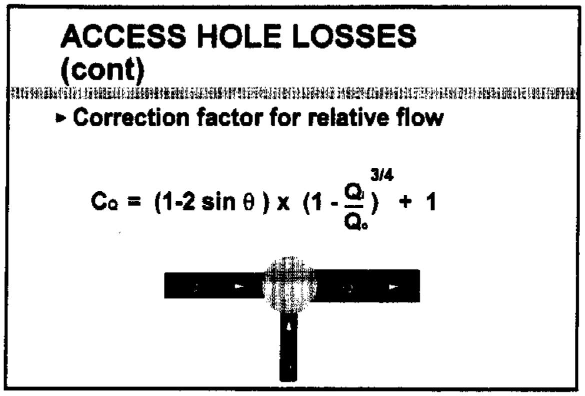
6.22 Schematic for benching factor

ACCESS HOLE LOSSES (cont)

Correction factors for benching (C_B):

Bench Type	Submerged (Pressure flow; d/Do > 3.2)	Unsubmerged (Free surface flow; d/Do <1.0)
Flat Floor	1.00	1.00
Benched 1/2 pipe diameter	0.95	0.15
Benched one pipe diameter	0.75	0.07
Improved	0.40	0.02

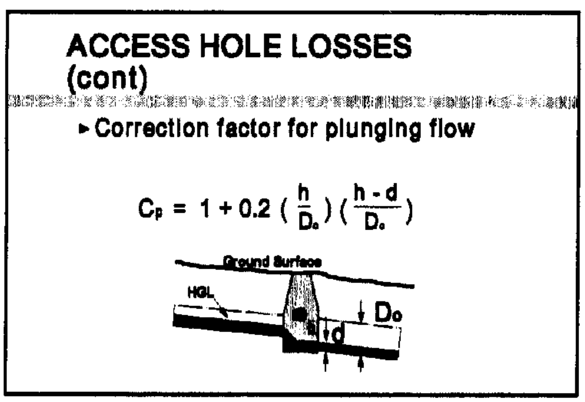
6.23 Access hole losses (cont)



6.20 Access hole losses (cont)

$$C_Q = (1 - 2 \times \sin\theta) \times \left[1 - \frac{Q_i}{Q_o}\right]^{\frac{3}{4}} + 1$$

where
- C_Q = Correction factor for relative flow
- θ = Angle between the original inflow and outflow pipes
- Q_i = Flow in the original inflow pipe
- Q_o = Flow in the outlet pipe



6.21 Access hole losses (cont)

The benching correction factor, C_B, is based on tabulated values:

Bench Type	Correction Factors C_B Submerged*	Unsubmerged**
Flat floor	1.00	1.00
Benched one-half of pipe diameter	0.95	0.15
Benched one pipe diameter	0.75	0.07
Improved	0.40	0.02

* pressure flow, $d/Do > 3.2$

**free surface flow, $d/Do < 1.0$

NOTES:

LESSON 7

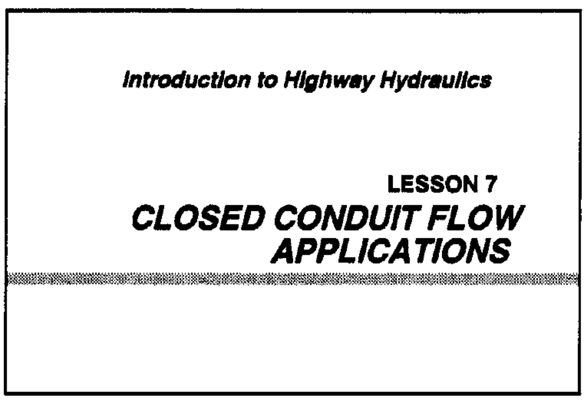
CLOSED-CONDUIT FLOW APPLICATIONS

OVERVIEW: Method of Instruction: Lecture, Demonstration and Workshop

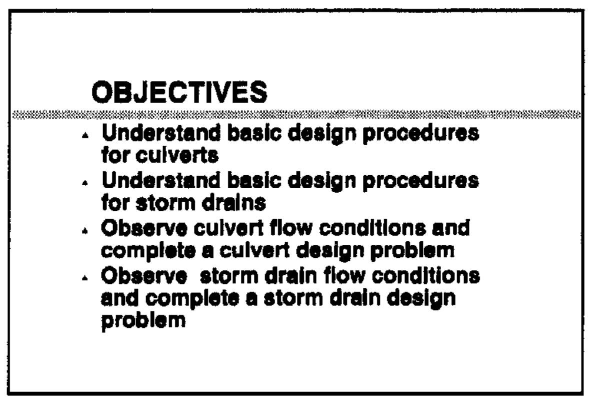
Lesson Length:	Part 1: Culvert Design Procedures	- 90 minutes
	Part 2: Storm Drain Design Procedures	- 30 minutes
	Part 3 and 4: Demonstration and Workshop	- 120 minutes

Resources:

Lesson Outline
Slides
Video
Demonstration Flume
HDS-4, Chapter 7



7.0 Title



7.1 Objectives

OBJECTIVES: The objectives of this lesson are to:

1. Understand basic design procedures for culverts.

2. Understand basic design procedures for storm drains.

3. Observe typical culvert flow conditions in the demonstration flume and complete a culvert design problem.

4. Observe typical storm drain flow conditions in the demonstration flume and complete a storm drain design problem.

LESSON 7

CLOSED-CONDUIT FLOW APPLICATIONS

TYPICAL APPLICATIONS OF CLOSED CONDUITS

- Culverts often used:
  - when cross drainage is necessary
  - when downdrains are required
- Storm Drains often used:
  - when allowable spread or channel capacity has been exceeded
  - when erosion control is a problem

7.2 Applications of closed conduits

STORM DRAIN/CULVERT CHARACTERISTICS

- Constructed of reinforced concrete, corrugated metal and plastic
- Available in nominal sizes
- Size should not be decreased in downstream direction
- Energy dissipation often required at outlet
- Maintenance is required due to sediment and debris deposition

7.3 Storm drain and culvert characteristics

I. Typical Highway Applications of Closed Conduits

A. Typical closed-conduit facilities in highway drainage include culverts and storm drains.

B. Culverts are typically shorter conduits that are used for:

1. Cross drainage and can range in size from a single small culvert draining an isolated depression to multiple barrel designs and/or very large culverts for passing major stream channels under a roadway.

2. Downdrains to protect fill slopes or to divert roadway water from a bridge deck.

C. Storm drain systems consist of inlets connected to an underground pipe and an outlet facility. Storm drains are typically used:

1. When allowable pavement spread or channel capacity has been exceeded.

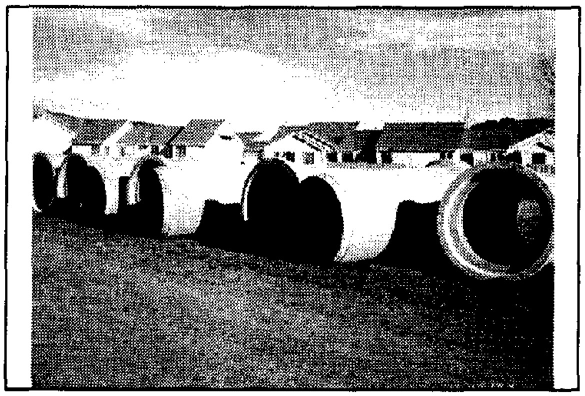
2. In high gradient situations where erosion control is a problem.

D. Typical pipe materials used in culverts and storm drains include reinforced concrete, corrugated metal and plastic.

E. Conduit and culvert material are typically available in standard (nominal) sizes. Conduit size should not be decreased in the downstream direction, even if hydraulic calculations suggest this is possible.

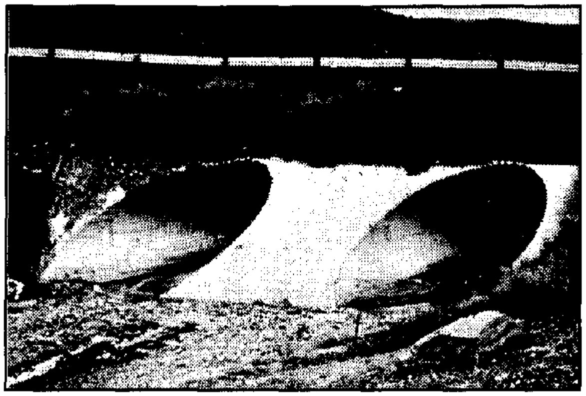
F. Energy dissipation is often required at the outlet of a storm drain or culvert to prevent erosion.

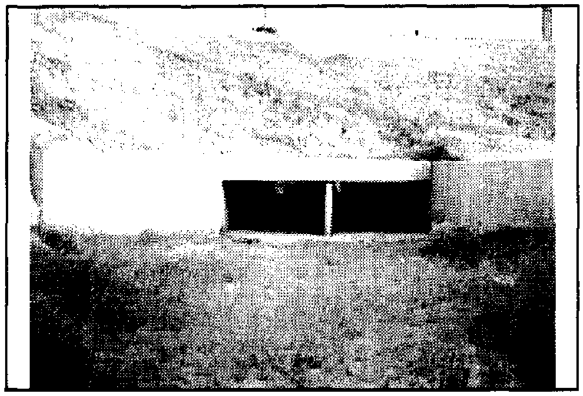
G. Maintenance is required for any closed-conduit facility. Sediment deposition within the conduit and debris removal at the entrances are typical maintenance items.



7.4a Storm drain pipe

7.4b Installing storm drain pipe


7.5a CMP culvert


7.5b RCB culvert

NOTES:

NOTES:

LESSON 7

CLOSED CONDUIT FLOW APPLICATIONS

PART 1: Culvert Design Procedures

Introduction to Highway Hydraulics

LESSON 7, Part 1

CULVERT DESIGN PROCEDURES

7.6 Lesson 7, Part 1

CULVERT DESIGN APPROACH

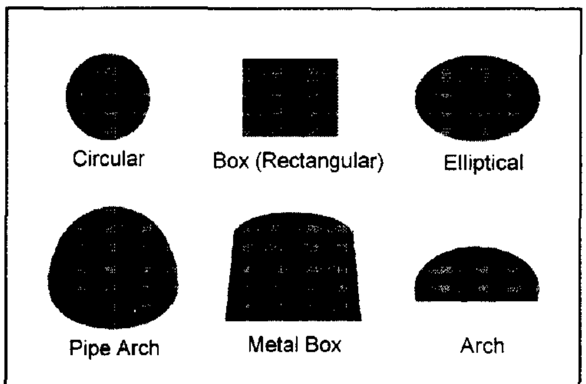
- **Culvert material selection depends on strength, roughness, durability and abrasion factors**
- **Standard shapes available**

7.7 Culvert design approach

II. Culvert Design Approach

A. The selection of culvert material depends on structural strength, hydraulic roughness, durability, and corrosion and abrasion resistance.

B. The most common cross sectional shapes for culverts are illustrated in slide 7.8.



7.8 Culvert shapes

CULVERT DESIGN APPROACH

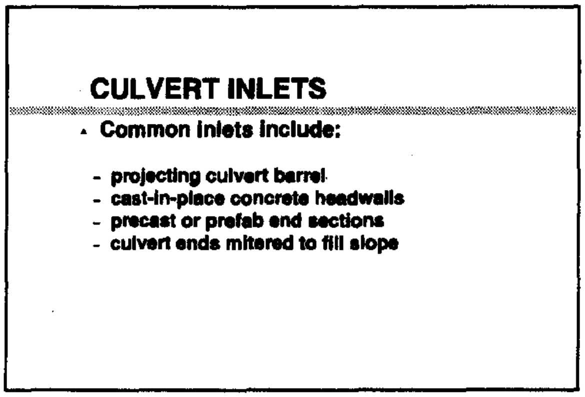
- **Culvert material selection depends on strength, roughness, durability and abrasion factors**
- **Standard shapes available**
- **Flow may occur as open channel, gravity full or pressure flow**
- **Theoretical analysis is difficult**
- **For purposes of design, standard procedures and nomographs are used (see HDS-5)**

7.9 Culvert design approach (cont)

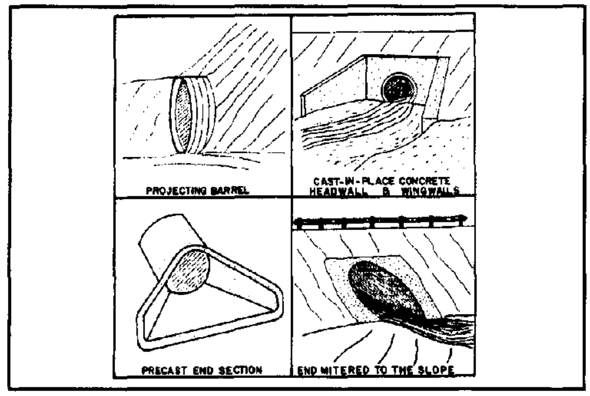
C. Flow conditions in a culvert may occur as open-channel flow, gravity full flow or pressure flow, or in some combination of these conditions.

D. A complete theoretical analysis of the hydraulics of culvert flow is time-consuming and difficult.

E. For purposes of design, standard procedures and nomographs have been developed to simplify the analysis of culvert flow. These procedures are detailed in HDS-5, "Hydraulic Design of Highway Culverts," and taught in NHI Course 13056, "Culvert Design."



7.10a Types of inlets and outlets



7.10b Four standard inlets

II. Culvert Inlets

A. Culvert inlets are available in a variety of configurations and may be prefabricated or constructed in place. Commonly used inlet configurations include:

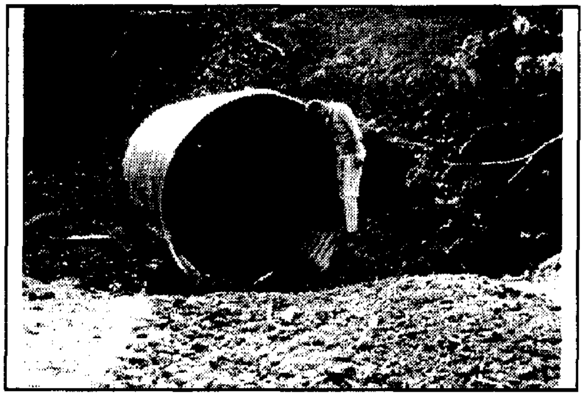
1. projecting culvert barrels,
2. cast-in-place concrete headwalls,
3. precast or prefabricated end sections, and
4. culvert ends mitered to conform to the fill slope.

B. Inlet configuration affects energy loss and hence culvert performance.

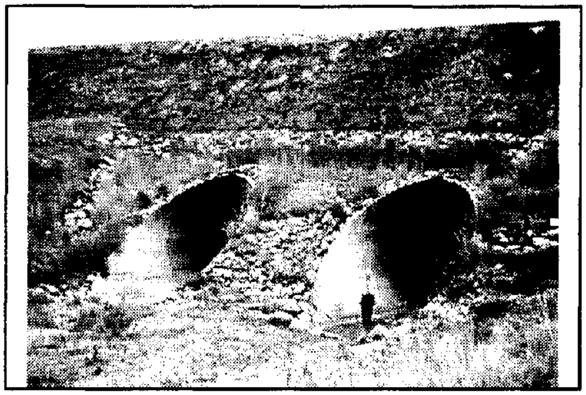
Entrance Loss Coefficients

Inlet Configuration	Entrance Loss Coefficient
Thin Edge Projecting Barrel	*0.9*
Mitered End	*0.7*
Headwall	*0.5*
Precast End Section	*0.5*
Groove-end, Beveled or Improved Inlet	*0.2*

7.11 Inlet loss coefficients

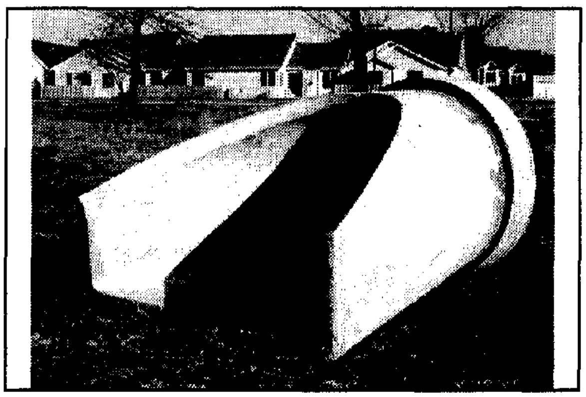


7.12 Projecting barrel

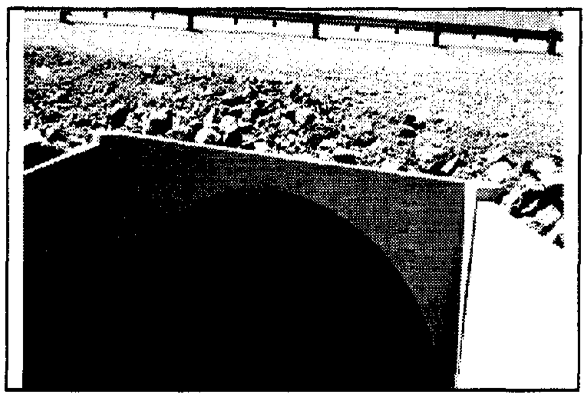


7.13 End mitered to slope

7.14 Cast-in-place headwall



7.15 Precast end section



7.16 Beveled inlet

CULVERT INLETS (cont)

- Selection of an inlet based on:
  - structural stability
  - aesthetics
  - erosion control and fill retention
  - hydraulic performance
  - economics and safety

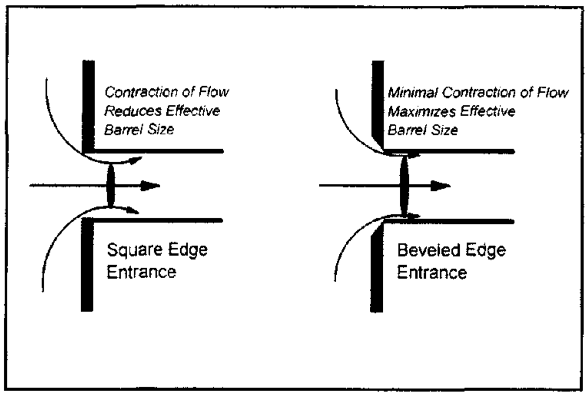
7.17 Types of inlets... (cont)

C. Structural stability, aesthetics, erosion control, fill retention, hydraulic performance, economics and safety are considerations in the selection of an inlet.

CULVERT INLETS (cont)

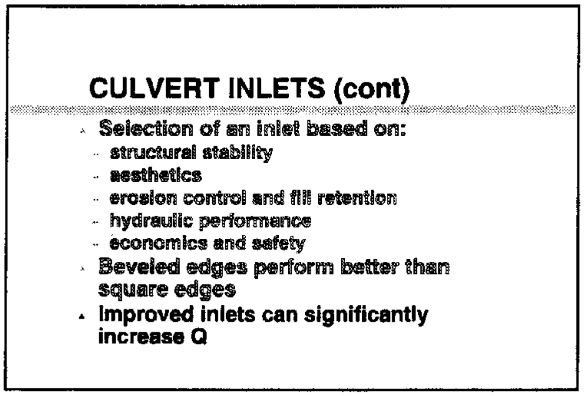
- Selection of an inlet based on:
  - structural stability
  - aesthetics
  - erosion control and fill retention
  - hydraulic performance
  - economics and safety
- Beveled edges perform better than square edges

7.18a Beveled inlets improve performance

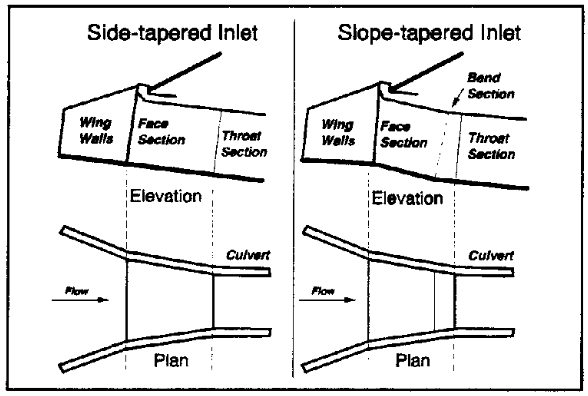


7.18b Beveled inlet schematic

D. Hydraulic performance is improved by use of beveled edges rather than square edges.

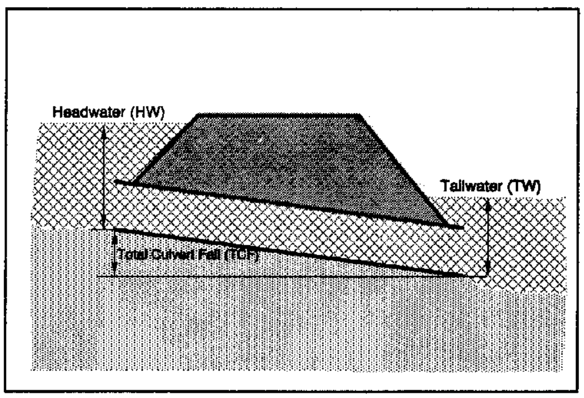


7.19 Improved inlets...

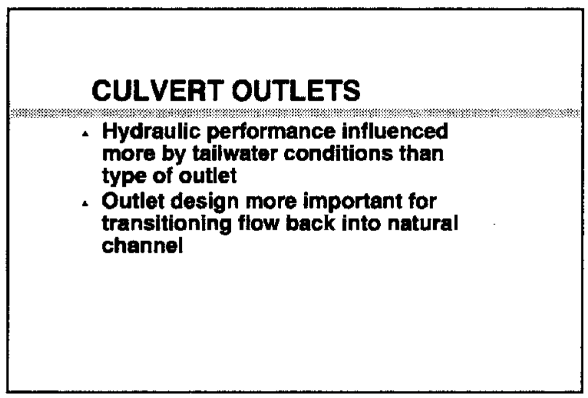


7.20 Improved inlets schematic

F. Side-tapered and slope-tapered inlets, commonly referred to as improved inlets, can significantly increase culvert capacity. A side-tapered inlet provides a more gradual contraction of flow and reduces energy losses. A slope-tapered inlet, or depressed inlet, increases the effective head on the control section and improves culvert efficiency.



7.21 Schematic of HW and TW



7.22 Culvert outlets

III. Culvert Outlets

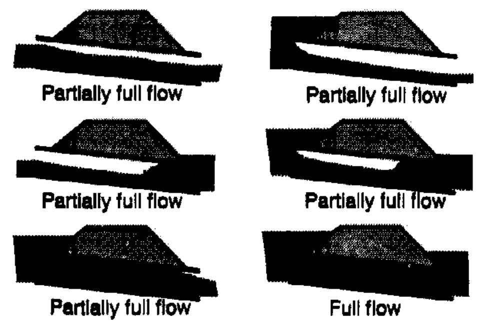
A. For culvert outlets, hydraulic performance is influenced more by tailwater conditions in the downstream channel than by the type of outlet.

B. Outlet design is important for transitioning flow back into the natural channel, since outlet velocities are typically high and can cause scour of the downstream streambed and bank.

CULVERT FLOW CONDITIONS

- Culverts can flow full or partially full
  - water surface profile analysis required to determine part full flow conditions
  - partly full flow may be subcritical, critical or supercritical
  - gravity full flow is a special case of free surface flow

7.23 Culvert flow conditions



7.24 Flow conditions schematic

IV. Culvert Flow Conditions

A. A culvert may flow full over all its length or partially full. Full flow throughout a culvert is rare, and generally some portion of the barrel flows partly full.

1. A water surface profile analysis is the only way to accurately determine how much of the barrel flows full.

2. Pressure flow conditions in a culvert can be created by either high downstream or upstream water surface elevations.

3. Partly full flow, or open channel flow, in a culvert may occur as subcritical, critical or supercritical flow.

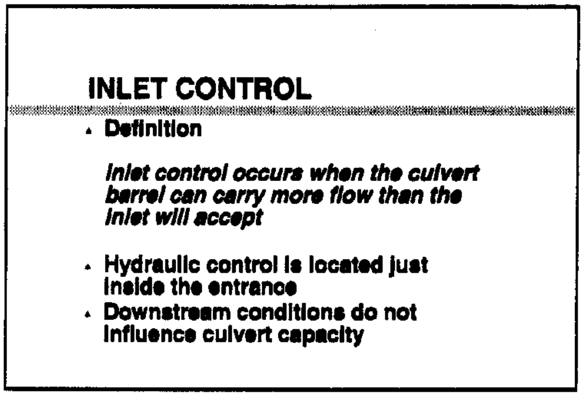
B. Gravity full flow, where the pipe flows full with no pressure and the water surface just touches the crown of the pipe, is a special case of free surface flow and is analyzed in the same manner as open channel flow.

CULVERT FLOW CONDITIONS

- Culverts can flow full or partially full
  - Water surface profile analysis required to determine part full flow conditions
  - Partly full flow may be subcritical, critical or supercritical
  - Gravity full flow is a special case of free surface flow
- Two types of flow control:
  - inlet control
  - outlet control

7.25 Culvert flow conditions

C. Based on a variety of laboratory tests and field experience, two basic types of flow control have been defined for culverts: inlet control and outlet control.



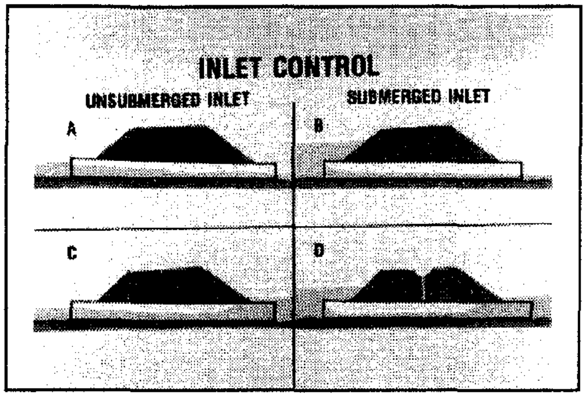
7.26 Inlet control

V. Inlet Control

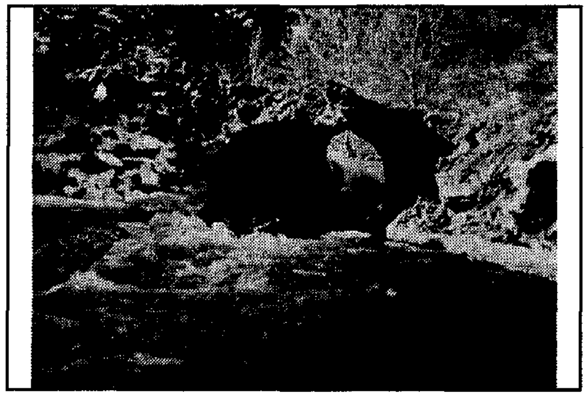
A. Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept.

B. The hydraulic control section of a culvert operating under inlet control is located just inside the entrance. Critical depth occurs at or near this location and the flow regime immediately downstream is supercritical.

C. Hydraulic characteristics downstream of the inlet do not affect culvert capacity. The upstream water surface elevation and inlet geometry are the primary factors influencing culvert capacity.



7.27 Types of inlet control


7.28 Example of inlet control

D. Slide 7.27 illustrates typical inlet control conditions. The type of flow depends on the submergence of the inlet and outlet ends of the culvert; however, in each case the control section is at the inlet end of the culvert. For low headwater conditions the entrance of the culvert operates as a weir, and for headwaters submerging the entrance the entrance operates as an orifice.

OUTLET CONTROL

- **Definition**

  Outlet control occurs when the culvert barrel cannot carry as much flow as the inlet will accept

- **Hydraulic control is at the barrel exit, or further downstream**
- **Either subcritical or pressure flow occur in the barrel**

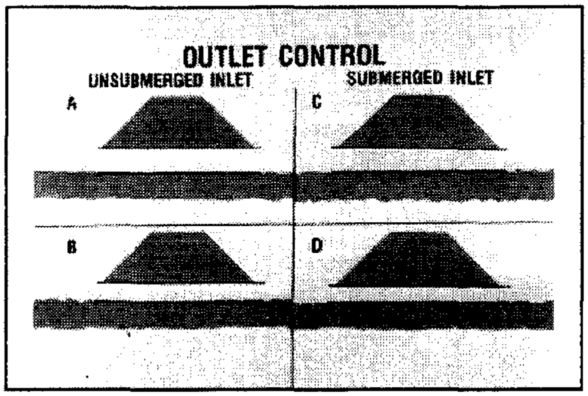
7.29 Outlet control

V. Outlet Control

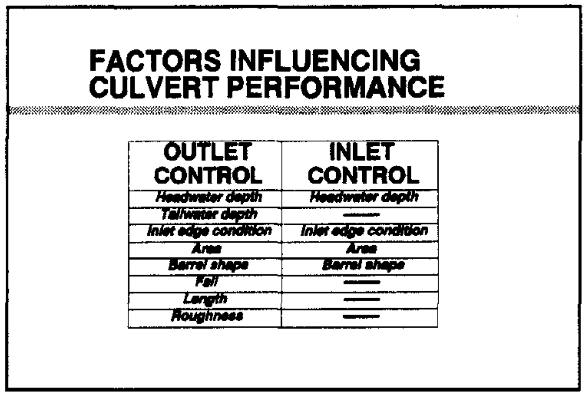
A. Outlet control occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept.

B. The control section for outlet control is located at the barrel exit or further downstream.

C. Either subcritical or pressure flow exists in the culvert under outlet control. All the geometric and hydraulic characteristics of the culvert play a role in determining culvert capacity.



7.30 Types of outlet control



7.31 Inlet/outlet summary

D. Slide 7.30 illustrates typical outlet control conditions.

HEADWATER/TAILWATER CONSIDERATIONS

- Increased water surface provides necessary energy
- Headwater (HW) is the depth of water at the culvert entrance
- Ponding will occur upstream of a culvert and may attenuate flood peak
- Tailwater (TW) is depth at the culvert outlet, measured from the invert
- Tailwater defined by downstream channel conditions

7.32 Headwater and tailwater

VII. Headwater and Tailwater Considerations

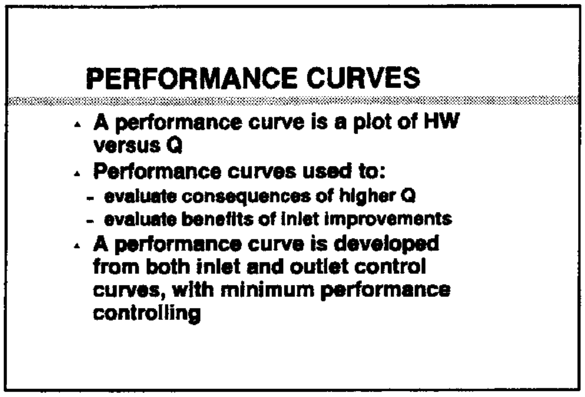
A. Energy is required to force flow through the constricted opening represented by a culvert. This energy occurs as an increased water surface elevation on the upstream side of the culvert.

B. The headwater depth (HW) is defined as the depth of water at culvert entrance.

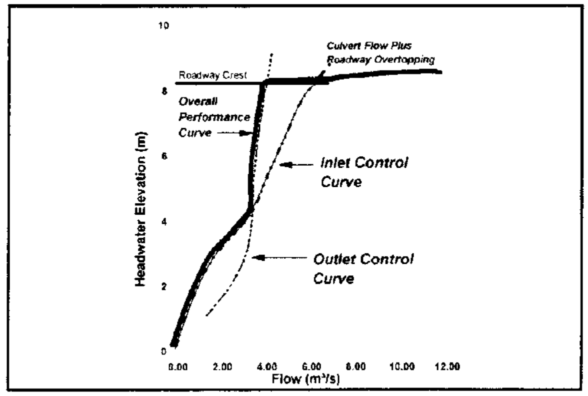
C. In areas with flat ground slope or high fills a considerable amount of ponding may occur upstream of culvert. If significant, this ponding can attenuate flood peaks and may justify a reduction in the required culvert size.

D. Tailwater is defined as the depth of water downstream of the culvert, measured from the outlet invert. Tailwater is an important factor in determining culvert capacity under outlet control conditions.

E. Tailwater conditions are most accurately estimated by water surface profile analysis of the downstream channel; however, tailwater conditions may be estimated by normal depth approximations.



7.33 Performance curves



7.34 Performance curve example

VIII. Performance Curves

A. A performance curve is a plot of headwater depth or elevation versus flow rate.

B. A performance curve can be used to evaluate the consequences of higher flow rates, such as the potential for overtopping the roadway if the design event is exceeded, or to evaluate the benefits of inlet improvements.

C. In developing a performance curve both inlet and outlet control curves must be plotted, since the dominant control is hard to predict and may shift over a range of flow rates.

CULVERT DESIGN METHOD

- Design is based on location of control (inlet or outlet)
- Concept of "minimum performance" is applied
- For a given Q, a culvert size and material are assumed and the HW calculated (inlet and outlet control).
- The higher HW is the controlling headwater elevation

7.35 Culvert design method

IX. Culvert Design Method

A. The basic design method is based on the location of the control (inlet or outlet).

B. Although control may oscillate from inlet to outlet, the concept of "minimum performance" is applied meaning that while the culvert may operate more efficiently at times, it will never operate at a lower performance than calculated.

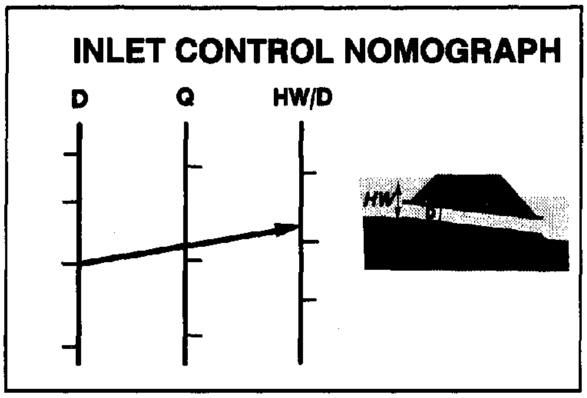
C. The design procedure then is to assume a pipe size and material and calculate the headwater elevation for both inlet and outlet control, using the HDS-5 nomographs or HYDRAIN, HY-8. The higher of the two is designated as the controlling headwater elevation.

CULVERT DESIGN METHOD (cont)

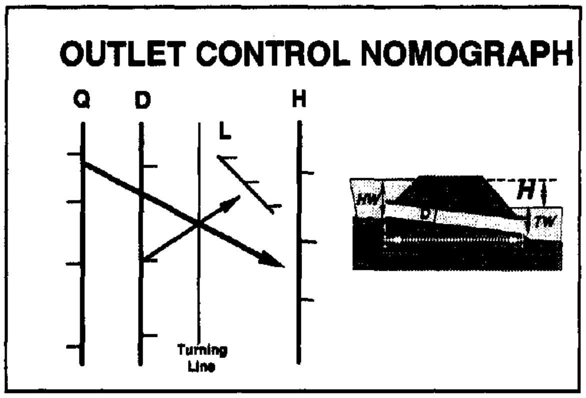
- Controlling HW compared to desired design HW to evaluate performance
- Outlet velocity evaluated to define need for outlet protection
- Evaluation of headwater and outlet velocity continues until an acceptable culvert configuration is defined
- Selected culvert must then fit roadway cross section

7.36 Culvert design method (cont)

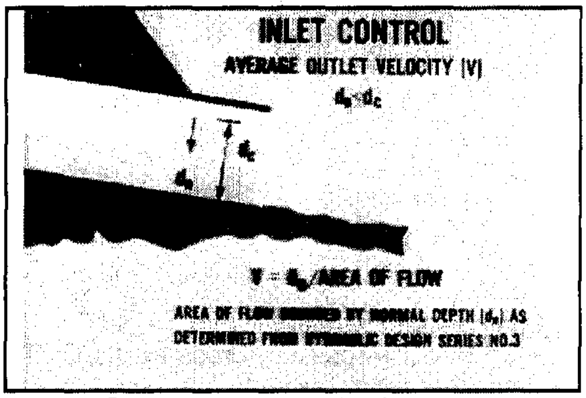
D. The controlling headwater elevation is compared to the desired design headwater, usually governed by overtopping considerations, to determine if the assumed culvert size is acceptable.



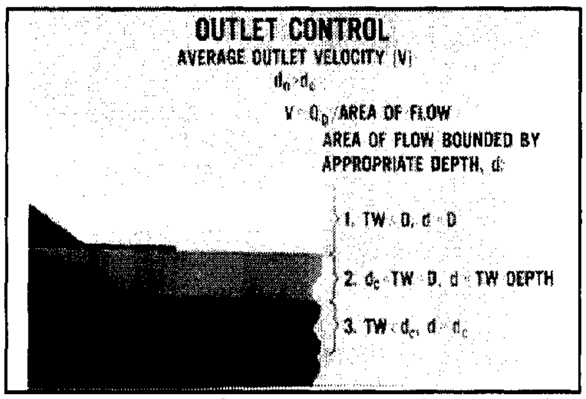
7.37a Inlet control nomograph



7.37b Outlet control nomograph



7.38a Outlet velocity under inlet control



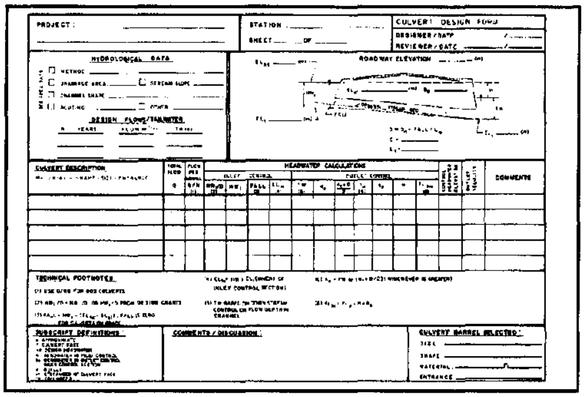
7.38b Outlet velocity under outlet control

E. The outlet velocity should then be considered to evaluate the need for outlet protection:

1. If the controlling headwater is based on inlet control, determine the normal depth and velocity in the culvert barrel. The velocity at normal depth is assumed to be the outlet velocity.

2. If the controlling headwater is based on outlet control, determine the area of flow at the outlet based on the barrel geometry and the following: (1) critical depth if the tailwater is below critical depth, (2) the tailwater depth if the tailwater is between critical depth and the top of the barrel, (3) the height of the barrel if the tailwater is above the top of the barrel.

F. The evaluation of headwater conditions and outlet velocity is repeated until an acceptable culvert configuration is determined. To facilitate the design process, a Culvert Design Form is provided in HDS-5.


PROJECT :
STATION
SHEET OF
CULVERT DESIGN FORM
DESIGNER / DATE
REVIEWER / DATE
HYDROLOGICAL DATA
METHOD
DRAINAGE AREA
STREAM SLOPE
CHANNEL SHAPE
ROUTING
OTHER
DESIGN FLOWS/TAILWATER
ROADWAY ELEVATION
CULVERT DESCRIPTION
HEADWATER CALCULATIONS
INLET CONTROL
OUTLET CONTROL
COMMENTS
TECHNICAL FOOTNOTES
SUBSCRIPT DEFINITIONS
COMMENTS / DISCUSSION :
CULVERT BARREL SELECTED :
SIZE
SHAPE
MATERIAL
ENTRANCE

7.39 Culvert design form

G. Once the barrel is selected, it must be fitted into the roadway cross section. The culvert barrel must have adequate cover, the length should be close to the approximate length, and the headwalls and wingwalls must be dimensioned. Culvert analysis and design can be completed using HYDRAIN.

IMPROVED INLET DESIGN

- **Improved inlets more important for *inlet control culverts***
- **Improved inlets include:**
  - **bevel-edged inlets**
  - **side-tapered inlets**
  - **slope-tapered inlets**

7.40 Improved inlet design

X. Improved Inlet Design

A. Improved inlet design can increase culvert capacity significantly for culverts operating under inlet control, but will have only a minor effect on culverts operating under outlet control.

1. Inlet improvements on culverts functioning under outlet control will reduce entrance losses, but these losses are only a minor part of the total headwater requirement. Therefore, only minor modifications of the inlet geometry which result in little additional cost are justified.

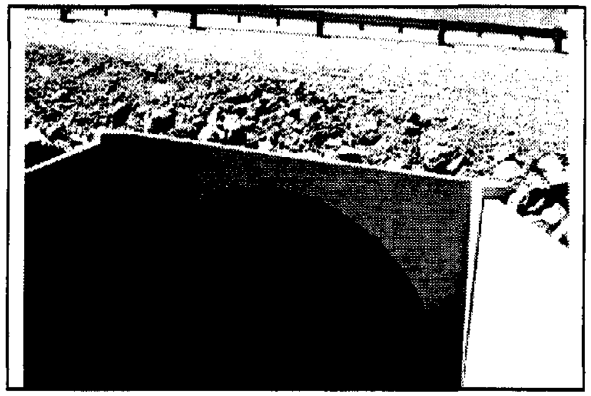
2. Inlet improvements on culverts functioning under inlet control can change the typically part full flow in the barrel to full or nearly full flow, better using barrel size and increasing culvert capacity significantly.

B. Inlet improvements consist of bevel-edged inlets, side-tapered inlets and slope-tapered inlets.

BEVEL-EDGED INLETS

- Beveled edges reduce contraction of flow by enlarging culvert face
- Bevels are plane surfaces; but rounded culvert edges are also effective

7.41 Bevel-edge inlets



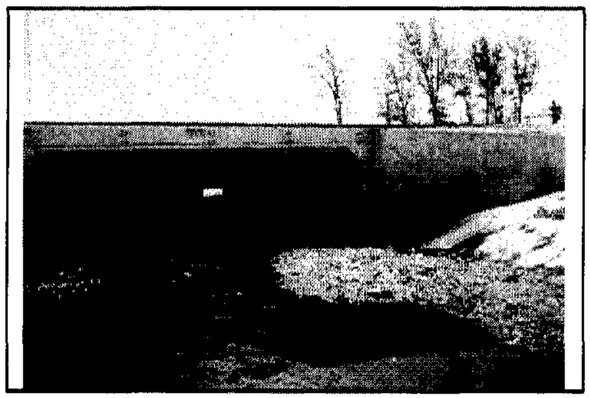
7.42 Beveled edge example

C. Beveled edges reduce the contraction of flow by effectively enlarging the face of the culvert. Bevels are plane surfaces, but rounded edges that approximate a bevel, such as the socket end of RCP, are also effective. Bevels are recommended on all headwalls.

SIDE-TAPERED INLETS

- Tapered inlets improve performance by providing more efficient control section
- Intersection of sidewall tapers and barrel is defined as the throat
- Two control sections possible
  - at the face
  - at the throat

7.43 Side-tapered inlets



7.44 Side-tapered example

D. A second degree of improvement is a side-tapered inlet. Tapered inlets improve culvert performance by providing a more efficient control section (the throat). The inlet has an enlarged face area with the transition to the culvert barrel accomplished by tapering the sidewalls.

1. The intersection of the sidewall taper and barrel is defined as the throat section.

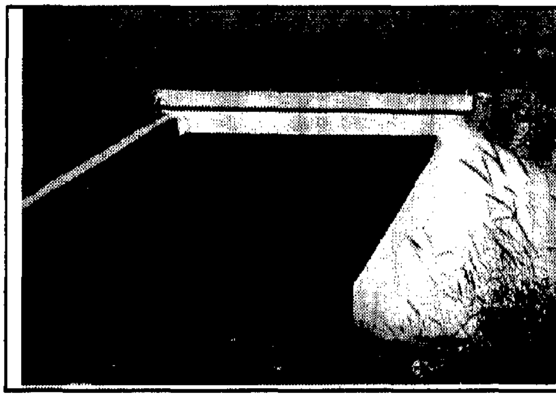
2. Two control sections occur on a side-tapered inlet: at the face and throat.

3. Throat control reduces the contraction at the throat.

SLOPE-TAPERED INLETS

- **Slope-tapered inlets provide a depression of the inlet that applies more head at the control section**
- **Control can occur at face or throat**
- **Throat control is preferrable and will cause more of the potential capacity of the barrel to be used**

7.45 Slope-tapered inlets



7.46 Slope-tapered example

E. A third degree of improvement is a slope-tapered inlet. The advantage of a slope-tapered inlet over a side-tapered inlet without a depression is that more head is applied at the control (throat) section.

1. Both face and throat control are possible in a slope-tapered inlet; however, since the major cost of a culvert is in the barrel portion and not the inlet structure, the inlet face should be designed with greater capacity at the allowable headwater elevation than the throat.

2. This will insure flow control will be at the throat and more of the potential capacity of the barrel will be used.

XI. Culvert Design with HYDRAIN

A. HY8 module of HYDRAIN can be used to design culverts.

NOTES:

LESSON 7

CLOSED CONDUIT FLOW APPLICATIONS

PART 2: Storm Drain Design Procedures

Introduction to Highway Hydraulics

LESSON 7, Part 2

STORM DRAIN DESIGN PROCEDURES

7.47 Lesson 7, Part 2

STORM DRAIN DESIGN APPROACH

- **Typically involves detailed calculations completed in an iterative manner**
- **Major steps include:**
  - **preliminary layout**
  - **sizing conduits**
  - **HGL calculation**
  - **revisions to optimize or correct problems**

7.48 Storm drain design approach

I. Storm Drain Design Approach

A. The design of a storm drain system is not a complicated process, but can involve detailed calculations that are often completed in an iterative manner.

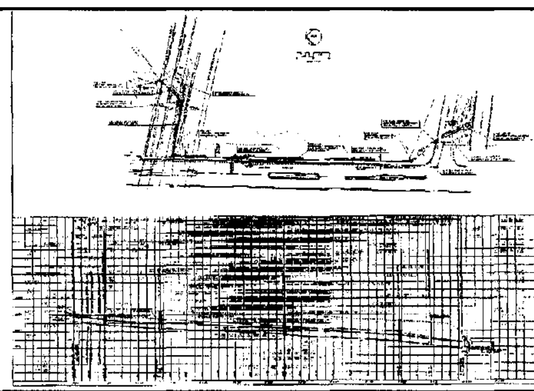
B. The major steps in storm drain design are:

1. Development of a preliminary layout.
2. Sizing the conduits in the storm drain system.
3. Computation of the HGL.
4. Adjustment of inlet sizes/locations and pipe size/location to correct HGL problems and/or optimize the design.

PRELIMINARY LAYOUT

- **The preliminary layout is a plan and profile drawing showing**
  - **plan view showing existing roadway, bridges, surface utilities, etc. and the proposed pipe layout, inlets, access holes, etc**
  - **profile view showing existing utilities and storm drain systems and proposed pipe profile**

7.49 Preliminary layout



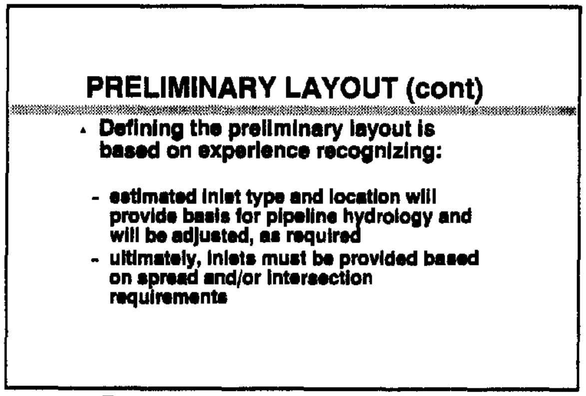
7.50 Example plan/profile sheet

II. Preliminary Layout

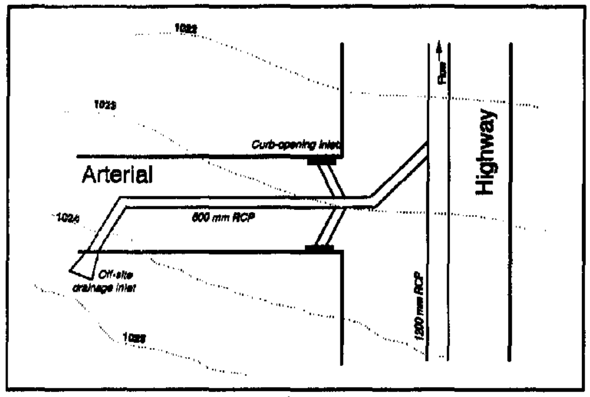
A. The first step in storm drain design is to develop a preliminary storm drain layout, including inlet, access hole and pipe locations.

1. The preliminary layout is usually completed on a plan and profile drawing that shows the roadway, bridges, adjacent land use conditions, intersections, and under/overpasses.

2. Other utility locations and situations should also be identified and shown, including surface utilities, underground utilities and any other storm drain systems. Storm drain alignment within the road right-of-way is usually influenced, if not dictated, by the location of other utilities. These other utilities, which may be public or private, may cause interference with the alignment or elevation of the proposed storm drain.



7.51 Preliminary layout (cont)



7.52 Intersection inlet example

B. Tentative inlets, junctions and access locations should be identified based primarily on experience factors, recognizing that:

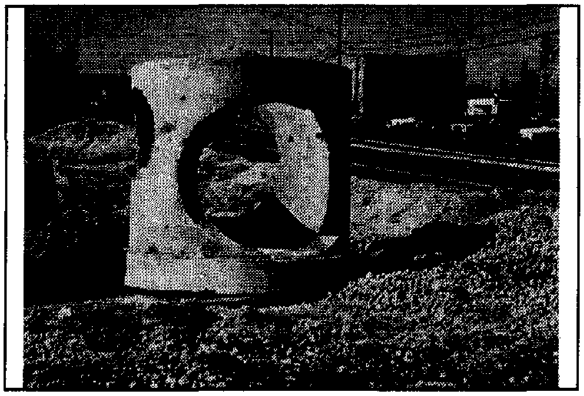
1. The initially estimated type and location of inlets will provide the basis for hydrologic calculations and pipe sizing, and will be adjusted as required during the design process.

2. Ultimately, inlets must be provided based on spread criteria and/or intersection requirements. Generally all flow approaching an intersection should be intercepted, as cross gutters are not practical in highway applications.

PRELIMINARY LAYOUT (cont)

- Other factors influencing layout include:
  - access hole requirements
  - junction requirements
  - minimizing excavation
  - traffic flow issues
  - use of curved alignments for larger systems

7.53 Preliminary layout (cont)



7.54 Access structure

C. Other factors influencing preliminary layout include:

1. Access is required for inspection and maintenance of storm drain systems. For storm drains smaller than about 1 m, access is required about every 120 m, while for larger sizes the spacing can be 180 m and larger.

2. Junctions are required at the confluence of two or more storm drains, where pipe size changes, at sharp curves or angle points (greater than 10 degrees), or at abrupt grade changes.

3. Generally, a storm drain should be kept as close to the surface as minimum cover and/or hydraulic requirements allow to minimize excavation costs.

4. Another location control is the demand of traffic and the need to provide for traffic flow during construction including the possible use of detours.

5. Providing curved storm drain alignments may be cost effective and should be considered for large pipe sizes, especially when headlosses are a concern.

PIPE SIZING

- Calculate discharge at each inlet
- Adjust inlet locations, as required
- Main line sized on longest time of concentration (not sum of discharges)

7.55 Pipe sizing

7.56 Inlet before turn lane

III. Pipe Sizing

A. Given the preliminary layout, it is possible to begin the hydrologic and hydraulic analysis necessary to size the storm drain system.

B. The first step is to calculate the discharge contributing to each inlet location and to size the inlet. Based on the small incremental drainage areas involved, the Rational method is typically used. Given the discharge at each selected inlet and considering spread criteria, it may be necessary to relocate an inlet or to incorporate additional inlets.

C. After adjusting inlet locations the storm drain laterals and main line can be sized. Laterals are sized based on the discharge used to size the inlets.

D. It is important to realize that the discharge for the main line is not simply the sum of the incremental discharges at each inlet. Recalling that in the Rational method the rainfall intensity used should be based on the longest time of concentration to the design point, the discharge for the main line should be determined based on longest time of concentration from various upstream approach branches and the corresponding accumulated CA values. This procedure satisfies the assumptions and stipulations for use of the Rational method.

PIPE SIZING

- Calculate discharge at each inlet
- Adjust inlet locations, as required
- Main line sized on longest time of concentration (not sum of inlets)
- Pipe size estimated from full flow Manning equation
- The crown should drop by the headloss across the structure
- Minimum velocity of 0.9 m/s

7.57 Pipe sizing (cont)

E. Preliminary pipe size is then calculated based on a full flow assumption given the discharge and pipe slope. This approach does not account for minor losses, which will be accounted for in the HGL calculation. Minor losses can be approximately accounted for at this stage of design by using a slightly higher roughness value in the full flow calculation.

F. The pipe slope is typically established in preliminary design based on the roadway grade and the need to avoid other existing utilities or storm drains.

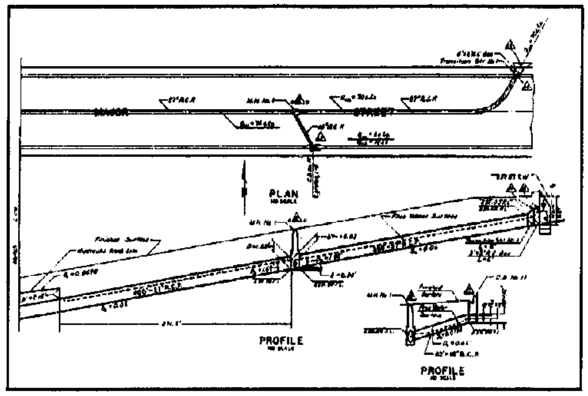
H. The crown of the downstream pipe should drop by the headloss across the structure.

I. Generally, storm drains should be designed to provide a velocity of at least 0.9 m/s when the conduit is full to insure that the pipe is self cleaning.

HGL CALCULATION

- HGL is used to evaluate overall performance of preliminary layout
- At design discharge and/or check discharge, inundation, backflow or other adverse effects should be avoided at inlets, access holes, and other appurtenances

7.58 HGL calculation



7.59 Example HGL profile

IV. Hydraulic Grade Line Calculation

A. The HGL is used to evaluate overall system performance and insure that at the design discharge the storm drain system does not inundate or adversely affect inlets, access holes or other appurtenances.

HGL CALCULATION (cont)

- Locate controls
- Establish tailwater
- Calculation proceeds on a reach-by-reach basis from a given control point (typically upstream)
- Calculation based on energy equation

7.60 HGL calculation (cont)

B. The first step in calculating the HGL is establishing the location of all hydraulic controls along the conduit alignment and locations where the water surface must not be exceeded. These may occur at the inlet, at the outlet or at intermediate points along the alignment.

C. If the tailwater elevation is not known or is low, use the process given in HDS-5 to estimate the approximate hydraulic grade line by averaging the height of the storm drain and critical depth. This value cannot exceed the height of the drain and should be used if it is greater than the actual tailwater depth. Storm drains are normally designed to flow full in outlet control; but if control is not at the outlet, then inlet control equations should be used.

D. The calculation proceeds on a reach-by-reach basis from a given control point. In contrast with the downstream direction of calculations used to design the overall system, the calculation of the HGL usually proceeds in the upstream direction depending on the location of the control(s).

ENERGY EQUATION IN HGL ANALYSIS

- Design based on uniform HGL in a given reach
- Friction losses based on full flow assumption
  - pressure flow when $S_f > S_o$
  - partial flow when $S_f < S_o$
- Form losses assumed as point losses across the structure
- HGL = EGL - velocity head

7.61 Energy equation

E. The calculation is based on application of the energy equation. For the first reach, the energy losses (friction and form) are calculated. The first reach is defined from the downstream control point to the first hydraulic structure or conduit grade break. Subsequent reaches are defined between hydraulic structures and/or grade breaks.

F. The design procedure is usually based on the assumption of a uniform hydraulic gradient within a conduit reach. Greater accuracy could be achieved with water surface profile computations, but such accuracy is seldom necessary.

G. Friction losses are calculated assuming full barrel flow using the full flow version of the Manning's equation. It defines the friction slope given the design discharge and preliminary pipe diameter.

1. If the calculated friction slope is steeper than the pipe slope, pressure flow conditions will exist.

2. If the friction slope is less than the pipe slope, partial flow will occur.

3. At the location where the pipe becomes unsealed, or transitions to partial flow, normal depth calculations used to estimate hydraulic conditions.

H. For each hydraulic structure, account for the form losses. Form losses are typically assumed to occur as point losses across a given structure.

I. The energy losses are added to the energy grade line when working upstream, or subtracted when working downstream. The HGL is then calculated by subtracting the velocity head from the energy grade line. Then plot the HGL on the storm drain profile plan sheet.

OPTIMIZATION OF SYSTEM

- **If HGL is too high, change pipe size and re-calculate HGL**
- **System should be designed to operate at or near gravity full flow**
- **Design should be evaluated using a higher check flood**

7.62 Optimization of system

V. Optimization of System

A. If the HGL is too high in a given reach, the pipe size or material must be changed which will require recalculation of the HGL.

B. As designed the system will operate basically at or near gravity full flow; however, if surcharging (pressure flow) is acceptable, the pipe sizes can be reduced and the system reanalyzed.

C. The initial design should be evaluated using a higher check flood and adjusted to reduce cost and risk, if necessary.

STORM DRAIN DESIGN USING HYDRAIN

- HYDRA module can be used for storm drain design
- Discharge may be calculated from Rational method, hydrologic simulation, or HYDRO analysis
- Can design or analyze sanitary, storm or combined systems, and provide cost estimate
- Cannot perform optimization

7.63 Storm drain design using HYDRAIN

VI. Storm Drain Design Using HYDRAIN

A. Storm drain analysis and design can be completed in HYDRAIN, using the HYDRA module.

B. HYDRA generates storm flows by using either the Rational method technique, hydrologic simulation techniques, or accepting a hydrograph generated by a HYDRO analysis.

C. It can be used to design or analyze storm, sanitary or combined collection systems. Additionally, HYDRA can be used for cost estimating.

D. Note that HYDRA is not an optimization program, thus individual case studies need to be run and analyzed by the designer/engineer.

NOTES:

LESSON 7

CLOSED-CONDUIT FLOW APPLICATIONS

PART 3: Culvert Design Demonstration and Workshop

I. Demonstration of Flow in a Culvert

Purpose:
To use the demonstration flume to illustrate outlet control, inlet control, and the effect of inlet type on culvert flow.

A. Demonstration 11

Outlet control conditions will be demonstrated with the double-barrel culvert insert in the demonstration flume. With the flume on a zero percent slope, the discharge will be increased to create full flow conditions in the pipe, with a low tailwater condition. The factors influencing outlet control will be evaluated and discussed during the demonstration.

NOTES:

B. Demonstration 12

Inlet control conditions will be demonstrated by increasing the flume slope. Under inlet control a free surface condition exists in the culvert barrel. Notice that initially, as the slope increases, the headwater remains constant and the culvert continues to operate in outlet control. However, eventually the headwater begins to increase and a free surface condition develops, indicating a change to inlet control. In the field, inlet control can occur at about a half of one percent slope for a smooth barrel.

The effect of inlet condition will be demonstrated with the single barrel culvert model. The changes in headwater for a sharp edge, miter, thick wall, headwall, groove end, headwall with a bevel, and side- and slope-tapered inlets will be demonstrated.

NOTES:

NOTES:

II. Culvert Design Workshop

Given: A culvert at a new roadway crossing must be designed to pass the 25-year flood. Hydrologic analysis indicates a peak flow rate of 6.0 m^3/s. The approximate culvert length is 60 m. and the natural stream bed slope approaching the culvert is 1 percent. The elevation of the culvert inlet invert is 600 m and the roadway elevation is 603 m. To provide some capacity in excess of the design flood, the desired headwater elevation should be at least 0.5 m below the roadway elevation. The tailwater for the 25-year flood is 1 m.

Find: The size of RCP culvert necessary for the 25-year flood.

1. The design will be completed using the culvert nomographs. The Culvert Design Form will be used to facilitate the trial and error design process. The Culvert Design Form provides a summary of all the pertinent design data, and a small sketch with important dimensions and elevations. All necessary nomographs are provided with this workshop. To expedite the solution process, begin with a 1500-mm RCP culvert.

2. The completed Culvert Design Form (see following page) indicates that a 1500-mm RCP with a (projecting) groove end entrance, operating under inlet control, will result in a headwater elevation that is ***0.9 m*** below the roadway.

3. The outlet velocity can be computed by calculating the full flow discharge (HDS-4, equation 51), and full flow velocity (from continuity), and then using the part-full flow relationships (HDS-4, figure 40) to find the V/V_f ratio given Q/Q_f.

NOTES:

PROJECT : ______________________	STATION : ______________ SHEET ____ OF ____	CULVERT DESIGN FORM DESIGNER / DATE : ________ / ______ REVIEWER / DATE : ________ / ______

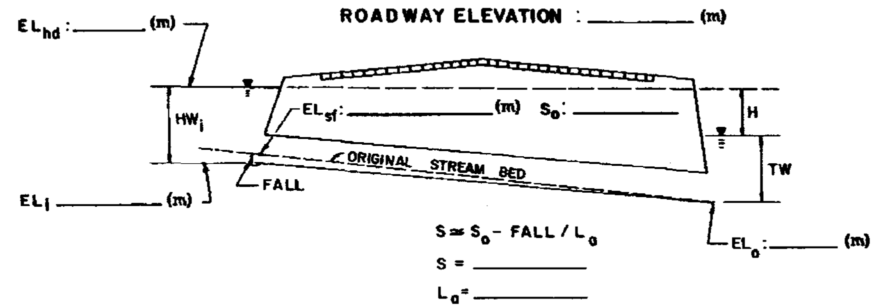
HYDROLOGICAL DATA

SEE ADD'L. SHTS.

- ☐ METHOD: ____________
- ☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______
- ☐ CHANNEL SHAPE: ____________
- ☐ ROUTING: ______ ☐ OTHER: ______

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (m^3/s)	TW (m)



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q	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	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) FALL = $HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) h_o = TW or $(d_c + D/2)$ (WHICHEVER IS GREATER)

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

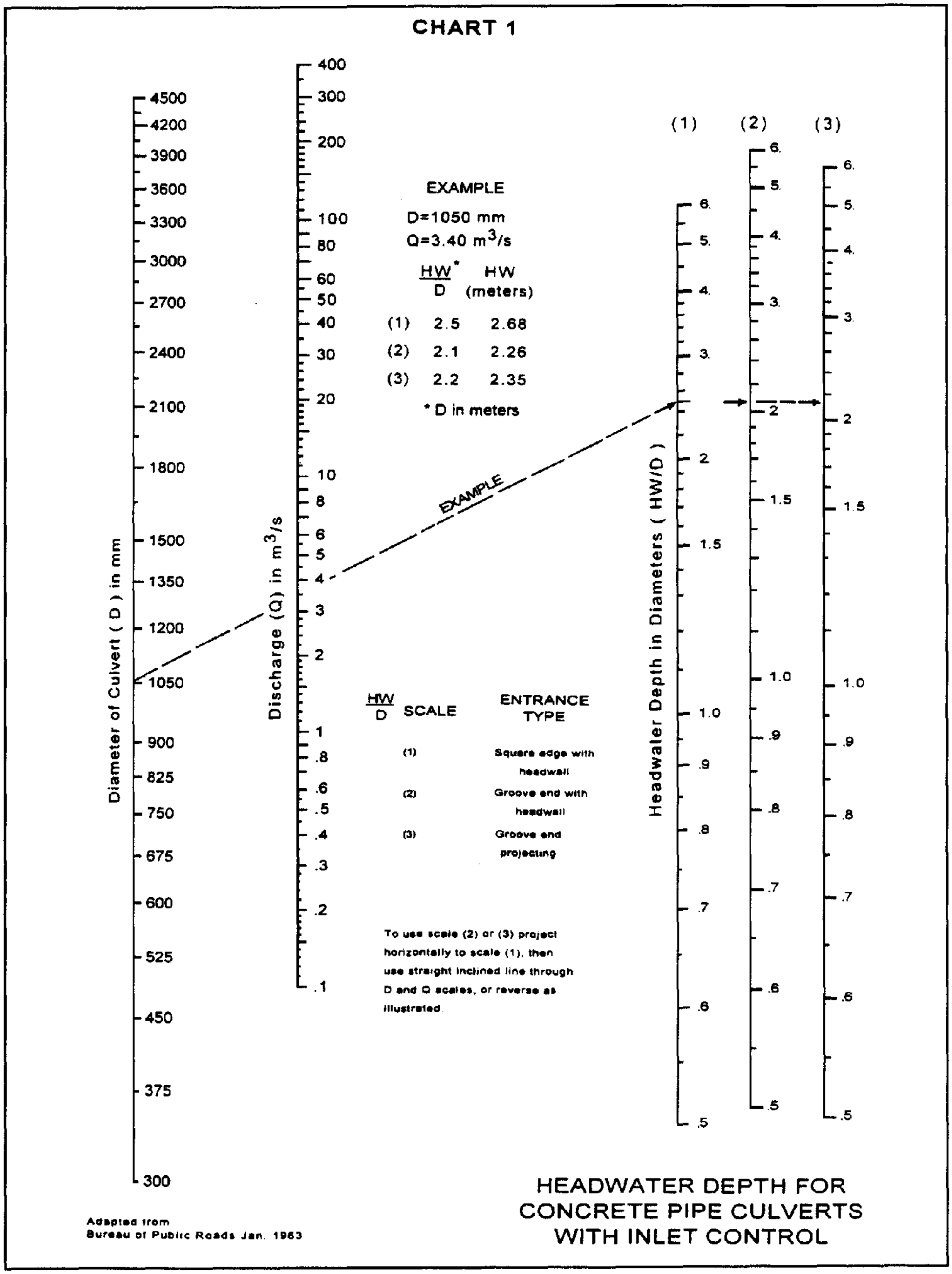
SUBSCRIPT DEFINITIONS :

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

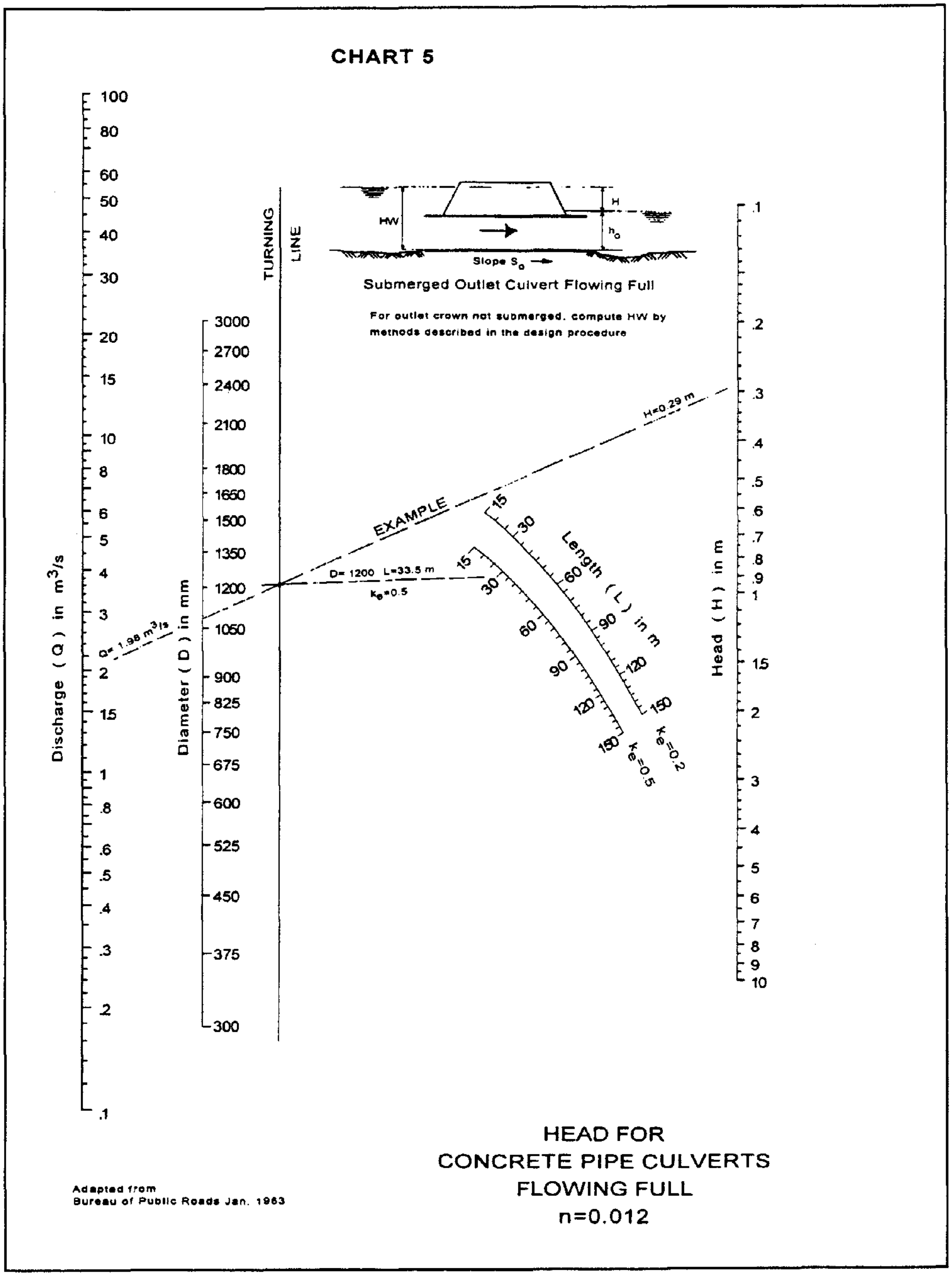
COMMENTS / DISCUSSION :

CULVERT BARREL SELECTED :

SIZE: ______________
SHAPE: ______________
MATERIAL: ________ n: ______
ENTRANCE: ______________

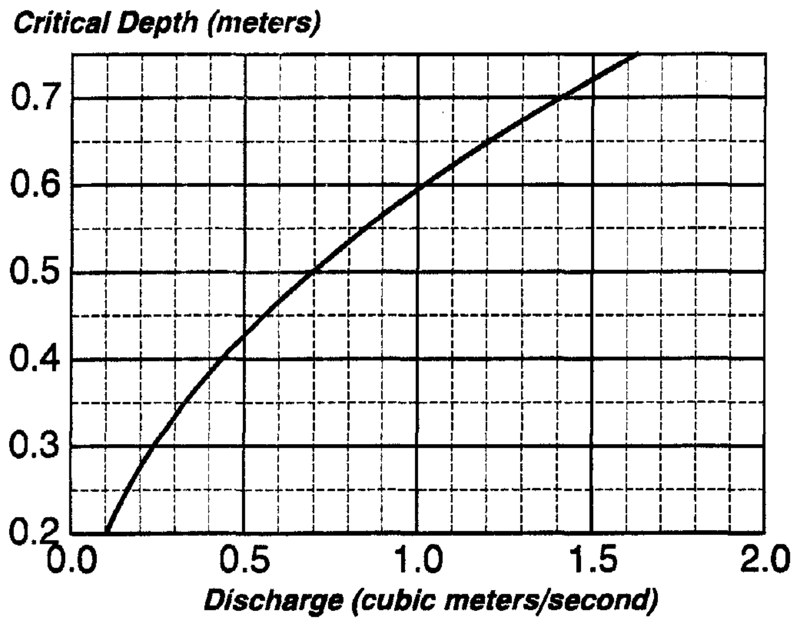


RCP inlet control nomograph (HDS-4, figure 47)

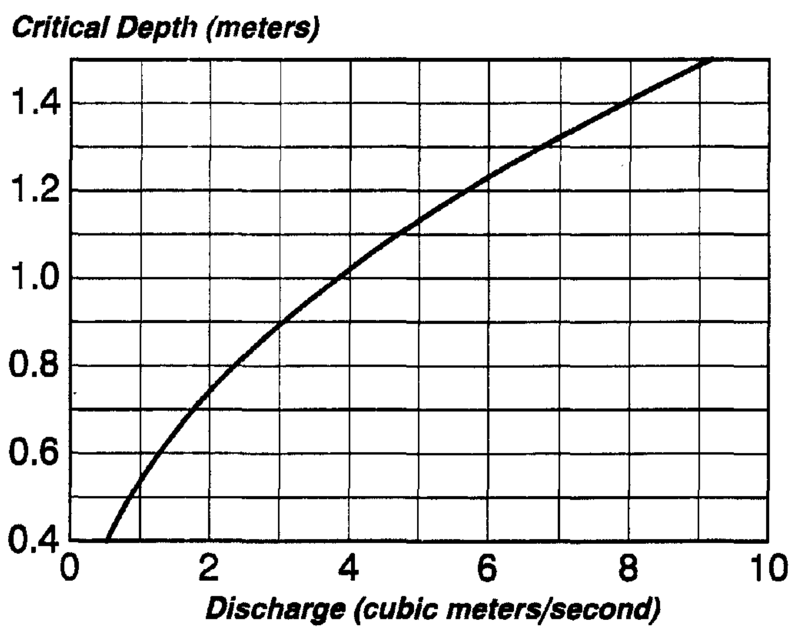


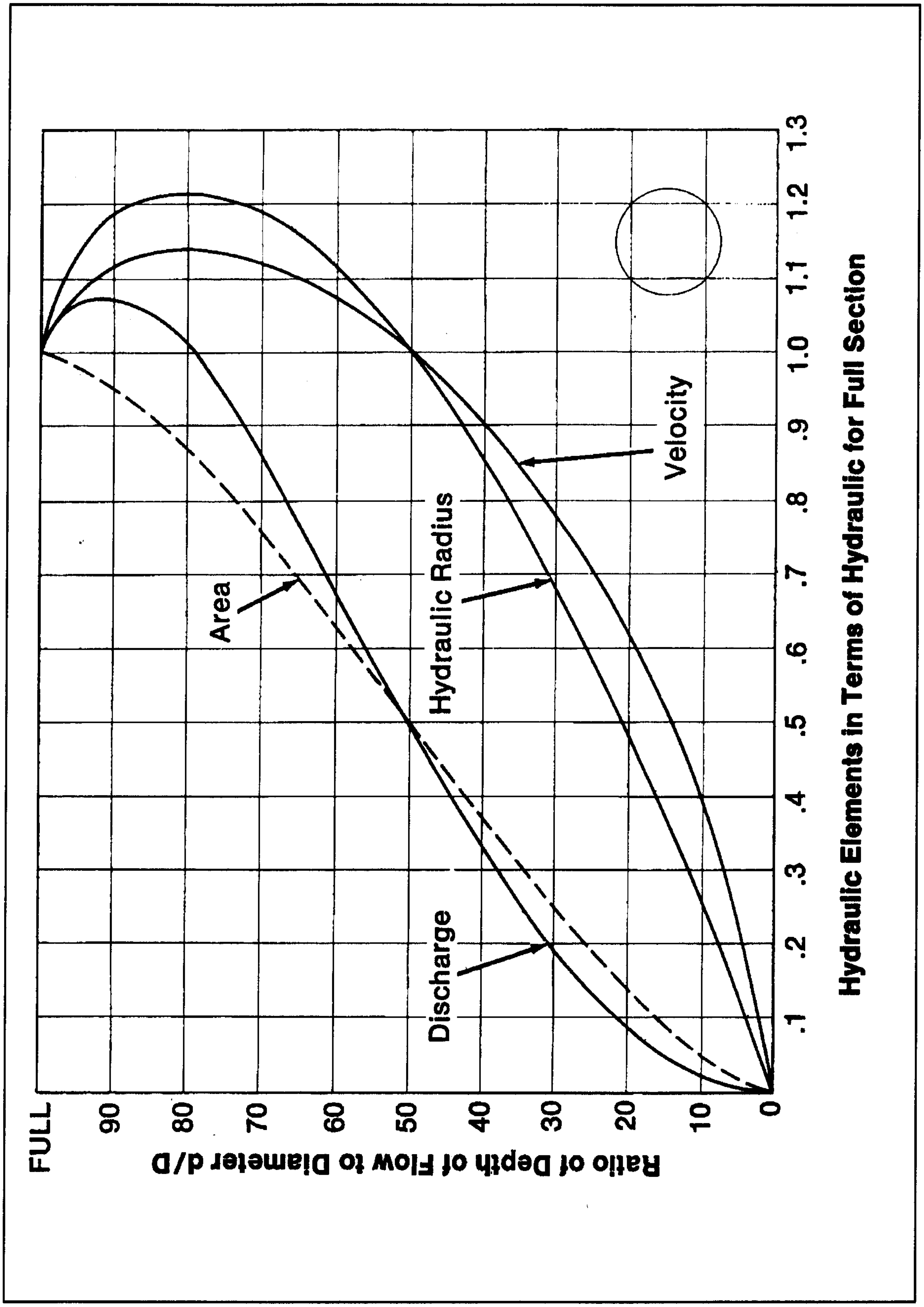
RCP outlet control nomograph (HDS-4, figure 48)

Critical Depth for 750 mm Circular Pipe



Critical Depth for 1500 mm Circular Pipe





Hydraulic Elements in Terms of Hydraulic for Full Section
Ratio of Depth of Flow to Diameter d/D
FULL
90
80
70
60
50
40
30
20
10
0
.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0
1.1
1.2
1.3
Velocity
Hydraulic Radius
Area
Discharge

NOTES:

LESSON 7

CLOSED-CONDUIT FLOW APPLICATIONS

PART 4: Storm Drain Design Demonstration and Workshop

I. Demonstration of Storm Drain Concepts

Purpose:

To discuss benching in access holes and to use the demonstration flume to qualitatively illustrate differences in storm drain inlet grates.

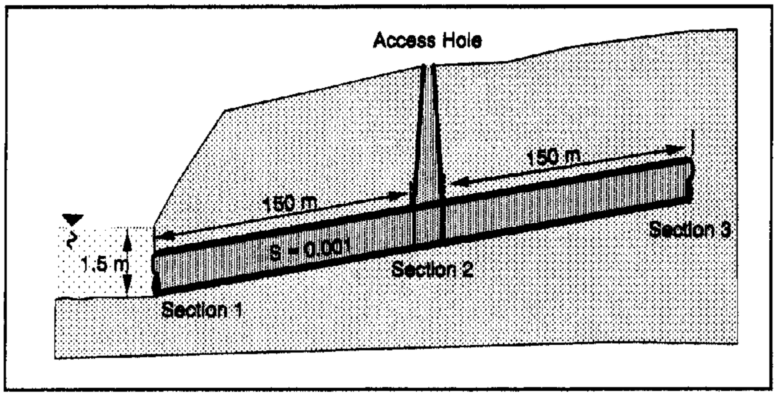
A. Demonstration 13

Storm drain inlet conditions will be demonstrated using a variety of different grate designs. The amount of flow intercepted depends on the momentum of the flow and the forces created by different inlet designs to counteract that momentum.

NOTES:

II. Storm Drain Design Workshop

Given: An RCP storm drain, 600 mm in diameter, carries 0.30 m^3/s on a 0.001 slope. The storm drain is 301.5 m long, with a 1.5 m (diameter) access hole in the middle. The HGL at the outlet is 1.50 m above a datum defined by the pipe invert. Calculate the HGL and EGL profiles.



Find: HGL
EGL

1. Check for pressure flow conditions: from the full-flow Manning's equation, the full flow capacity is

$$Q_{full} = \frac{0.312}{n} D^{8/3} S^{1/2}$$

2. Flow velocity in pipe

$$Q = VA, \; V = \frac{Q}{A}$$

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

3. Calculate the losses and apply energy equation

The HGL at the outlet (section 1) was given as 1.5 m using a datum at the invert of the pipe.

Assume:

$$Z_1 = 0$$

Then:

$$Z_2 = 150(0.001) = 0.15 \text{ m}$$

Apply the energy equation from the ponded water to section 1, the outlet of the pipe

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + Z_1 = \frac{V_p^2}{2g} + \frac{P_P}{\gamma} + Z_P + h_L$$

The headloss is the exit loss created at the outlet

$$h_L = K \frac{V^2}{2g}$$

For the exit, the value of K is 1.0 and the loss is

Apply the energy equation from section 1 to the downstream side of the access hole at section 2

$$\frac{V_{2_d}^2}{2g} + \frac{P_{2_d}}{\gamma} + Z_{2_d} = \frac{V_1^2}{2g} + \frac{P_1}{\gamma} + Z_1 + h_f$$

The headloss is due to friction.

$$h_f = L\ S_f = L \left(\frac{Qn}{0.312\ D^{8/3}} \right)^2$$

Therefore, the friction headloss from section 1 to 2 and from section 2 to 3 is

Access Hole Loss

For the access hole, the coefficient K is defined as

$$K = K_o \times C_D \times C_d \times C_Q \times C_p \times C_B$$

As provided in HDS-4, appendix B, the initial headloss coefficient K_o, is estimated as a function of the relative access hole size and angle between the inflow and outflow pipes:

$$K_o = 0.1 \times [\frac{b}{D_o}] \times [1 - \sin\theta] + 1.4 \times [\frac{b}{D_o}]^{0.15} \times \sin\theta$$

where

θ = Angle between the inflow and outflow pipes
b = Access hole diameter
D_o = Outlet pipe diameter

The correction factor for pipe diameter (C_D) is 1.0 since the incoming and outgoing pipe diameters are the same. Similarly, the correction factor for relative flow (C_Q) is 1.0 since the inflow and outflow discharge rates are the same, the plunging flow correction is 1.0 since there are not multiple inflows and with no benching $C_B = 1$.

The correction factor for flow depth C_d is a function of the HGL at the upstream end of the outlet pipe (1.72 m). Therefore,

$$C_d = 0.5 \left[\frac{d}{D_o}\right]^{3/5} =$$

The coefficient K is then

and the headloss is

$$h_L = K \frac{V^2}{2g} =$$

The headloss at a structure, such as a access hole, is typically assumed to be uniformly distributed across the structure. Apply the energy equation across the structure. Allow for a crown drop equal to the headloss through the access hole.

$$\frac{V_{2_u}^2}{2g} + \frac{P_{2_u}}{\gamma} + Z_{2_u} = \frac{V_{2_d}^2}{2g} + \frac{P_{2_d}}{\gamma} + Z_{2_d} + h_L$$

Then, applying the energy equation from section 2_u to section 3

$$\frac{V_3^2}{2g} + \frac{P_3}{\gamma} + Z_3 = \frac{V_{2_u}^2}{2g} + \frac{P_{2_u}}{\gamma} + Z_2 + h_f$$

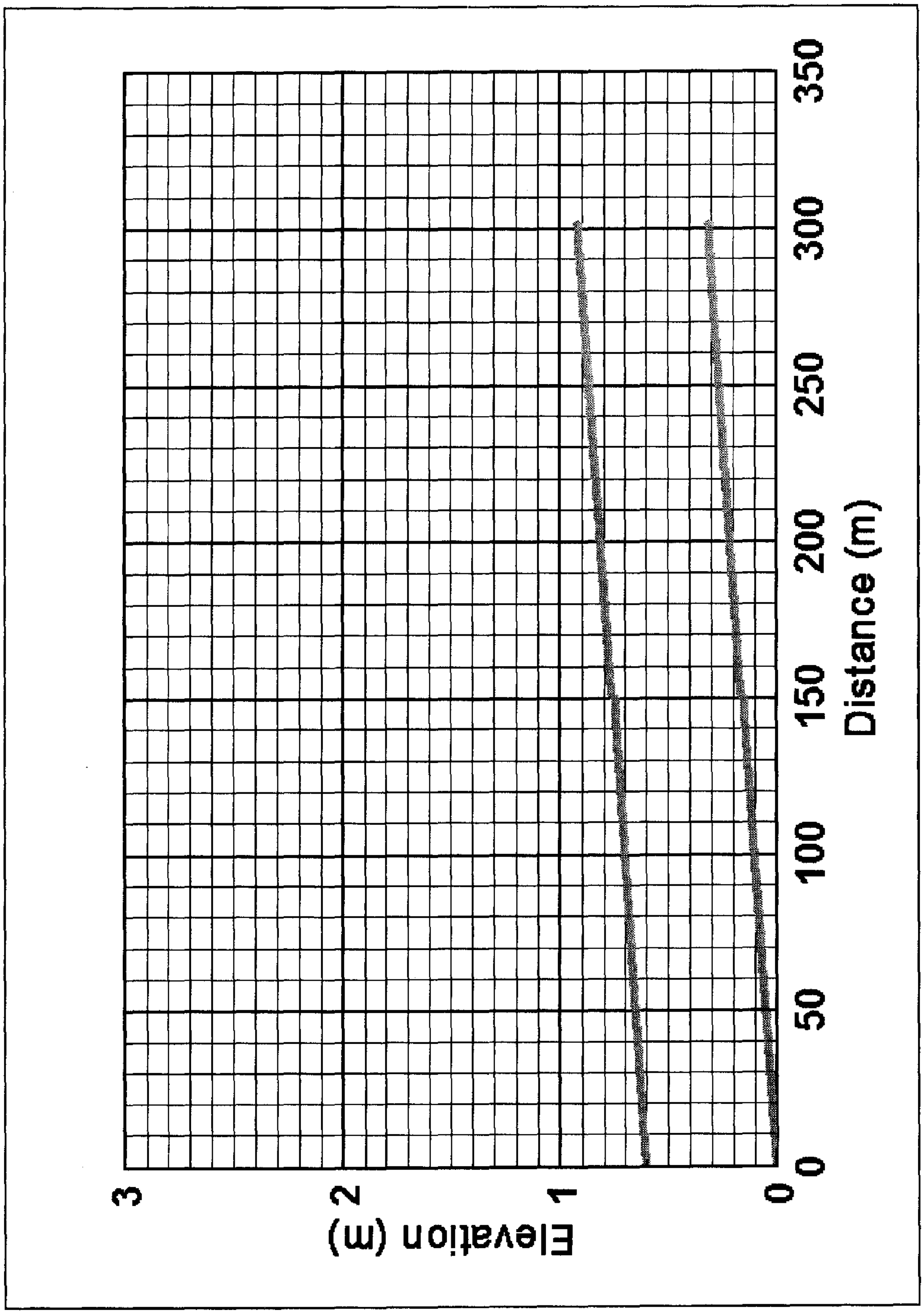
4. Calculate the HGL and EGL at each section

$$HGL = \frac{P}{\gamma} + Z$$

$$EGL = HGL + \frac{V^2}{2g}$$

Section	P/γ	Z	HGL	$V^2/2g$	EGL
1					
2d					
2u					
3					

5. Plot the pipe profile, HGL and EGL on the graph paper below:


Elevation (m)
0
1
2
3
Distance (m)
0
50
100
150
200
250
300
350

LESSON 8

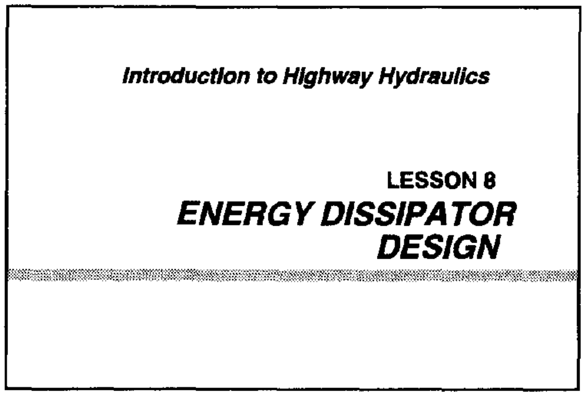
ENERGY DISSIPATOR DESIGN

OVERVIEW: Method of Instruction: Lecture

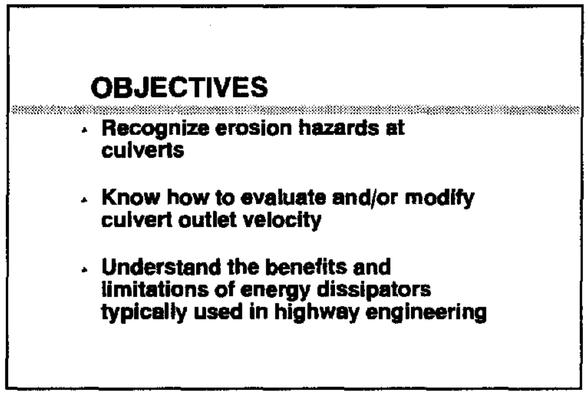
Lesson Length: 45 minutes

Resources:

Lesson Outline
Slides
HDS-4, Chapter 8
Video



8.0 Title



8.1 Objectives

OBJECTIVES: At the conclusion of this lesson, the Participant should:

1. Recognize erosion hazards at culverts.

2. Know how to evaluate and/or modify culvert outlet velocity.

3. Understand the benefits and limitations of energy dissipators typically used in highway engineering.

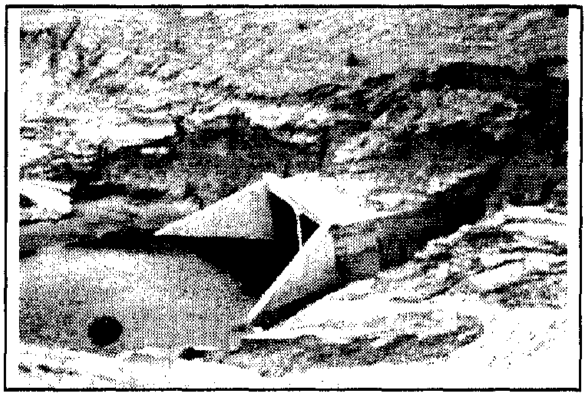
LESSON 8

ENERGY DISSIPATOR DESIGN

GENERAL DESIGN CONCEPTS

- **Erosion is a natural phenomenon that is sometimes accelerated by highway activities**
- **Energy dissipating devices are sometimes necessary**

8.2 General design concepts



8.3 Scour problem

I. General Design Concepts

A. Erosion is part of the natural environment; however, erosion is often increased by construction of a highway.

B. To protect the highway and adjacent areas it is sometimes necessary to employ an energy dissipating device.

GENERAL DESIGN CONCEPTS (cont)

- **Energy dissipators should be an integral part of overall design**
  - **energy dissipator requirements may change with changes in culvert design**
  - **downstream conditions impact energy dissipator design**

8.4 General design concepts (cont)

C. Energy dissipators should be considered as an integral part of the overall system, not something that is "added on" since:

1. Energy dissipator requirements may be reduced, increased or possibly eliminated by changes in the culvert design.

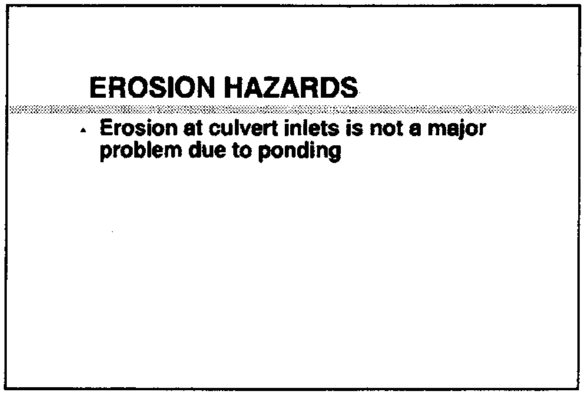
2. Downstream channel conditions (velocity, depth and channel stability) will impact the selection and design of appropriate energy dissipation devices.

GENERAL DESIGN CONCEPTS (cont)

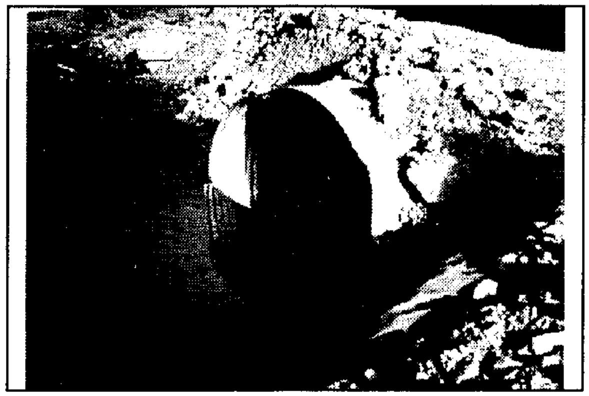
- **Primary objective of an energy dissipator is to prevent excessive damage due to erosion**
  - **return flow to the downstream channel in a condition that approximates normal conditions**
  - **reducing flow conditions below "normal conditions" not necessarily beneficial**
- **HEC-14 is primary design reference**

8.5 General design concepts (cont)

D. Throughout the design process, the designer should keep in mind that the primary objective is to protect the highway structure and adjacent area from excessive damage due to erosion. One way to accomplish this objective is to return flow to the downstream channel in a condition that approximates the natural flow regime.



8.6 Erosion hazards



8.7 Flow conditions at culvert inlet

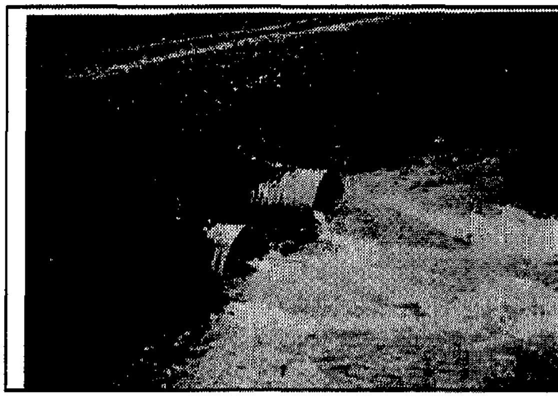
II. Erosion Hazards

A. Erosion at a culvert inlet is typically not a major problem since water normally ponds at the inlet.

EROSION HAZARDS

- Erosion at culvert inlets is not a major problem due to ponding
- Erosion at culvert outlets is a common problem
  - evaluation of erosion at the outlet should be completed for all culvert designs
  - the only safe solution is to assume that erosion will occur, and provide appropriate protection

8.8 Erosion hazards (cont)



8.9 Flow conditions at culvert outlet

B. Erosion Hazards at Culvert Outlets

1. Erosion at culvert outlets is a common problem and should be evaluated in the design of all highway culverts.

2. Ultimately, the only safe procedure is to design on the basis that erosion at a culvert outlet and downstream channel will occur, and must be protected against.

CULVERT OUTLET VELOCITY

- Continuity equation is always applicable to evaluate outlet velocity
  - discharge
  - flow area (inlet or outlet control)
- Outlet velocity is an indicator of erosion potential
- modification of barrel velocity possible; but often of limited value

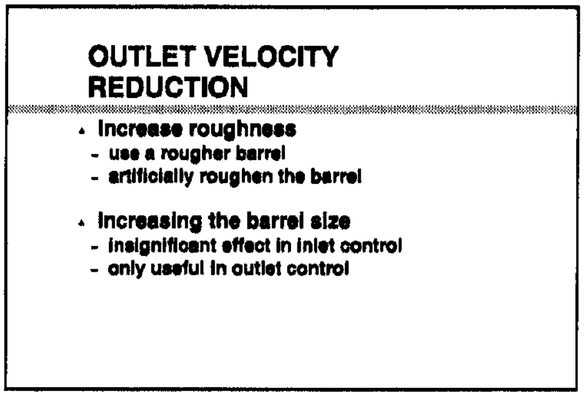
8.10 Culvert outlet velocity

III. Culvert Outlet Velocity

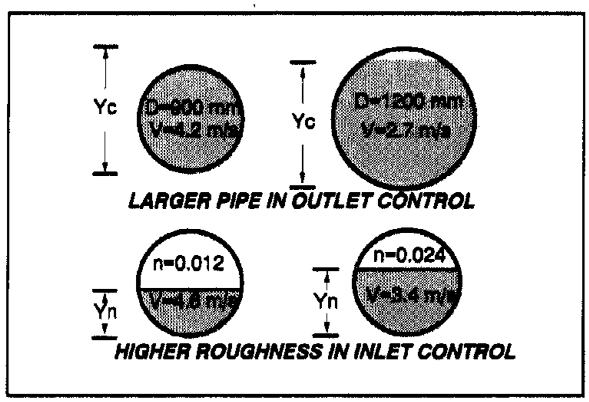
A. The continuity equation can be used to compute culvert outlet velocity, either within the barrel or at the outlet. Given the design discharge, the only other information needed is the flow area, and it is a function of the type of control (outlet or inlet).

B. Culvert outlet velocity is one of the primary indicators of erosion potential.

C. If the velocity is higher than in the downstream channel, measures to modify or reduce velocity within the culvert barrel should be considered. However, the degree of velocity reduction is typically limited and must be balanced against the increased costs generally involved.



8.11 Outlet velocity reduction



8.12 Ways to reduce outlet velocity

D. Outlet Velocity Reduction

1. Under inlet control, use a pipe with a higher roughness (e.g., CMP) or artificially increase roughness (e.g., roughness rings).

2. Increasing the barrel size is a possibility in outlet control. However, this approach may not be cost-effective. Another alternative to reduce velocity is to flare the outlet wingwalls and use a paved apron. Under inlet control when partially full flow occurs, the flow area will not change significantly with a larger pipe size or wingwall flare; therefore, the velocity will not be reduced much.

3. HEC-14 discusses the various methods of creating additional roughness and details the appropriate design procedures.

HYDRAULIC JUMP
ENERGY DISSIPATORS

- **Hydraulic jumps naturally occur**
- **Considerable turbulence and loss of energy occurs in a hydraulic jump**
- **Hydraulic jump energy dissipators force a jump to occur**
- **Typical structures used in highway engineering**
  - **CSU rigid boundary basin**
  - **SAF basin**
  - **USBR type IV basin**

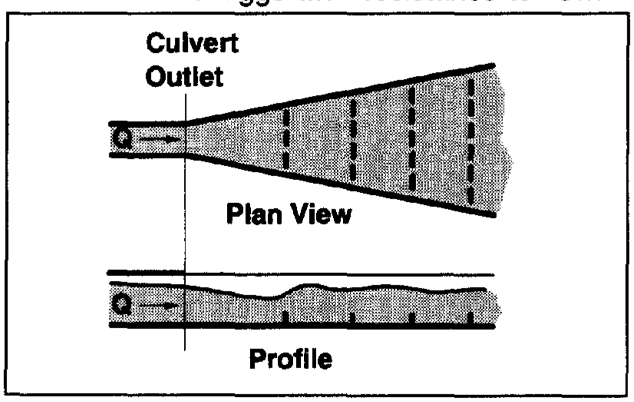
8.13 Hydraulic jump energy dissipators

IV. Hydraulic Jump Energy Dissipators

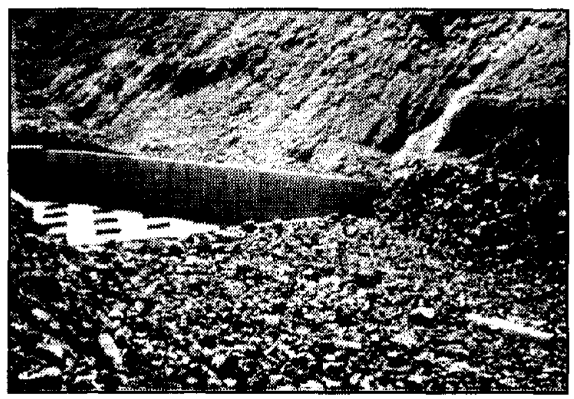
A. The hydraulic jump is a natural phenomenon which occurs when supercritical flow changes to subcritical flow.

B. This abrupt change in flow condition in a hydraulic jump is accomplished by considerable turbulence and loss of energy, making the hydraulic jump an effective energy dissipation device.

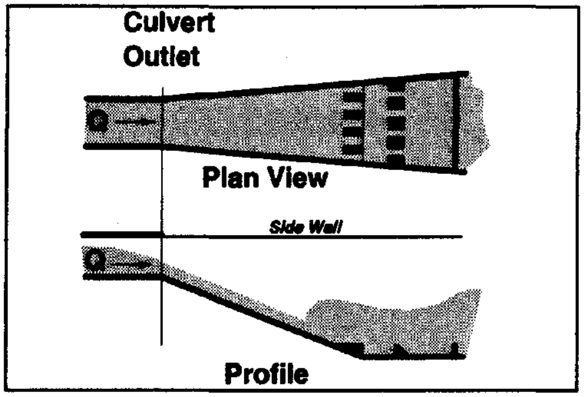
C. To better define the location and length of a hydraulic jump, standard design structures have been developed to force the hydraulic jump to occur. These structures typically use blocks, sills or other roughness elements to impose exaggerated resistance to flow.



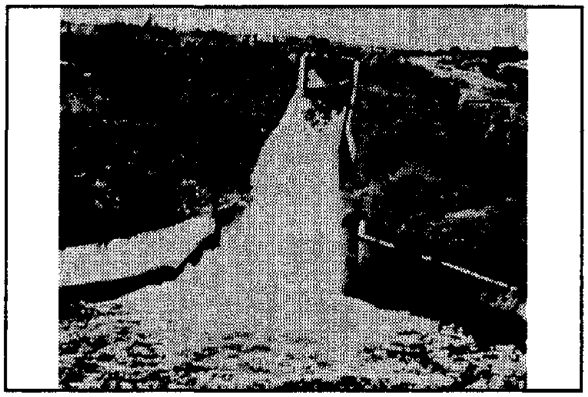
8.14a Schematic of CSU rigid boundary basin



8.14b CSU rigid boundary basin



8.15a Schematic of SAF stilling basin



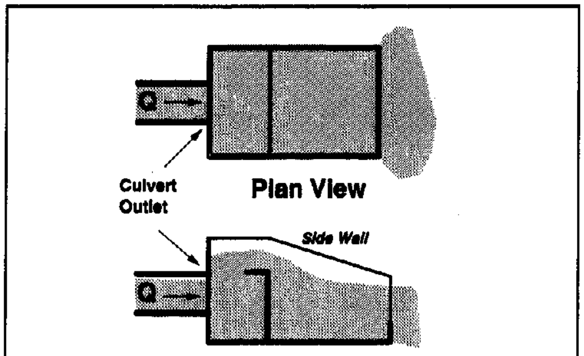
8.15b SAF stilling basin

D. Forced hydraulic jump structures applicable in highway engineering include the CSU rigid boundary basin, the St. Anthony Falls basin and the USBR type IV basin.

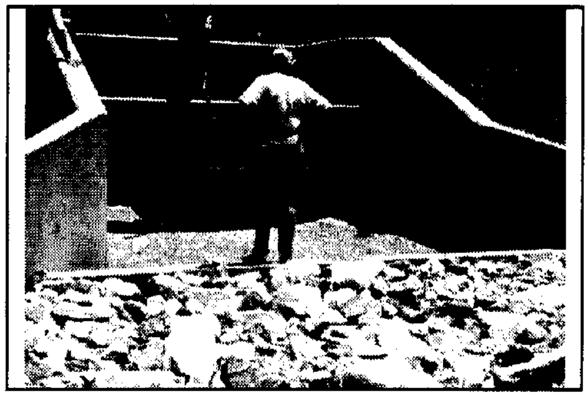
IMPACT BASIN ENERGY DISSIPATORS

- **Impact basins are structures that physically block free discharge**
- **The impact creates turbulence and loss of energy**
- **Typical impact basin used in highway engineering is the USBR type VI stilling basin**

8.16 Impact basins



8.17a Schematic of USBR Type VI

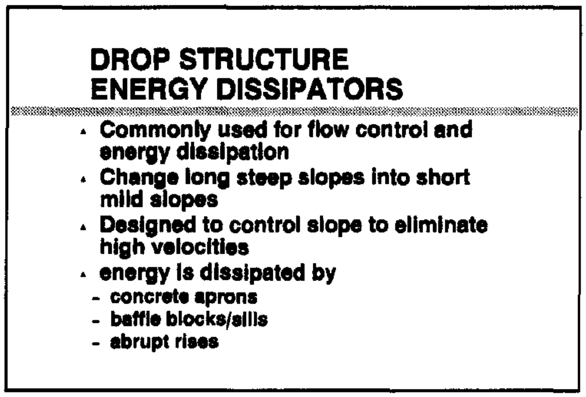


8.17b Baffle-wall energy dissipator - USBR Type VI

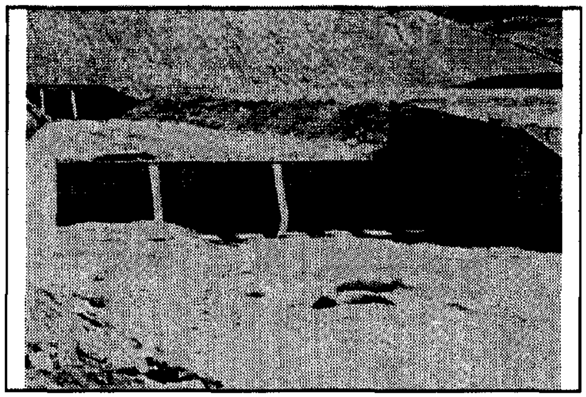
V. Impact Basins

A. As the name implies, impact basins are designed with part of the structure physically blocking the free discharge of water. The action of water impacting on the structure dissipates energy and modifies the downstream flow regime.

B. Impact basins include the Contra Costa Energy Dissipator, Hook type energy dissipator, and the USBR Type VI Stilling Basin.



8.18 Drop structures



8.19 Straight drop spillway stilling basin

VI. Drop Structures with Energy Dissipation

A. Drop structures are commonly used for flow control and energy dissipation.

B. Reducing channel slope by placing drop structures at intervals along the channel changes a continuous steeper sloped channel into a series of milder sloped reaches with vertical drops.

C. Instead of slowing down and transferring high erosion producing velocities into lower nonerosive velocities, drop structures control the slope of the channel so that high velocities never develop.

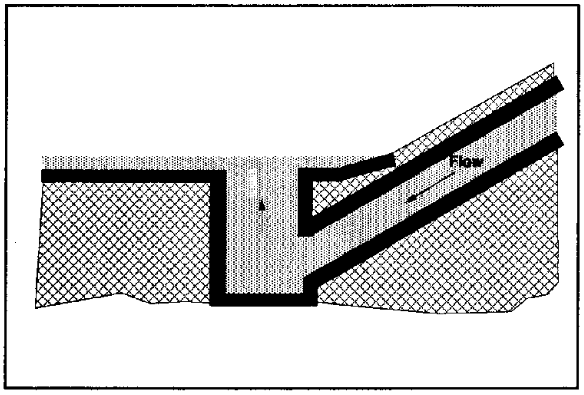
D. The kinetic energy or velocity gained by the water as it drops over the crest of each structure is dissipated by specially designed aprons or stilling basins.

E. The stilling basin used to dissipate excess energy can vary from a simple concrete apron to an apron with flow obstructions such as baffle blocks, sills, or abrupt rises. The length of the concrete apron required can be shortened by addition of these appurtenances.

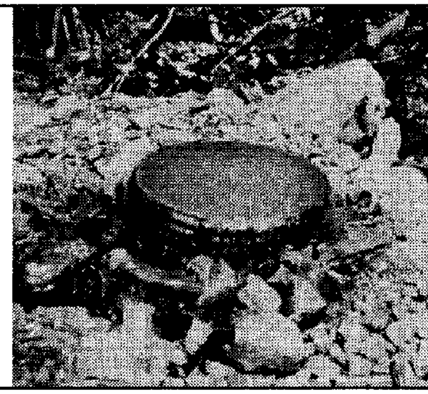
STILLING WELL ENERGY DISSIPATORS

- Stilling wells dissipate energy by forcing water vertically upward
- Stilling wells most applicable where debris is not a problem
- Typical highway applications include pipe down drains

8.20 Stilling wells



8.21a Schematic of COE stilling well



8.21b COE stilling well

VII. Stilling Wells

A. Stilling wells dissipate kinetic energy by forcing flow to travel vertically upward to reach the downstream channel.

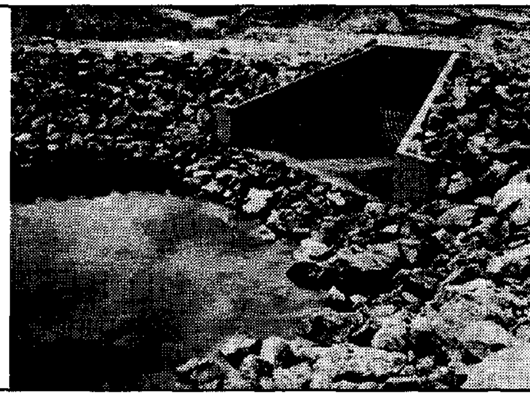
B. The stilling well most commonly used in highway engineering is the Corps of Engineers Stilling Well. This stilling well has application where debris is not a serious problem.

C. Its greatest application in highway engineering is at pipe down drains where little debris is expected. It is recommended that riprap or other types of channel protection be provided around the stilling well outlet.

RIPRAP STILLING BASINS

- Riprap stilling basins commonly used at culvert outlets
- Design based on model study tests of scour holes at culvert outlets
- Additional energy dissipation occurs from mound of rock downstream of scour hole
- General design procedure is to preshape the scour hole and line it with riprap

8.22 Riprap basins



8.23 Riprapped culvert energy dissipator

VIII. Riprap Stilling Basins

A. Riprap stilling basins are commonly used at culvert outfalls.

B. The design procedure for riprap energy dissipators was developed from model study tests. The results of this testing indicated that the size of the scour hole at the outlet of a culvert was related to the size of the riprap, discharge, brink depth and tailwater depth.

C. The mound of rock material that often forms on the bed downstream of the scour hole contributes to dissipation of energy and reduces the size of scour hole.

D. The general design guidelines for riprap stilling basins include preshaping the scour hole and lining it with riprap.

ENERGY DISSIPATOR DESIGN USING HY-8

- HYDRAIN module HY-8 implements HEC-14 design procedures

8.24 Design using HY-8

IX. Energy Dissipator Design Using HY-8

A. Energy dissipator design for culvert outlets can be completed within the HY-8 module of HYDRAIN. The design is based on FHWA publication HEC-14.

X. FWHA Video "Energy Dissipators for Highway Structures"

LESSON 9

COMPREHENSIVE EXAMPLE

OVERVIEW:	Method of Instruction:	Workshop	
	Lesson Length:	Overview	30 minutes
		Problem Solving/Design:	
		Channel Design	60 minutes
		Storm Drain Design	60 minutes
		Culvert Design	60 minutes

Resources:

Lesson Outline
HDS-4

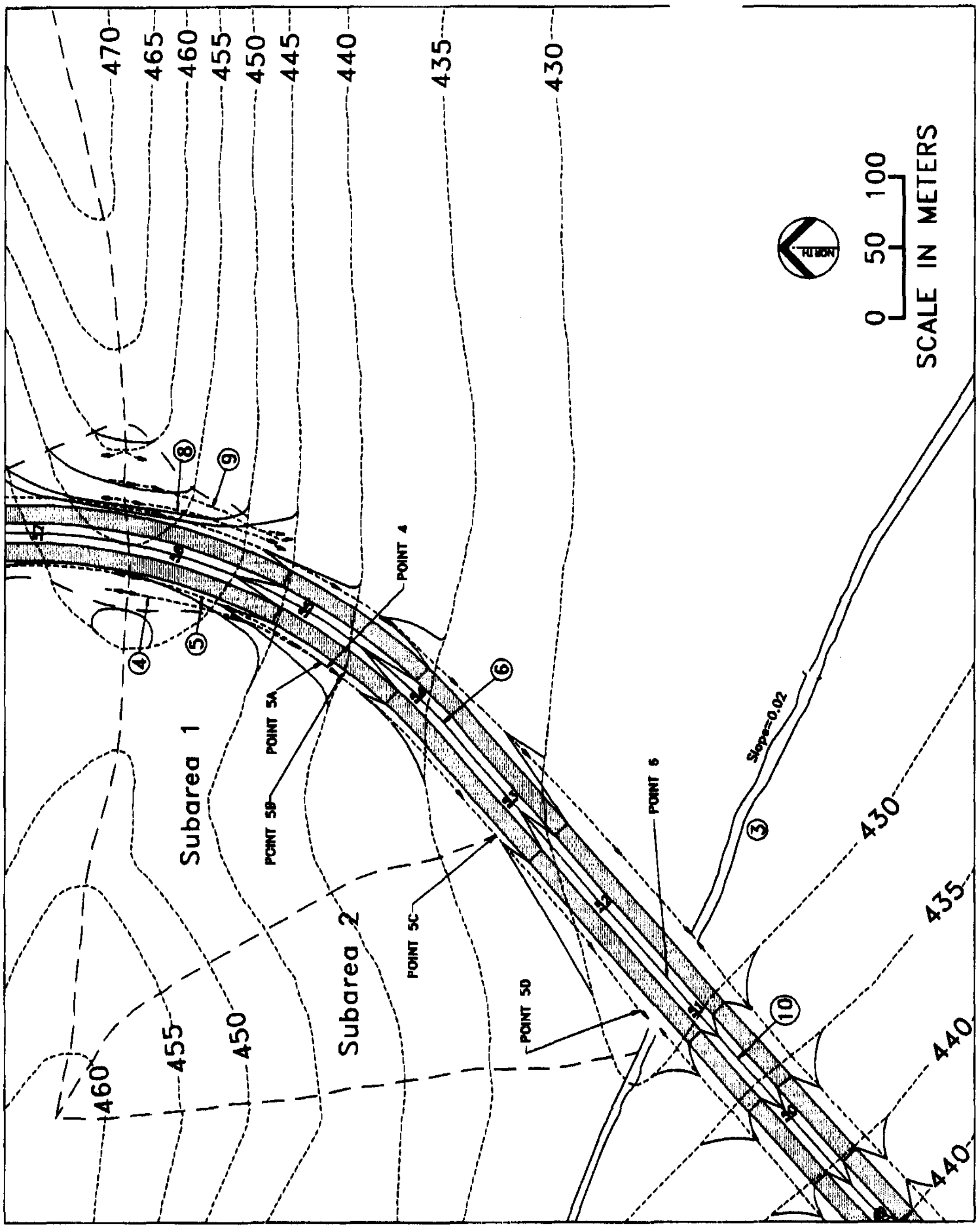
OBJECTIVES: At the conclusion of this lesson, the Participant will have worked on a "real world" highway drainage design problem, will have applied specific drainage calculation/design procedures presented in this course, and should have a better understanding of highway drainage engineering.

LESSON 9

COMPREHENSIVE EXAMPLE

I. Problem Statement

Many of the procedures discussed during the last three days will be used to design various drainage components of an interstate highway project. The layout of the drainage system is shown in the plan view map. The grading for the project has defined channel courses within the highway right-of-way, or in the case of the cut slopes within an easement. The original topographic contours are shown as dotted lines, and the revised contours based on the grading for the roadway are shown as solid lines. The stationing for the roadway is given by the solid line in the median swale between the north and southbound lanes.



470
465
460
455
450
445
440
435
430
Subarea 1
Subarea 2
POINT 4
POINT 5A
POINT 5B
POINT 5C
POINT 5D
POINT 6
Slope=0.02
0 50 100
SCALE IN METERS

The drainage channels are indicated by dashed lines and are numbered for easy reference. The points for which channel sizes are calculated are indicated by letters. Roadway and toe-of-slope channels are used to capture the runoff within the right-of-way and carry water to the natural drainage course. Intercepting channels are used on the cut slopes to capture off-site drainage and to protect the cut slopes from erosion. Note that intercepting channels 8 and 9 dispose of water by distributing it over the hillside, while intercepting channel 4 conducts water to channel 5. A lining for channel 9 was designed in Part 3 of Lesson 5. Drainage components considered in this example include the roadway/toe-of-slope channels 4 and 5, and a storm drain at the lower end of channel 5. There is not enough time in the lesson to complete every component of the design; therefore, some portions of the design are provided.

The design will be completed for a 10-year discharge. For all Rational method calculations use the IDF curve provided at the end of this lesson. This IDF curve was calculated using HYDRAIN. A metric engineering scale and additional nomographs necessary for solution are also provided on a separate page at the end of this lesson.

II. Calculations

A. Channel 4 Design

This channel drains a grassed cut slope. The first step was to use the Rational method to define the 10-year design discharge. The kinematic wave formula was used for the overland flow t_c and the shallow concentrated flow nomograph for the channel that drains the cutslope.

The total t_c was calculated to be 6.8 minutes and the computed Rational method runoff was 0.05 m^3/s for a drainage area of 0.2 ha with C = 0.7.

The second step was to determine an adequate channel geometry to carry the computed flow. Given the small area and flow on a bench surface, a symmetrical V-shaped ditch was assumed with 1V:3H sideslopes.

An iterative solution of the Manning's equation found Y = 0.11 m. From continuity, the velocity V = 1.4 m/s.

B. Channel 5 Design

Reach 5A

This channel drains the cutslope below channel 4, and both southbound lanes for a distance of 160 m. The area of the roadway consists of 2 shoulders (3 m wide) and 2 lanes (each 4 m wide). Therefore, the pavement area is

pavement area = 14 m x 160 m = 0.22 ha, which has a C = ______

The ditch area is 2.2 m wide by 160 m long, so that its area is 0.04 ha. The cut slope area is 0.2 ha, and the C for slope and ditch are both _____ (grass cover). Therefore, the total area is

A =

Application of the Rational method will be based on a weighted C value.

	<u>A</u>	<u>C</u>
Pavement		
Grass		

Weighted C =

Next, determine the time of concentration. The overland flow t on the cut slope will be based on the kinematic wave formula with n = 0.5. A trial and error solution of the kinematic wave formula is required by assuming a value for t, finding the corresponding intensity on the IDF curve, and comparing the calculated t with the assumed t. The overland flow slope is

$$S = \frac{449\ m - 444\ m}{15\ m} = \frac{5\ m}{15\ m} = 0.33$$

$$t = 6.99\ \frac{n^{0.6}\ L^{0.6}}{i^{0.4}\ S^{0.3}} =$$

Assume shallow concentrated flow exists in the channel. Scaling from the map, the toe-of-slope channel has a slope of

$$S = \frac{444\ m - 437\ m}{160\ m} = \frac{7\ m}{160\ m} = 0.04$$

From the shallow flow nomograph (HDS-4, figure 4), V = ***1.0*** m/s.

Therefore,

t =

To confirm that the cutslope flow path represents the longest t, compute the t for the roadway. For the roadway area assume n = 0.02, and a typical cross slope of 2 percent.

Use i = 150 mm/h in the kinematic wave equation

$t =$

The total t_c is calculated by:

$$t_c = t_{cut\ slope} + t_{channel}$$

$t_c =$

Checking the IDF curve with a t_c = 7.1 min, i = ______ mm/h. The discharge is then

$$Q = \frac{CiA}{360} =$$

Assuming n = 0.025, use Manning's equation to define hydraulic conditions. Given that this channel is adjacent to the roadway, the channel front slope (cross slope adjacent to the roadway) should be flatter than the backslope for safety should a vehicle leave the roadway. Assume a V-shaped channel with a 1V:3H backslope and a 1V:6H front slope, where

$$A = BY + \left(\frac{Z_1 + Z_2}{2}\right) Y^2$$

$$P = B + Y\left(\sqrt{1 + Z_1^2} + \sqrt{1 + Z_2^2}\right)$$

$$0.14 = \frac{1}{0.025}(0 + 4.5Y^2)\left[\frac{0 + 4.5Y^2}{0 + Y(\sqrt{1 + 3^2} + \sqrt{1 + 6^2})}\right]^{2/3}(0.04)^{1/2}$$

By iteration Y = 0.15 m, and by continuity V = 1.4 m/s. This is similar to the channel hydraulic conditions calculated for channel 4.

Reach 5B

At concentration point 5B, calculate the combined discharge immediately below the confluence of channel 4 with channel 5. Note that this is not necessarily the sum of the two channel discharges. The discharge will be calculated with the Rational method using the longest t_c from the total area to find i. First find the weighted C:

A C

C =

Next, find i. The t_c for channel 4 was _____ min, and _____ min for channel 5A; therefore, use t_c = _____ min. From the IDF curve, i = ______ mm/h. The area is the total area of all contributing watersheds, which is _____ ha + _____ ha = _______ ha.

Therefore,

$$Q = \frac{CiA}{360} =$$

In this case, the sum of the two discharges (______________) is equal to the discharge for the combined areas and longest t_c.

Reach 5C

The discharge at concentration point 5C should be based on the longest t_c, which may result from subarea 1 or from the t_c at point 5B plus travel time in the swale to point 5C. The t_c at point 5B was 7.1 min. The travel time in the swale, based on the previously calculated velocity of 1.4 m/s and a travel distance of 170 m, would be 170/1.4 = 2.0 min. Therefore, the total t_c for this flow path would be 9.1 min.

The t_c based on a flow path through subarea 1 draining to point 5C would include both overland flow and shallow flow components.

For the overland flow component, the kinematic wave equation was used with an assumed maximum allowable overland flow length of 130 m. The computed t = 34.4 min.

The travel time beyond 130 m was based on the shallow concentrated flow nomograph. The shallow flow travel distance is 370-130 = 240 m. Using the nomograph (HDS-4, figure 4), V = 1.3 m/s for S = 0.08, and

$$t = \frac{240}{1.3} = 1.85\ S = 3.1\ \text{min.}$$

Therefore, the total time of concentration is

$$t_c = 34.4\ \text{min} + 3.1\ \text{min} = 37.5\ \text{min}$$

The time of concentration for this flow path is longer than for the flow path from point 5B; therefore, 37.5 min was used to compute the intensity in the Rational method. Based on a weighted C value, the peak discharge was computed to be 1.01 m^3/s.

This channel is also adjacent to the roadway and a V-shaped channel was used with a backslope of 1V:3H and a front slope of 1V:6H. Solving the Manning's equation by iteration results in Y = 0.31 m, and from continuity V = 2.3 m/s. The shear stress ($\tau = \gamma y S$) created by these hydraulic conditions is 121 Pa.

<u>Reach 5D</u>

Next determine the discharge at point 5D, based on the longest t_c, which may result from overland flow from subarea 2 or from the t_c at point 5C plus additional channel travel time. The t_c at point 5C was 37.5 min. The travel time in the swale, based on the previously calculated velocity of 2.3 m/s and a travel distance of 160 m, would be 160/2.3 = 1.2 min. Therefore, the total t_c for this flow path would be 38.7 min.

The t_c based on a flow path through subarea 2 draining to point 5D would include both an overland flow and a shallow flow components. For the overland flow component, kinematic wave equation was used with an assumed maximum allowable flow length of 130 m. The computed t = 34.4 min.

The travel time beyond 130 m was based on the shallow concentrated flow nomograph. The shallow flow length is 430 m - 130 m = 300 m on a slope of 8 percent. Using the nomograph (HDS-4, figure 4), V = 1.3 m/s and

$$t = \frac{300\ \text{m}}{1.3\ \text{m/s}} = 231\text{s} = 3.8\ \text{min.}$$

The total time of concentration is

t_c = 34.4 min + 3.8 min = 38.2 min

In this case, the time of concentration for this overland flow path is less than the flow path from point 5C; therefore, 38.7 min was used to compute the intensity in the Rational method. Based on a weighted C value, the computed discharge is 1.83 m^3/s.

This is another roadway channel and so a backslope of 1V:3H and a front slope of 1V:6H was assumed. Solving the Manning's equation by iteration, Y = 0.41 m, and from continuity V = 2.4 m/s. The shear stress created by these hydraulic conditions is 121 Pa.

Evaluate Channel Stability Below Concentration Point 5C

Is an open channel suitable below concentration point 5C? Compare the computed shear stress with allowable shear stress criteria.

C. Storm Drain Design

A storm drain beginning at concentration point 5C will discharge into the stream channel just upstream of an existing culvert. The headwater elevation (HW) created by this culvert defines the hydraulic control point at the outlet of the storm drain. In order to design the storm drain, the culvert HW for the 10-year flood must be calculated.

Culvert Analysis

The RCP culvert length is 70 m, D = 1500 mm, S = 2 percent, and the upstream invert elevation of the culvert is 421 m. The 10-year discharge in the stream channel is 8.5 m^3/s. A backwater analysis calculated the tailwater at the outlet to be 1 m. Using the culvert design form determine the HW assuming the inlet condition is a groove-end in a headwall.

For Inlet Control:

For Outlet Control:

The controlling condition is that which produces the highest headwater. In this case, the culvert is under ________ control since______ > ______.

The outlet velocity is calculated by comparing the design discharge with the discharge at full flow.

Design Q = 8.5 m^3/s

$$\text{Full flow } Q_o = \frac{0.312}{0.012}\, 1.5^{8/3}\, 0.02^{1/2}$$

$$Q_o = 10.8 \text{ m}^3/\text{s}$$

Thus we get

$$Q/Q_o = \frac{8.5 \text{ m}^3/\text{s}}{10.8 \text{ m}^3/\text{s}} = 0.78$$

Using the part-full flow nomograph, we find V/V_o = ______

$$V_o = \frac{Q_o}{A_o} = \frac{10.8 m^3/s}{\frac{\pi\ (1.5)^2}{4}} = 6.11 m/s$$

$$V = 1.1\ (6.11\ m/s) = 6.7\ m/s$$

The results of the culvert analysis are provided on the culvert design form on the following page.

PROJECT : ______________________

____

STATION : ______________________

SHEET ______ OF ______

CULVERT DESIGN FORM

DESIGNER / DATE : ____________ / ________

REVIEWER / DATE : ____________ / ________

HYDROLOGICAL DATA

SEE ADD'L. SHTS.

☐ METHOD : ______________________

☐ DRAINAGE AREA : ________ ☐ STREAM SLOPE : ________

☐ CHANNEL SHAPE : ______________________

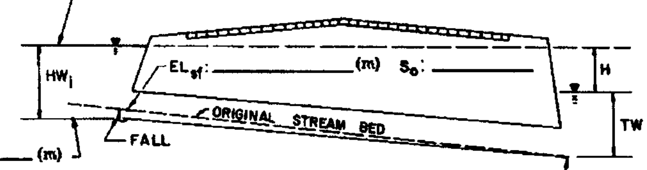
☐ ROUTING : ________ ☐ OTHER : ________

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (m^3/s)	TW (m)

EL_{hd} : ________ (m)

ROADWAY ELEVATION : ________ (m)



EL_i ________ (m)

EL_o : ________ (m)

$S = S_o - FALL / L_a$

S = ________

L_a = ________

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW	FLOW PER BARREL	HEADWATER CALCULATIONS											CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
			INLET CONTROL				OUTLET CONTROL									
	Q	Q/N (1)	HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H	EL_{ho} (8)			

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(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 CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

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

SUBSCRIPT DEFINITIONS :

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION :

CULVERT BARREL SELECTED :

SIZE : ______________________

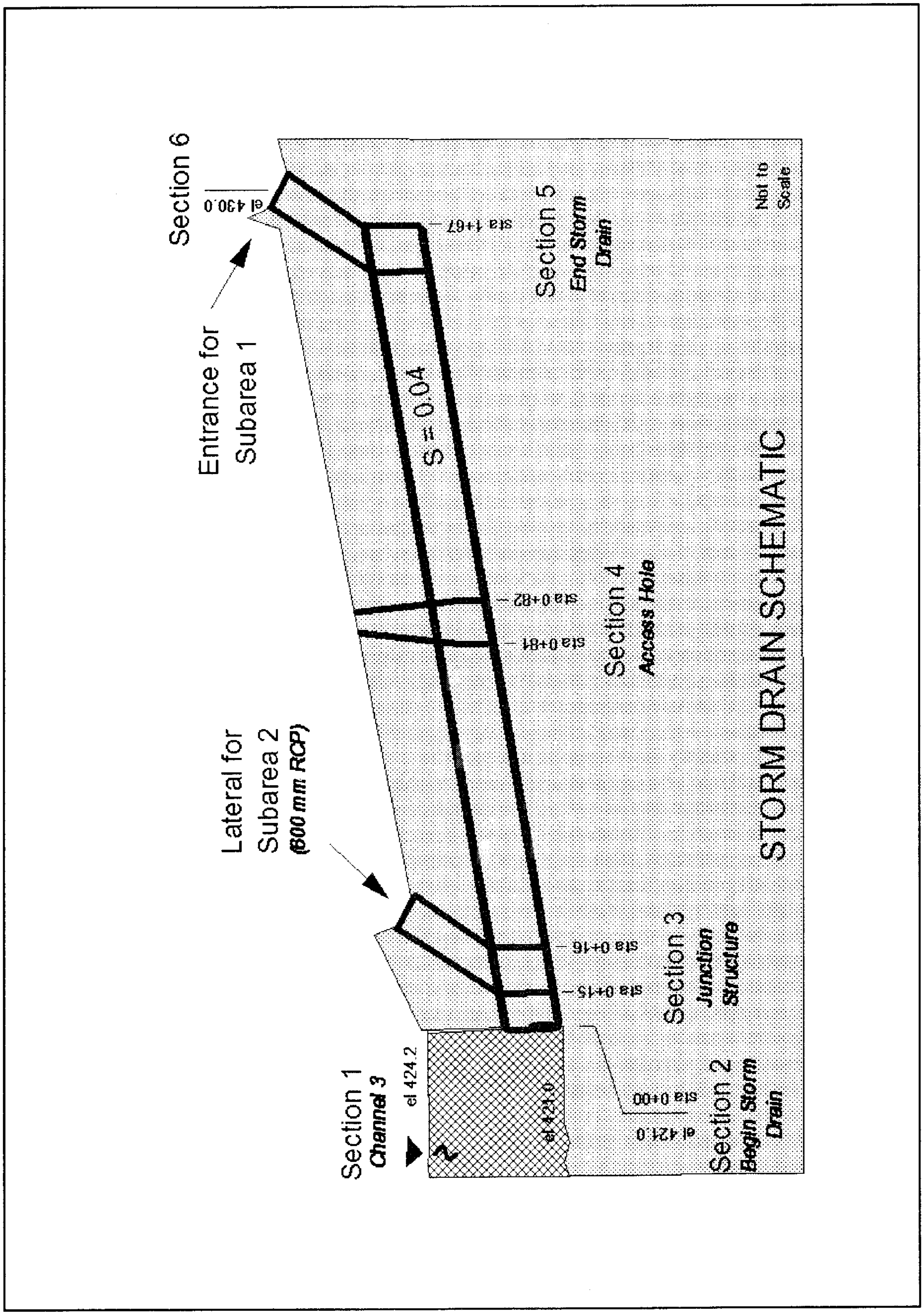
SHAPE : ______________________

MATERIAL : ____________ n: ________

ENTRANCE : ______________________

Storm Drain Design

Assume the storm drain will be constructed on a slope of 4 percent. From the culvert analysis, a controlling downstream water surface elevation of 424.2 exists. At the outlet, the invert of the storm drain will be at the channel's invert. A junction structure will be required to bring in flow from subarea 2 (concentration point 5D), and an inlet at the upstream end will collect subarea 1 (concentration point 5C) flow. Assume that the storm drain daylights into the swale for the subarea 1 inlet. The discharge at concentration point 5D was based on a t_c from concentration point 5C plus travel time in the ditch. With a storm drain pipe the travel time from 5C to 5D will be less. Consequently, the overland flow path will now control the t_c. The t_c for the overland flow path was nearly identical to the ditch flow path, and so, the design discharge at 5D remains the same. Locate a 1 m access hole with no benching at station 0+80. The following figure summarizes the station/elevation information necessary for HGL evaluation.



Section 6
el 430.0
Entrance for Subarea 1
sta 1+67
Section 5
End Storm Drain
S = 0.04
sta 0+82
sta 0+81
Section 4
Access Hole
Lateral for Subarea 2
(600 mm RCP)
sta 0+16
sta 0+15
Section 3
Junction Structure
Section 1
Channel 3
el 424.2
el 421.0
sta 0+00
el 421.0
Section 2
Begin Storm Drain
STORM DRAIN SCHEMATIC
Not to Scale

Begin the design by calculating the preliminary storm drain size using the full flow Manning's equation. For purposes of this example, use a constant storm drain size for the entire project.

$$Q = \frac{0.312}{n} D^{8/3} S^{1/2}$$

Based on nominal pipe sizes, use a ____ mm diameter pipe.

Using the energy equation, begin calculating at section 1 and move to 2, then 2 to 3d and so on.

$$\frac{V_2^2}{2g} + \frac{P_2}{\gamma} + Z_2 = \frac{V_1^2}{2g} + \frac{P_1}{\gamma} + Z_1 + h_L$$

h_L in this case is the headloss due to expansion where the flow enters the stream. Therefore,

$$h_L = K \frac{V_2^2}{2g}, \quad \text{where } K = 1 \qquad A = \frac{\pi(0.75)^2}{4} = 0.44 m^2$$

$$V_2 = \frac{Q}{A} =$$

$$h_L =$$

Substituting into the energy equation to solve for P_2/γ:

$$\frac{(4.2)^2}{2(9.81)} + \frac{P_2}{\gamma} + 421 = \frac{(0)^2}{2(9.81)} + 3.2 + 421 +$$

(Note that this is the depth of water at the outlet of the storm drain.)

Now calculate from 2 to 3d, that is, from the pipe's outlet, to the downstream side of the junction, where flow from concentration point 5D enters the storm drain.

$$\frac{V_{3d}^2}{2g} + \frac{P_{3d}}{\gamma} + Z_3 = \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + Z_2 + h_f$$

Here, h_f represents the losses due to friction in the pipe. The hydraulic radius is

$R =$

and

$$h_f = L\left[\frac{Qn}{AR^{2/3}}\right]^2 =$$

Apply the energy equation to solve for P_{3d}/γ

Going across the junction

$$\frac{V_{3u}^2}{2g} + \frac{P_{3u}}{\gamma} + Z_3 = \frac{V_{3d}^2}{2g} + \frac{P_{3d}}{\gamma} + Z_3 + h_L$$

Now, h_L represents the form loss through the junction. Based on junction loss relationships (see HDS-4, section 6.3.2), the computed $h_L = 0.22$ m.

Apply the energy equation to solve for P_{3u}/γ, allowing for a crown drop equal to the headloss across the junction.

Now, calculate the pipe reach from 3_u to 4_d

$$\frac{V_{4d}^2}{2g} + \frac{P_{4d}}{\gamma} + Z_{4d} = \frac{V_{3u}^2}{2g} + \frac{P_{3u}}{\gamma} + Z_{3u} + h_f$$

The headloss due to friction is 0.53 m and the pressure head at 4d is computed to be

$$\frac{P_{4d}}{\gamma} = 1.56$$

Going across the access hole

$$\frac{V_{4u}^2}{2g} + \frac{P_{4u}}{\gamma} + Z_{4u} = \frac{V_{4d}^2}{2g} + \frac{P_{4d}}{\gamma} + Z_{4d} + h_L$$

Now, h_L represents the form loss through the access hole.

This is calculated by

$$h_L = K\left[\frac{V^2}{2g}\right] \text{ where } K = K_o \times C_D \times C_d \times C_Q \times C_P \times C_B$$

$$K_o = 0.1[\frac{b}{D}] (1\text{-sin } \theta) + 1.4 [\frac{b}{D}]^{0.15} \text{ SIN } \theta$$

$$= 0.1 [\frac{1}{0.75}] (1\text{-sin } 180) = 0.13$$

$$C_D = [\frac{D_o}{D_i}]^3; \quad \textit{since } D_o = D_i, \ C_D = 1$$

$$C_d = 0.5 \left(\frac{d}{D}\right)^{3/5} = 0.5 \left[\frac{1.56}{0.75}\right]^{3/5} = 0.78$$

$$C_Q = (1\text{-}2 \sin \theta) \left(1 - \frac{Q_i}{Q_o}\right)^{0.75} + 1$$

= 1.0 since there are not multiple inflows

$$C_p = 1 + 0.2 \times [\frac{h}{D_o}] \times [\frac{h - d}{D_o}]$$

= 1 since there is no plunging flow condition.

With no benching (flat floor condition), $C_B = 1.0$.

Then, $K = K_o \times C_D \times C_d \times C_Q \times C_p \times C_B$

$h_L =$

Apply the energy equation to solve for P_{4u}/γ, given the invert elevation at 4_d of 421.6 + 0.22 + 0.04(65) = 424.42

Now calculate the pipe reach from 4_u to 5

$$\frac{V_5^2}{2g} + \frac{P_5}{\gamma} + Z_5 = \frac{V_{4u}^2}{2g} + \frac{P_{4u}}{\gamma} + Z_{4u} + h_f$$

The headloss due to friction is 0.69 m and the pressure head at 5 is computed to be

$$\frac{P_5}{\gamma} = -1.15$$

The negative pressure indicates that the pipe becomes unsealed somewhere between point 4_u and 5; therefore, further analysis is required. First, determine if supercritical flow is occurring away from the influence of the high downstream tailwater. Calculate critical depth and normal depth for the pipe.

Critical depth can be determined from nomographs or by solving HDS-4, equation 36. Using the critical depth figure provided with this example, for the given discharge and pipe diameter, d_c = _____.

Normal depth is calculated by comparing the design discharge to the full flow discharge and using the hydraulic properties figure to determine the depth.

$$Q_o = \frac{0.312}{0.013} (0.75)^{8/3} (0.04)^{1/2} = 2.23 \text{ m}^3/\text{s}$$

$$\frac{Q_5}{Q_o} = \frac{1.01}{2.23} = 0.45$$

From the hydraulic properties figure

$$\frac{d_5}{D_o} = 0.48 \text{ and } \frac{V_5}{V_o} = 0.97$$

Therefore,

V_5 = 2.2 m/s and

d_5 = 0.48 (0.75) = 0.36 m

Since 0.36 is less than critical depth of 0.60 m, supercritical flow exists and a hydraulic jump occurs somewhere between point 4_u and 5.

Assume that the storm drain daylights into the drainage swale at concentration point 5c, with a berm across the swale to pond the water and force it into the storm drain. Calculate the HGL at this entrance (section 6) based on inlet control using the RCP inlet control nomograph. For Q = 1.01 m^3/s and a 750 mm RCP culvert with the groove end in a headwall, the HW/D ratio is ______ so that, HW = ____________________________.

Therefore, at the entrance (section 6) the HGL = EGL = ____________________.

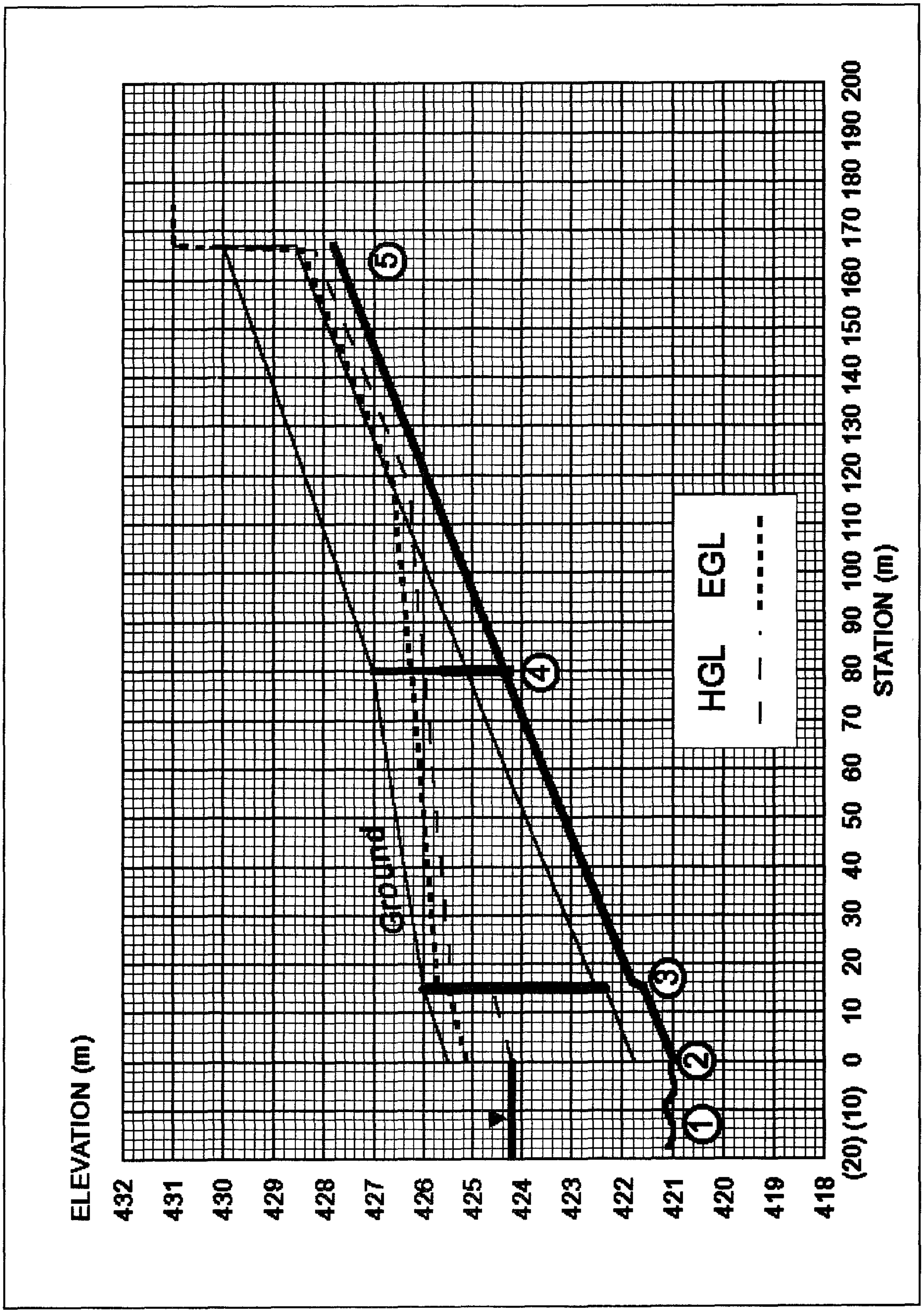
Then, assume normal depth flow will exist in the storm drain immediately upstream of the location of the jump. For purposes of storm drain design, the jump can be approximately located at the intersection of the upstream and downstream energy grade lines.

The EGL in the normal depth region is

$$EGL = \frac{(2.2)^2}{2(9.81)} + 0.36 + Z$$

The intersection of the upstream and downstream EGL's are located graphically and the HGL can then be sketched, as shown on the following page.

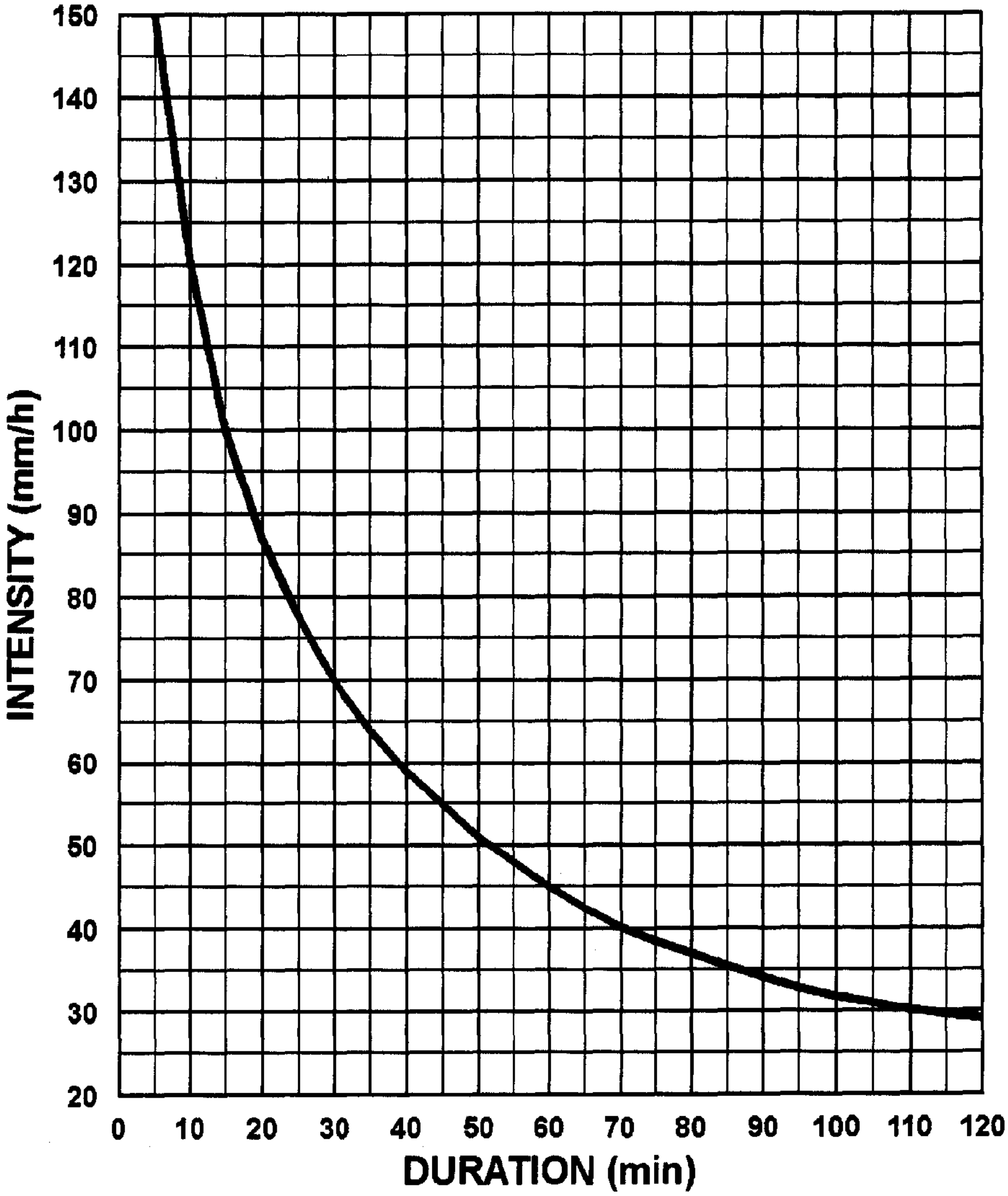
Fill in the table provided to tabulate the results of the calculation.



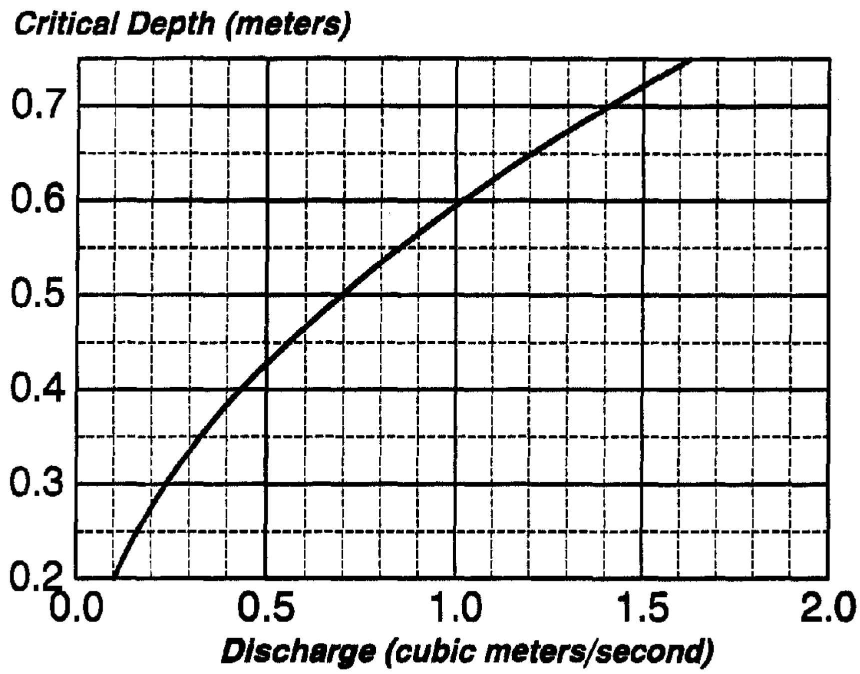
ELEVATION (m)
432
431
430
429
428
427
426
425
424
423
422
421
420
419
418
STATION (m)
(20)
(10)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
Ground
HGL
EGL
1
2
3
4
5

Section	$\frac{P}{\gamma}$	Z	HGL	$\frac{V^2}{2g}$	EGL
1					
2					
3d					
3u					
4d					
4u					
5					
6					

IDF CURVE
10-YR EVENT



Critical Depth for 750 mm Circular Pipe



Critical Depth for 1500 mm Circular Pipe

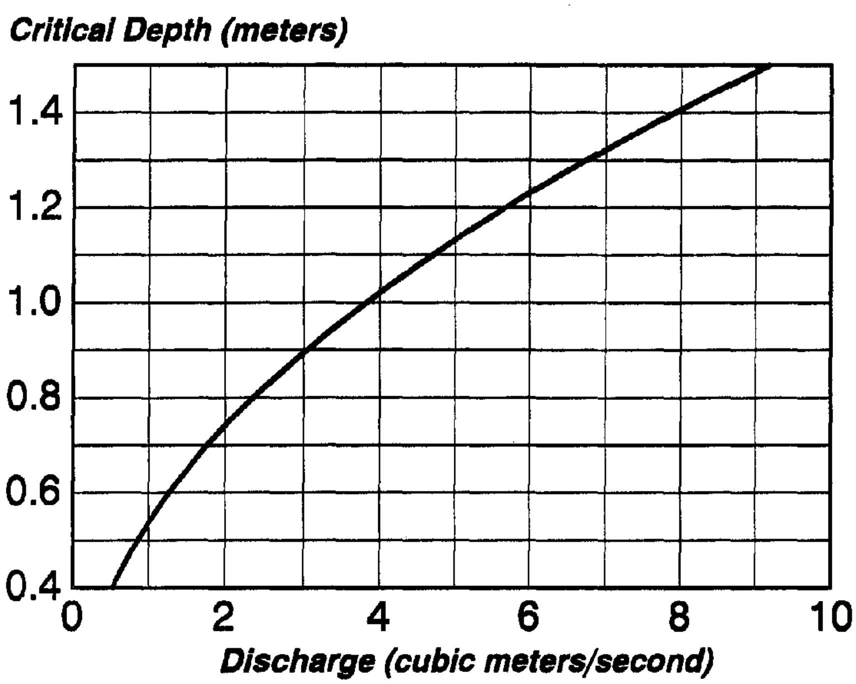
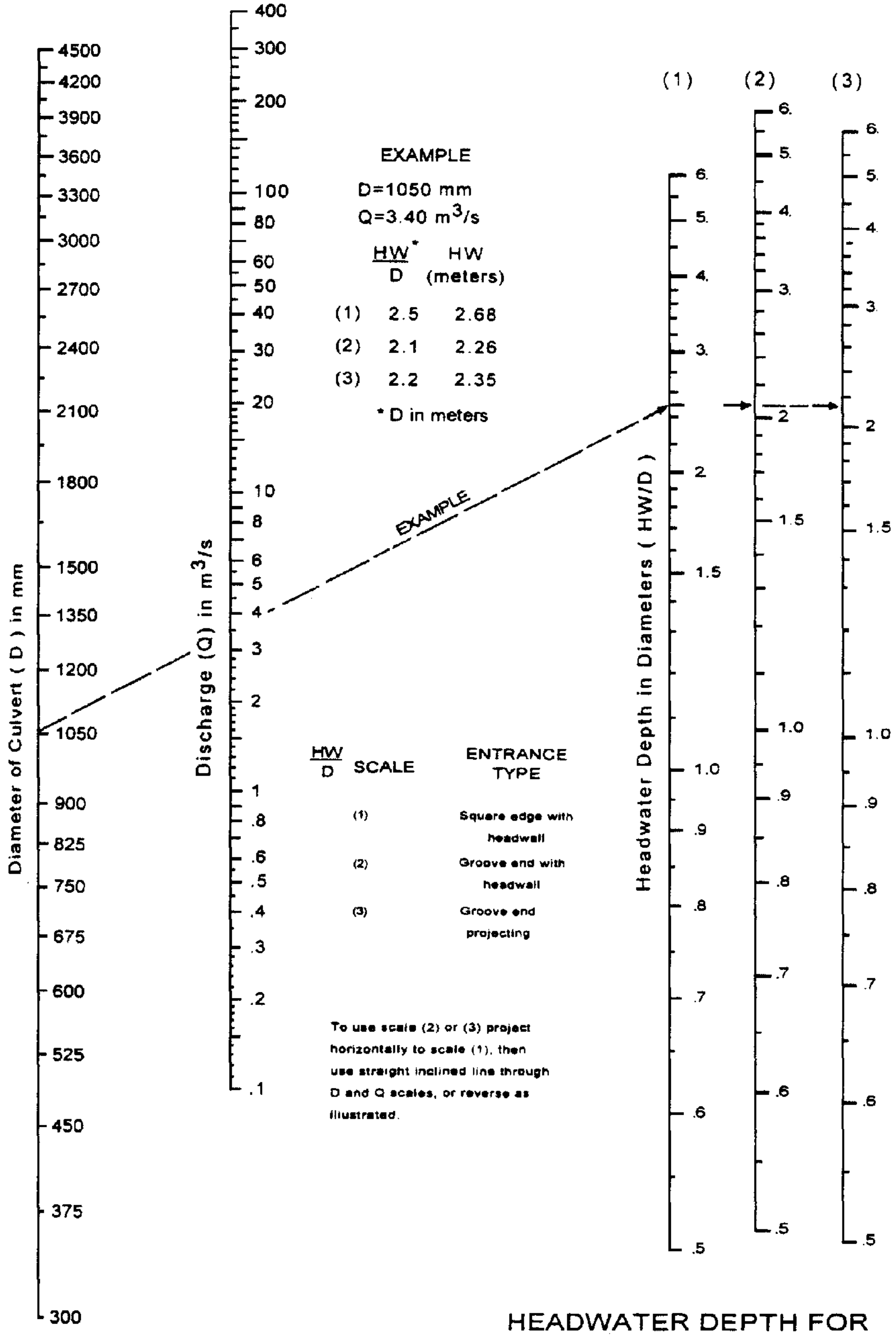


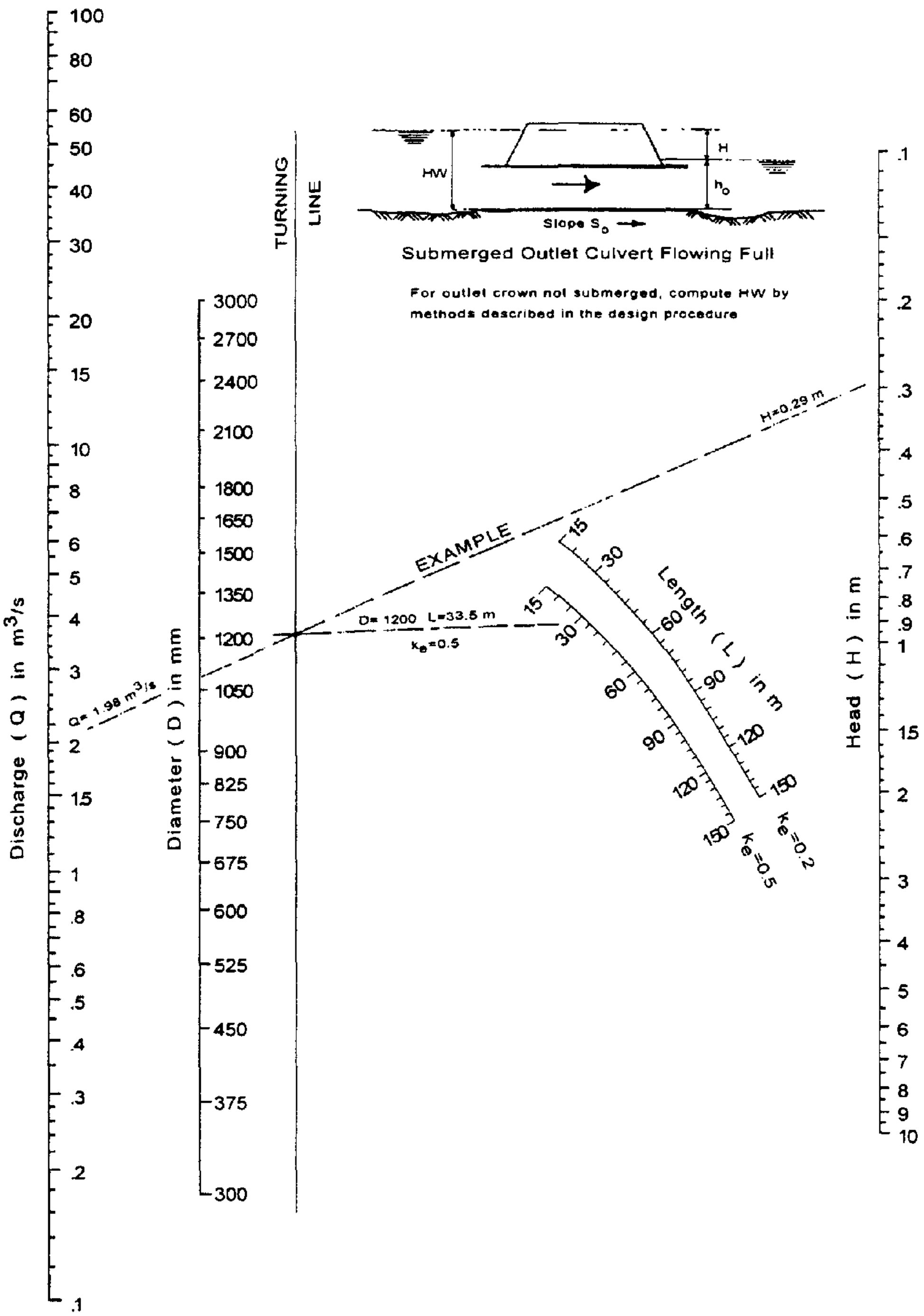
CHART 1



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

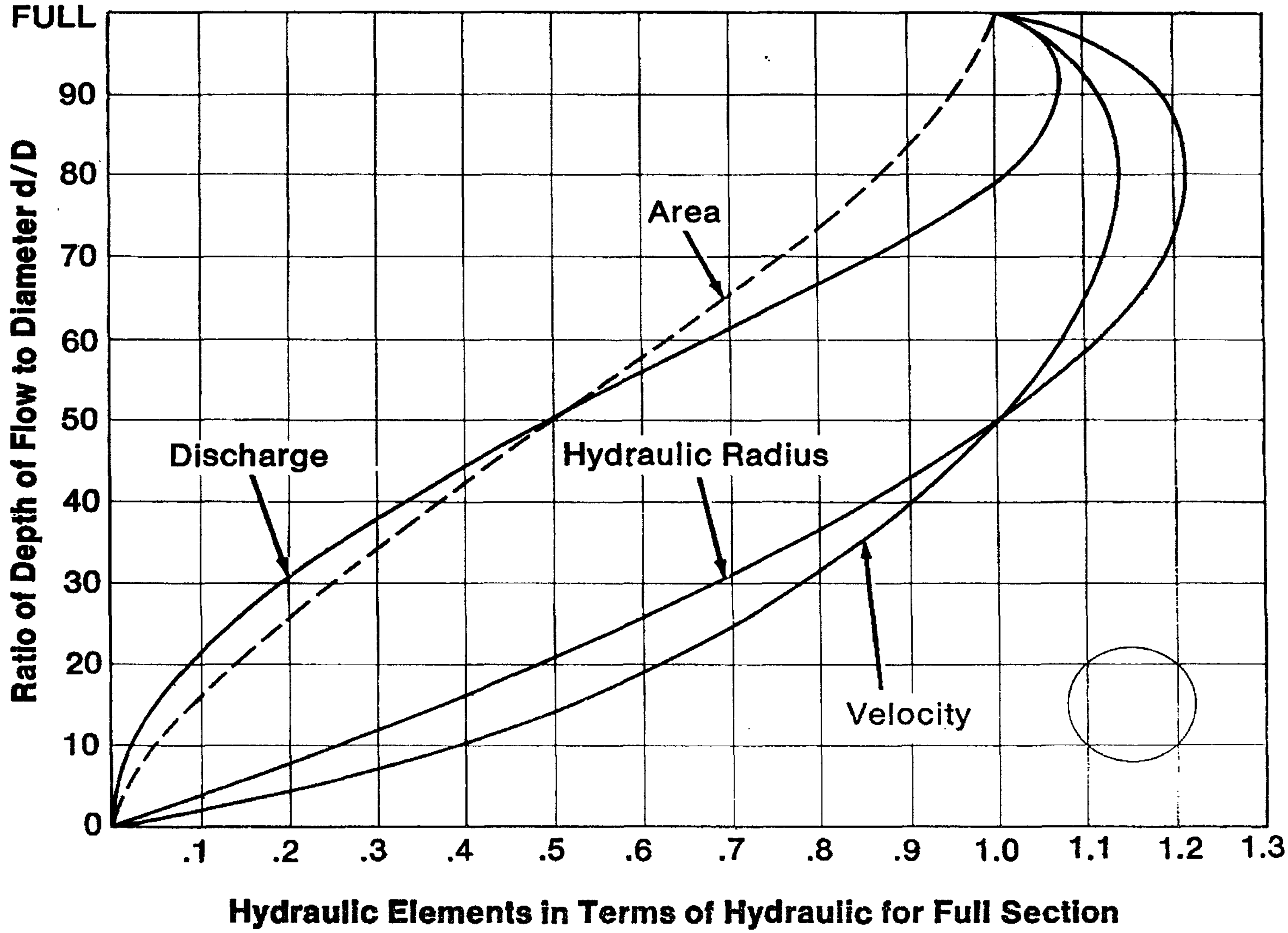
Adapted from
Bureau of Public Roads Jan. 1963

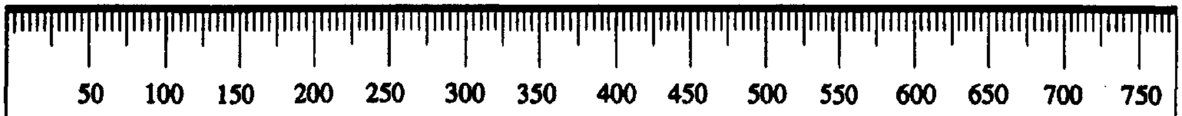
CHART 5



Adapted from
Bureau of Public Roads Jan. 1963

HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n=0.012


FULL
90
80
70
60
50
40
30
20
10
0
Ratio of Depth of Flow to Diameter d/D
Area
Discharge
Hydraulic Radius
Velocity
.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0
1.1
1.2
1.3
Hydraulic Elements in Terms of Hydraulic for Full Section



FHWA Training Course "Introduction to Highway Hydraulics"

Metric Ruler 1cm=50m

NOTES: