Bridge and Culvert Hydraulics(HENG – 5203)

Lecture-01: **Introduction**

Programme: ***BSc in Hydraulic and Water Resources Engineering***

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Course Objectives and Outcomes

Course Objectives

- To Select the appropriate site for bridges and culverts
- To Plan, design and constructional view of bridges and culverts
- To Model bridges and culverts on natural rivers using HEC-RAS

Course Outcomes

At the end of this course, the student will be able to:

- Have an idea of appropriate site selection for both bridges and culverts structures.
- Have a concepts on data collection, plan and design of bridges and culverts
- Have a knowledge of bridges and culverts model on natural rivers

Course Outlines

Chapter One

1. INTRODUCTION

1.1. Why you study culvert and bridge hydraulics?

1.2. Hydraulic causes of culvert and bridge failure

1.3. Site selection and data required

Chapter Two

2. CULVERT HYDRAULICS

2.1. Introduction

2.2. Classification of flow types at culvert

2.3. Culvert flow controls (inlet and outlet control)

2.4. Principles and criteria of culvert design

2.5. Hydraulic design of culverts

Chapter Three

3. BRIDGE HYDRAULICS

3.1. Introduction

3.2. The effect of bridge on river flow

3.3. Hydraulic performance of bridge

3.4. Types of flow in bridge waterway

3.5. Hydraulic design of bridges

Chapter One: Introduction

- ✓ Culverts and bridges are highway cross-drainage hydraulic structures those accommodate the passage of:
  - water, sediment and natural debri materials
  - both the upstream and downstream movement of aquatic organisms.
- ✓ The purpose of both *bridge and culvert* is providing a transportation route over obstructions (such as river, lake or even another roadway or railway)
- ✓ According to P. Novak et al. (2007), culverts are submerged structures buried under embankment usually shorter span (< 6 m), with the top not normally forming part of the road surface like in a bridge structure.

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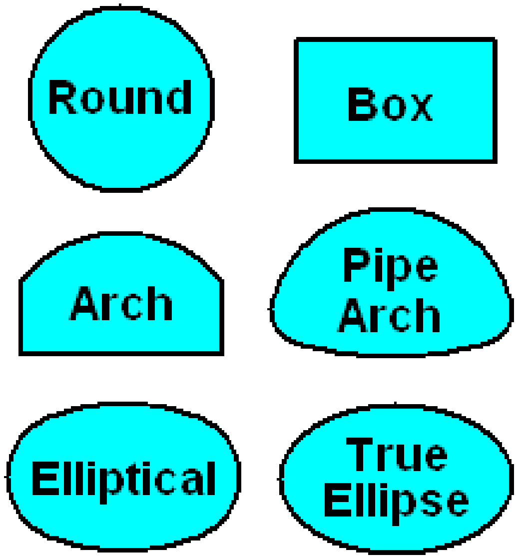
- ✓ Culverts are enclosed conduits like:
  - Complete pipe
  - Elliptical
  - Pipe arch
  - Box



- ✓ According to ERA (2011), culverts constructed at a water course (Stream Culverts) or low points of road where there is no define stream but cross drainage is required for the flow which cannot be accommodated by side drains (Relief Culverts).

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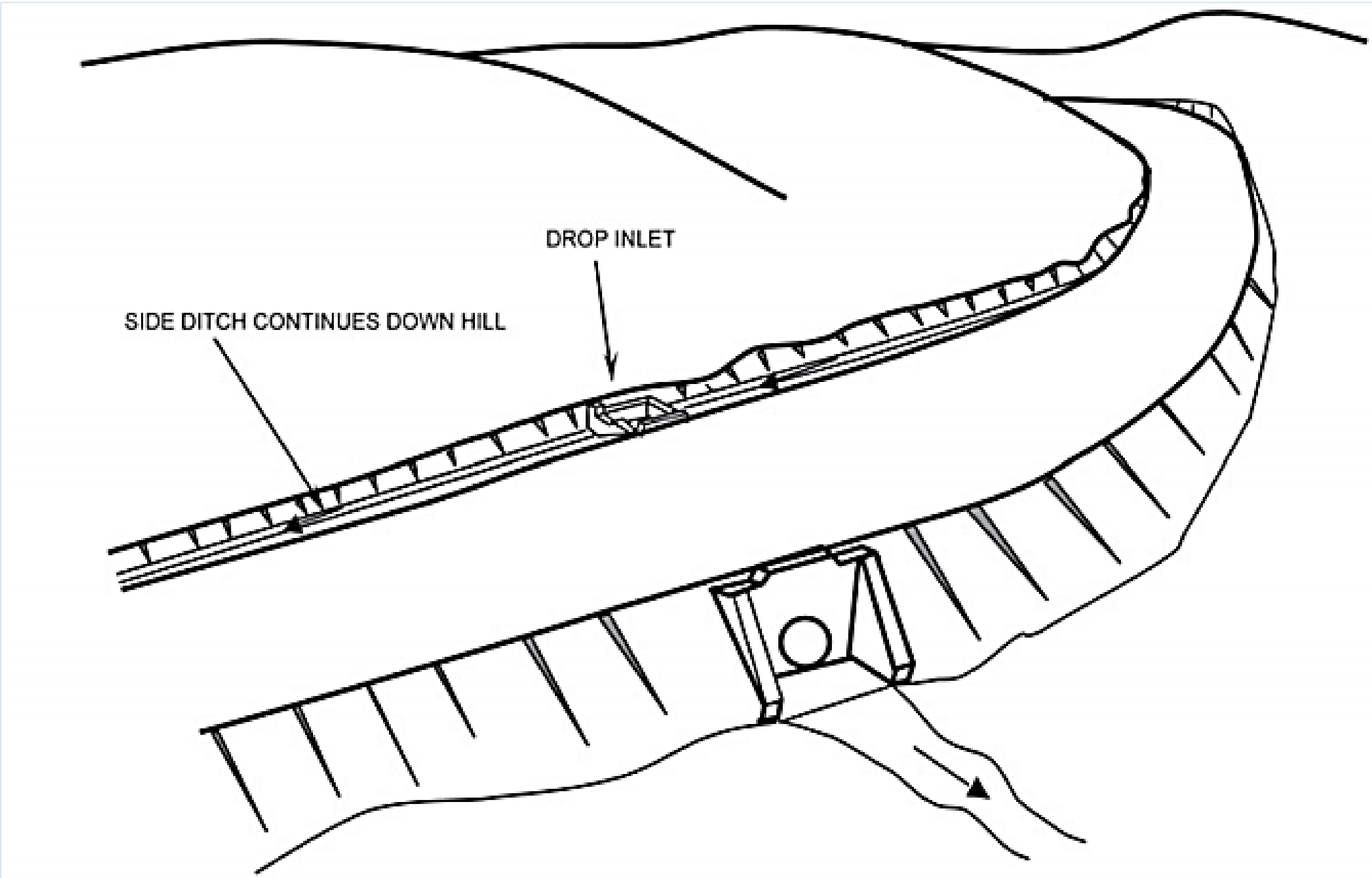

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- ✓ Bridges are highly costing structures that support a roadway over the water body by means of a span.
- ✓ Bridges does not have a constructed structural invert (bottom) and does not fully enclose the channel.
- ✓ Bridges consist of a deck supported on abutments or piers, open-bottom box or half-pipe, or other structural arch.
- ✓ Bridge construction was adopted in Ethiopia since 19^{th} century, in 1884 bridge erected over Temchi River (a tributary of Blue Nile River) and 1886 over Awash River.

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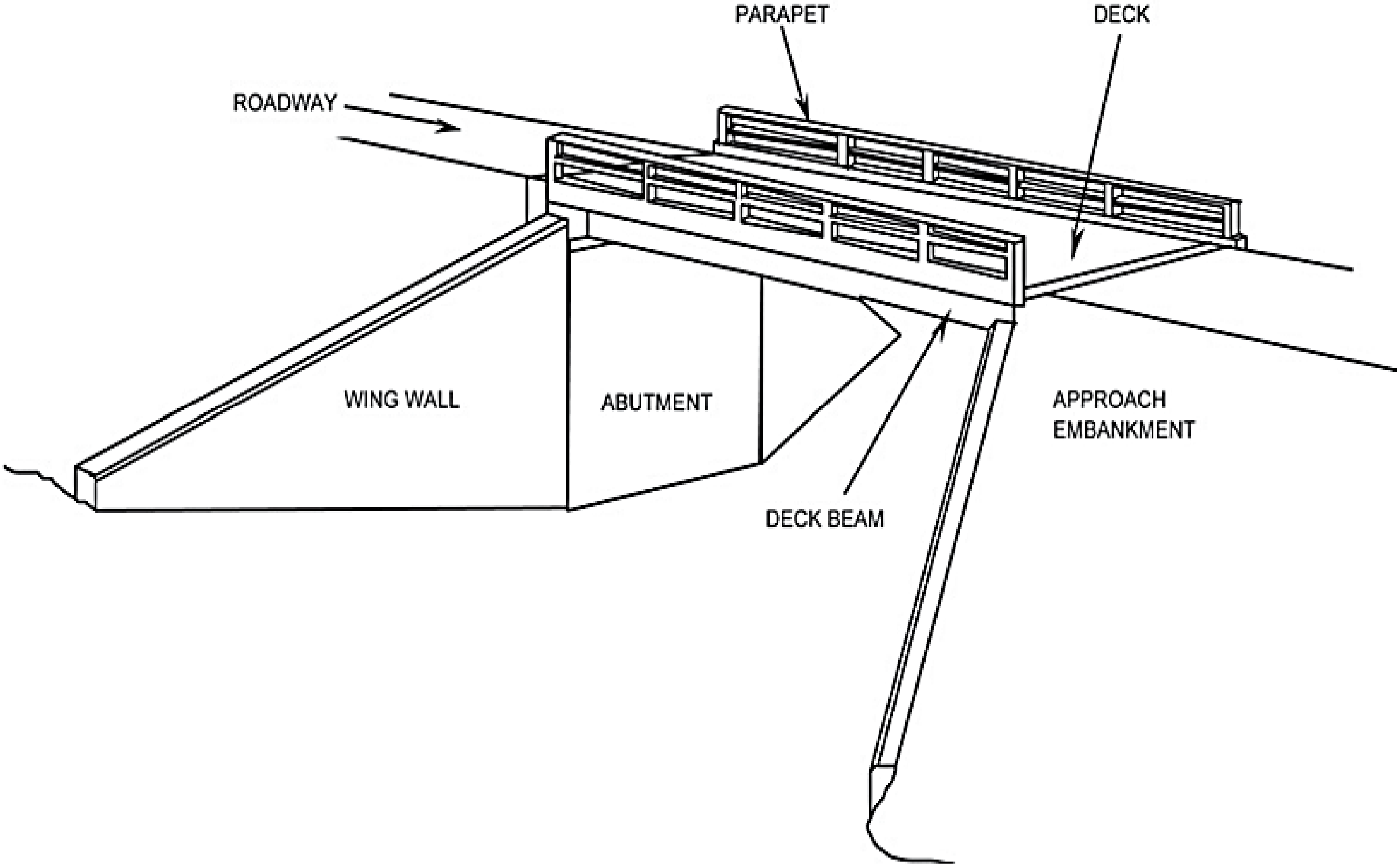
PARAPET
DECK
ROADWAY
WING WALL
ABUTMENT
APPROACH
EMBANKMENT
DECK BEAM

Cont’d…

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Comparisons to Culverts and Bridges

✓ Designer must determine:

- Which type of structure is best for a particular location
- Which one is chosen hydraulically, aesthetically and economically
- How should the structure be analyzed (as a bridge with free surface flow using GVF concepts or as a culvert with head water based)

Cont'd…

- ✓ In most of the sites, either a bridge or culvert will fulfill the structural and hydraulic requirements.
- ✓ The structural requirements should be based on:
  - Risk of property damage
  - Construction and maintenance costs
  - Traffic safety
  - Environmental considerations
  - Risk of failure

Cont'd…

- ✓ Culverts are used where:
  - *bridges are not hydraulically required*
  - *debris and ice potential are tolerable*
  - *more economical than bridge*
- ✓ Bridges are used where:
  - culverts are impractical
  - more economical than a culvert
  - to satisfy *land-use and access requirements*
  - *mitigate environmental concerns*
  - *accommodate ice and large debris of heavy sediment loads or moving boulders…etc.*

Cont'd…

- ✓ There are differences in the hydraulic assumptions and analyses used for culverts and bridges.
- ✓ Culvert hydraulic analysis assumes
  - – No velocity approaching the culvert or
  - – In the channel immediately downstream of the culvert, which overestimates entrance and exit energy losses and
  - – Tail water condition in the channel downstream of a culvert is typically based on normal depth analysis.
- ✓ Bridge hydraulic analysis is typically based on GVF calculations providing a more accurate water surface profile throughout the crossing.

1.1. Why study bridge and culvert hydraulics

- ✓ Studying the bridge & culvert hydraulics is important because of nobody can be allowed to build a new bridge/culvert in a river cross section without first being able to prove by:
  - *calculation or modeling that the resulting backwater will not cause flooding of land and upstream property*.
- ✓ In addition to the nature and geometry of the river channel, the shape, spacing, orientation of the bridge piers and abutments will affect the flow through a bridge and causes probability of scouring of the bed.

Cont'd…

- ✓ Well designed bridges and culvert are not immune to this problem, while bridges that are badly designed hydraulically are even more likely to fail and collapse
  - – How can a good design be obtained without a knowledge of bridge and culvert hydraulics?
- ✓ Therefore, hydraulic analysis is a prerequisite for the scour calculations & determine *how much of the flooding is caused by the bridge/culvert.*

1.2. Hydraulic causes of bridge and culvert failure

- ✓ According to Smith (1976, 1977), 143 bridge failures occurred throughout the world between 1847 and 1975. Almost half of the failures were due to floods.
- ✓ More specifically, two factors were identified:
  – one was ***scour*** and the second was ***debris***
- ✓ Damage caused by ***scouring*** of the bottom material around the foundation ***(most of the time around piers & abutments)*** tends to be the most dominant.

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- ✓ ***Scour*** include side erosion of the river banks in the vicinity of a bridge.
- ✓ This may result in skewed angle flow approaching the bridge, failure of the piers, abutments and highway embankments.
- ✓ *Location (site selection):-*Usually the alignment of a highway will be selected to minimize the cost of the bridges/culvert and the impact to the stream as much as possible.
- ✓ If the waterway openings is too narrow w/c causes scouring of river bank and lost any economic advantage because expensive river training works will be needed to compensate

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1.3. Site selection and Data Required

#Q 1. If you are assigned to select site for culvert and bridge across a roadway, what will be the ideal site for these structures?

#Q 2. List some factors which govern the maximum length of side drains those will be relieved by a turn out or cross structure?

Site Selection

- ✓ Culvert and bridge site selection is important in terms of:
  - Cost of construction
  - Maintenance
  - Service life
- ✓ **The characteristics of an ideal site for a culvert and bridge across a river are:**
  - straight reach of the river
  - Steady river flow without cross tides
  - A narrow channel with stable rock banks
  - Suitable high banks above high flood level on each side
  - Absence of sharp curves in the approaches
  - Absence of expensive river training works
  - Avoidance of excessive underwater construction

Cont'd…

- ✓ Side drains will be released by cross structure after a maximum length of 200m to:
  - – avoid exceeding capacity or
  - – causing erosion in the drain or in the outfall watercourse.
- ✓ Ideal *outfall sites* are at field boundaries where there is *vegetation or stable ground to minimize the risk of downstream erosion*.

Cont'd…

- ✓ Adjustment of the road alignment is often justified to minimize the cost of structures and risk of damage or erosion.
- ✓ *Careful site selection* is essential to ensure ease of construction and to minimize the whole life cost of the structure.
- ✓ *Poor site selection* can result in a longer, wider or higher structure than is actually necessary and can also lead to excessively high maintenance costs.

#Q 3. Explain why the crossing structure should be located away from horizontal curves in the watercourse?

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- ✓ According to ERA (2011), the following criteria should ideally be met when determining a site for water crossing:
  - The crossing should be *located away from horizontal curves in the watercourse*,
  - If there is no option is available a new channel should be made in some cases.
  - The crossing should be at an *area of uniform watercourse gradient*.
  - If the gradient is steepening there will be a possibility of scour, and for *reducing gradient there will be a risk of deposition*.

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- The crossing should ideally be at an area of the channel with a non-erodible bed. *These areas need low amount of watercourse protection*.
- The road should cross the watercourse at a point with *well-defined banks, where the stream will generally be narrower*.
- The watercourse should not be prone to flooding at the crossing point
- The skew angle shall be < 15^o.

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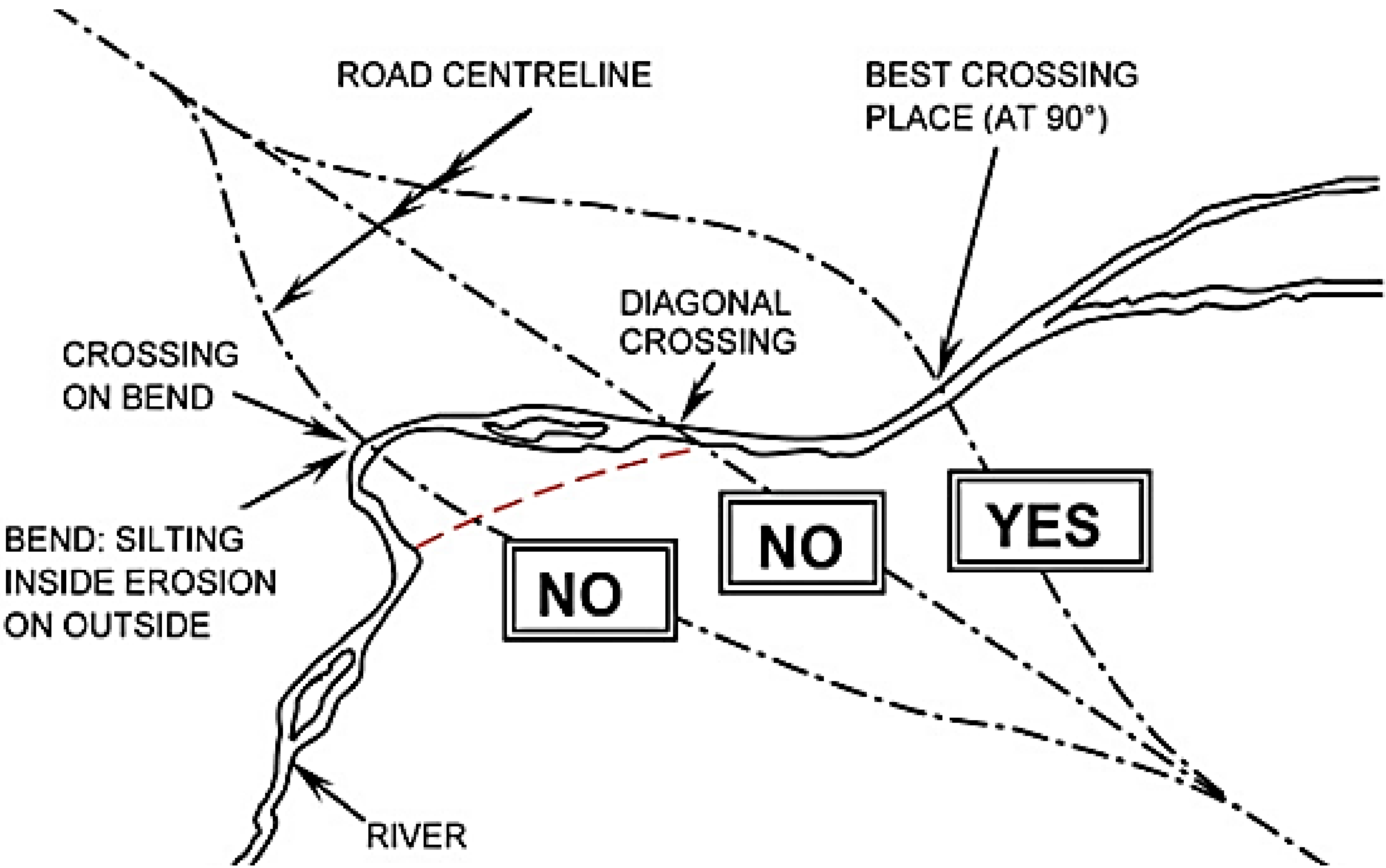

ROAD CENTRELINE
BEST CROSSING PLACE (AT 90°)
DIAGONAL CROSSING
CROSSING ON BEND
BEND: SILTING INSIDE EROSION ON OUTSIDE
NO
NO
YES
RIVER

Cont'd…

#Q 4. List data required to design culvert and bridge

#Q 5. Define design flood, base flood, overtopping flood, backwater and tail water.

Data Required

- ✓ *Topographic maps*:- to identify channel width, possible crossing sites, to obtain the channel gradient and to indicate floodplain.
- ✓ *Geological maps, Soil maps and Geological records*:- provide some details about the local geology.
- ✓ *Hydrological data*:-such as gauging station records, stage-discharge relationship and flow-duration curve.
- ✓ *Meteorological data*:- rainfall depths and intensities, temperature range and wind speed. These data may help in assessing the possibility of flash flood and ice loading on the super structure.

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- ✓ *River channel data*:- Roughness of the channel and floodplains.
- ✓ It may be difficult to determine the *width and depth of the river* channel from maps. This requires evaluation of the following:-
  - Design flood, base flood, overtopping flood, backwater and tail water.
  - ***Design flood*** is defined as the flood associated with the probability of exceedance (frequency) selected for the design of a highway breach.

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- Once the design frequency is determined, a discharge for the selected frequency can be determined which is also known as the "*design discharge*".
- *The base flood* is the flood having a 1-percent chance of being exceeded in any given year.
- *The overtopping flood* is described by the probability of exceedance and water surface elevation at which flow begins over the culvert/bridge.
- *Backwater* is the increase of water surface elevation induced upstream from a bridge, culvert, dam …etc.
- Tail water for sizing of cross drains and the determination of headwater and backwater elevations.

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- ✓ In addition to the data requirements explained above, the site and detailed investigation criteria should emphasize on the:
  - *Road Alignment*:- cross the watercourse at 90 degrees as this minimizes the span length of the bridge or pipe and to avoid additional scouring.
  - *Location*:- A site with a natural narrow channel width, at a straight stretch of watercourse, approach roads should preferably be straight on each side to ensure sufficient sight distances and prevent traffic hazards.
  - The location also should be away from waterfalls and confluence zones.

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- ✓ *Existing Structure Assessment*:- Where existing roads are being improved, existing drainage sites should already have been provided with an appropriate structure.
- ✓ *Site Investigation*:- Is to provide a clear picture of the ground conditions, to enable a suitable design to be carried out.
- ✓ The level of site investigation clearly depends on the type and complexity of the proposed structure.
- ✓ A site investigation will involve taking samples of the ground material to determine its bearing capacity.

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- ✓ These samples can either be obtained through digging trial pits or by using a hand auger.
- ✓ *Bearing capacity*:- The ground underneath a proposed structure should have an adequate bearing capacity to support the load of the structure itself and the vehicles which pass over it.
- ✓ *The bearing capacity will depend* on a range of different factors including: *the proportions of sand clay, organic and the mineralogy of the clay materials; and the level of the water table.*

Thank you!!!

Chapter Two: Culvert hydraulics

2.1. Introduction

- ✓ Culvert designed hydraulically to increase hydraulic capacity and used to convey surface runoff through embankments.
- ✓ Common culvert diameters are 600mm and 900mm.
- ✓ Cross culverts smaller than 600mm in diameter should not be installed as they are very difficult to clean or maintenance (ERA, 2011).
- ✓ During a storm event a culvert may operate under inlet control, outlet control or both.

Cont'd…

- ✓ Numerous cross-sectional shapes are available and the most commonly used shapes like circular, box (rectangular), elliptical, pipe-arch and arch.
- ✓ The shape selection is based on the cost of construction, the limitation on upstream water surface elevation, roadway embankment height and hydraulic performance.

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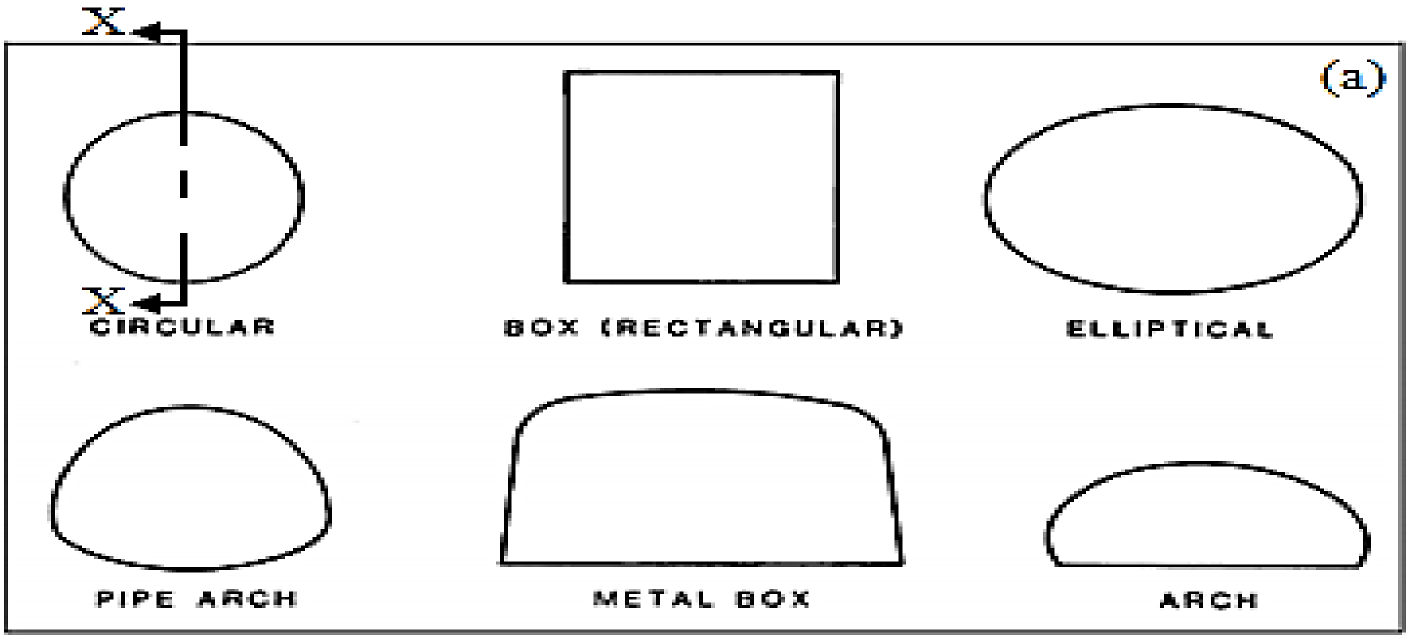
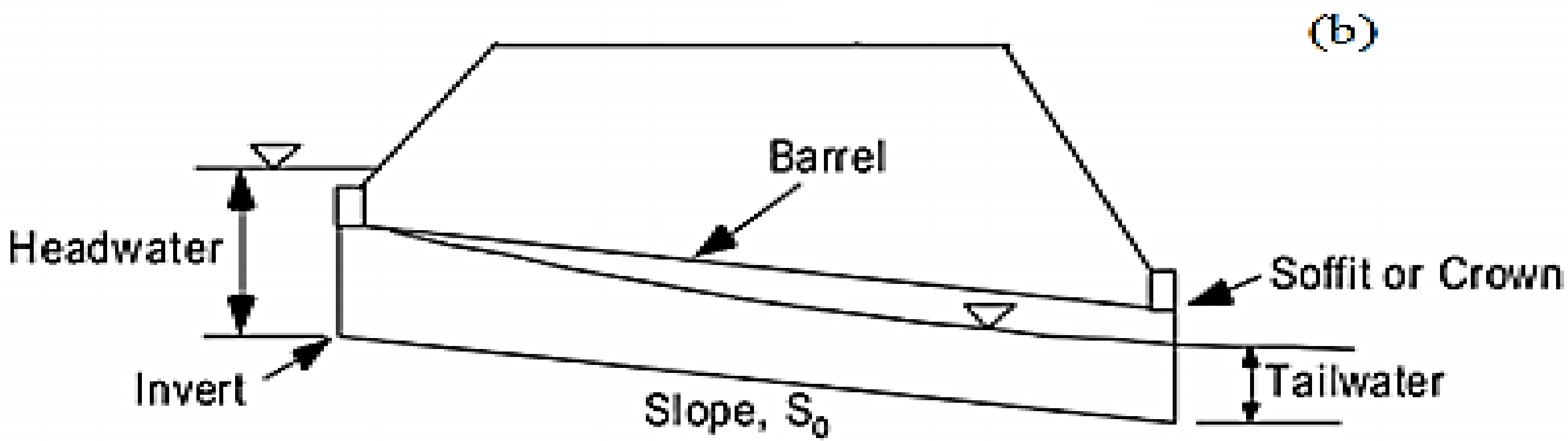

X
X
(a)
CIRCULAR
BOX (RECTANGULAR)
ELLIPTICAL
PIPE ARCH
METAL BOX
ARCH


(b)
Barrel
Headwater
Soffit or Crown
Invert
Tailwater
Slope, S_0

Cont'd…

- ✓ Based on construction materials used culverts can be concrete (reinforced and non-reinforced), steel (smooth and corrugated), corrugated aluminum, vitrified clay, plastic, cast iron, wood and stainless steel.
- ✓ The *discharge of a culvert* is determined by the application of the *continuity* and *energy equation* between the *approached section and the downstream section* which is within the culvert barrel.

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- ✓ Basic concepts that are important in culvert design include:
  - *Critical flow*: - is state of flow where the specific energy is a minimum for a given discharge. Also the velocity head is equal to one-half the hydraulic depth (Fr = 1).
  - *Critical depth*: - is the depth at the critical flow. For a given discharge and cross section geometry, there is only one critical depth.
  - *Critical slope*: - a slope that sustains a given discharge at a uniform and critical depth.
  - *Crown (soffit)*:- is the inside top of the culvert.

Cont'd…

- ✓ *Free outlet*: - has tail water equal to or lower than critical depth.
  - For culverts with free outlets, a *lowering of the tail water has no effect on the discharge or the backwater profile upstream* of the tail water.
- ✓ *Energy grade line*: - represents the total energy at any point along the culvert barrel.
  - The total energy at any section is the sum of flow depth, velocity head ($V^2/2g$), and all energy losses.
  - Improved inlet has an entrance geometry that decreases the flow contraction at the inlet and thus increases the capacity of culverts.
- ✓ *Invert* is the flow line of the culvert (inside bottom).

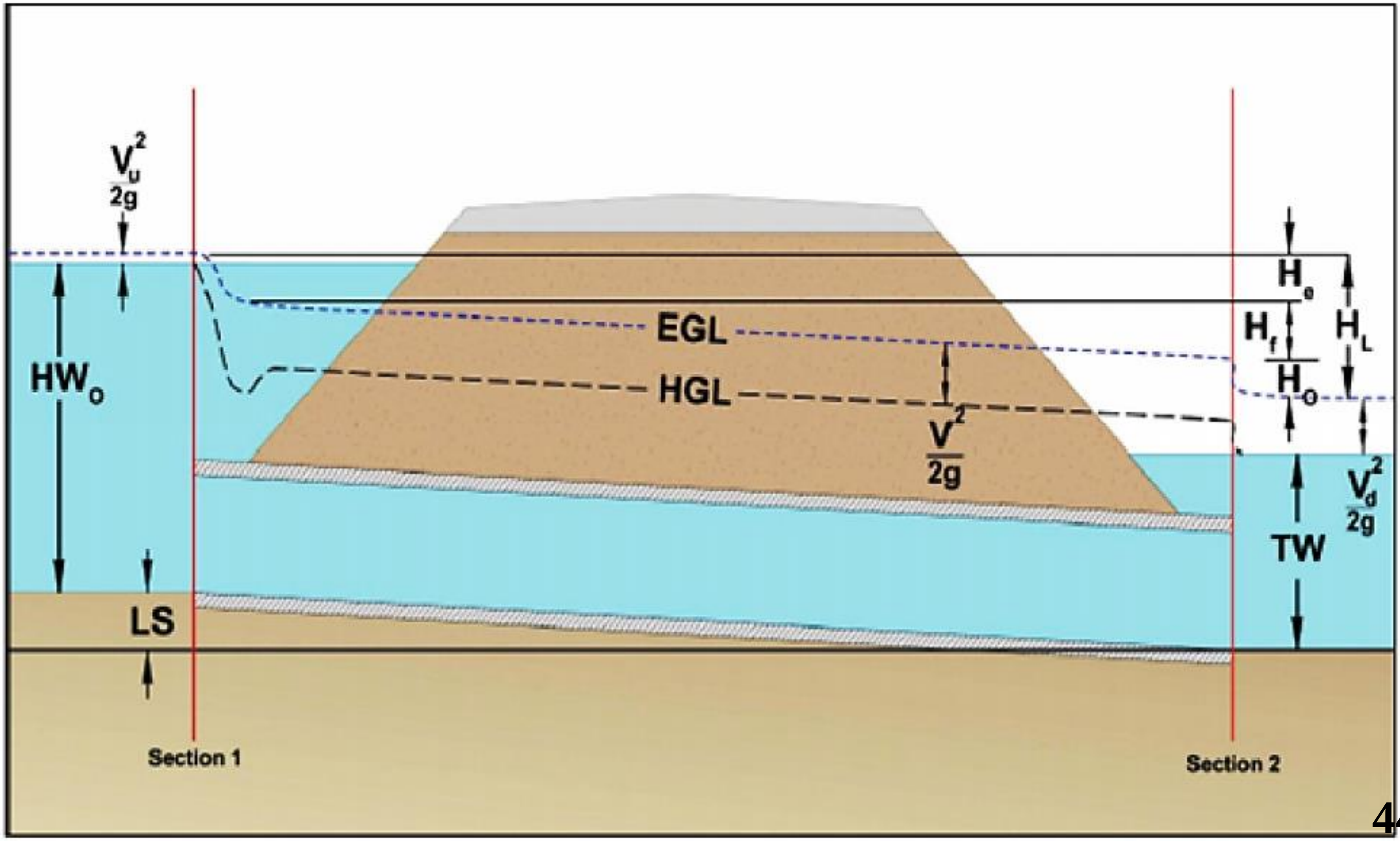
Cont'd…

- ✓ *Normal flow* occurs in a channel reach when the discharge, velocity, and depth of flow do not change throughout the reach.
  - The water surface profiles and channel bottom slope will be parallel.
  - This type of flow will exist in a culvert operating on a steep slope if the culvert is sufficiently long enough.
- ✓ *Normal depth*: - is the depth of water at a steady, uniform, constant velocity and flow at a given channel reach.

Cont'd…

- ✓ *Steep slope*:- steep water surface slope occurs where the critical depth is greater than the normal depth (supercritical flow).
- ✓ *Mild slope* occurs where critical depth is less than normal depth (subcritical flow).
- ✓ *Submerged outlet* occurs where the tail water elevation is higher than the crown of the culvert.
- ✓ *Submerged inlet* occurs where the headwater is greater than 1.2D.

- ✓ Freeboard is a safety margin over design water level before overflow occurs.



2.2. Classification of flow types at culvert

- ✓ A culvert barrel may flow full or partly full over all of its length.
- ✓ The hydraulic condition in a culvert flowing full is called *pressure flow*
  - – One condition which can create pressure flow in a culvert is the back pressure caused by a high downstream water surface elevation.
- ✓ *Partly full* (free surface) flow or open channel flow may be categorized as subcritical, critical, or supercritical.
- ✓ A determination of the appropriate flow regime is accomplished by evaluating the dimensionless number called the Froude number.

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- When $Fr > 1.0$, the flow is supercritical and is characterized as rapid
- When $Fr < 1.0$, the flow is subcritical and is characterized as tranquil (gradual)
- If $Fr = 1.0$, the flow is defined as critical
- $F_r = \frac{V}{\sqrt{gy}}$
- For steep culvert flowing partly full, critical depth occurs at the culvert inlet, subcritical flow exists in the upstream channel, and supercritical flow exists in the culvert barrel.

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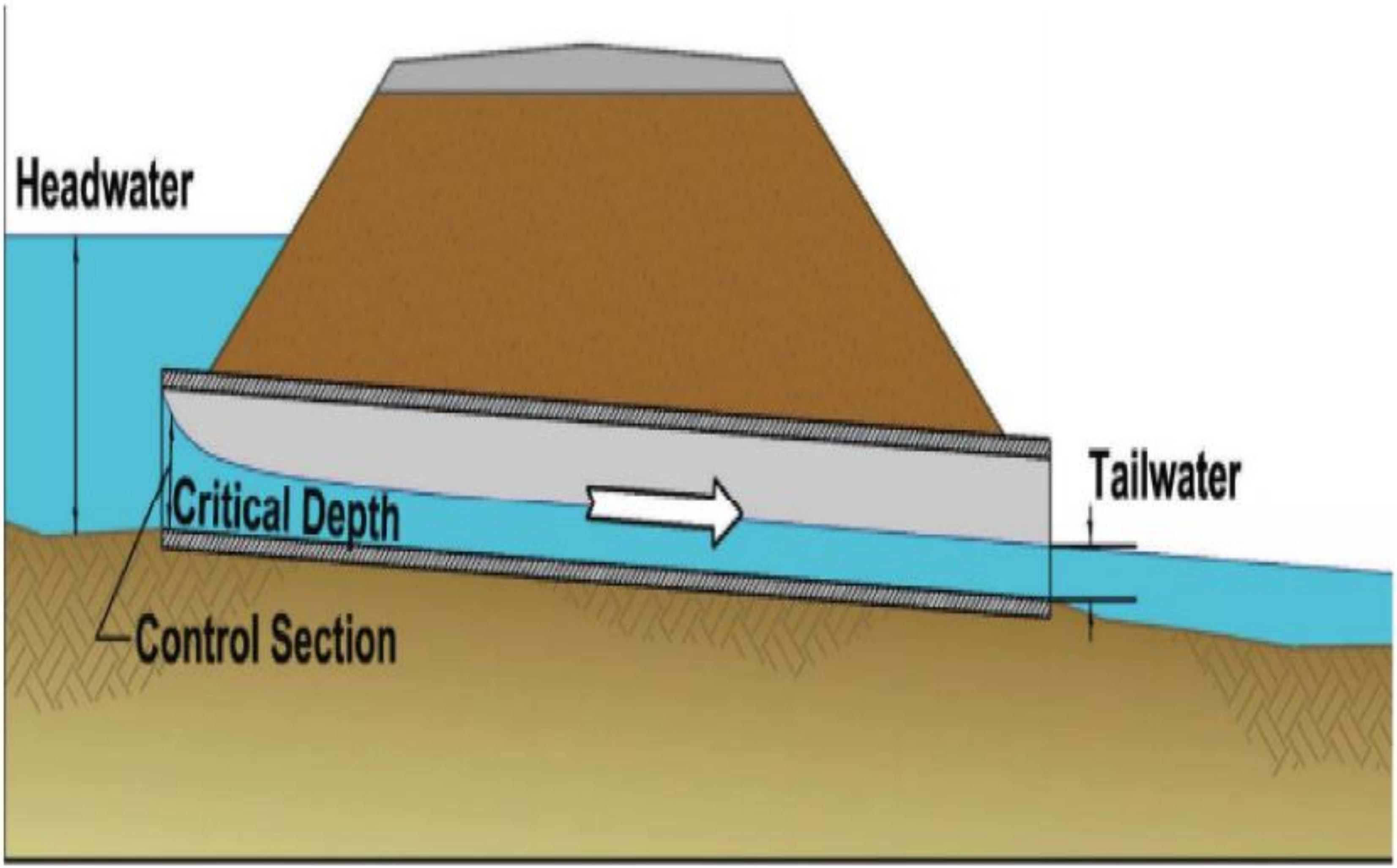
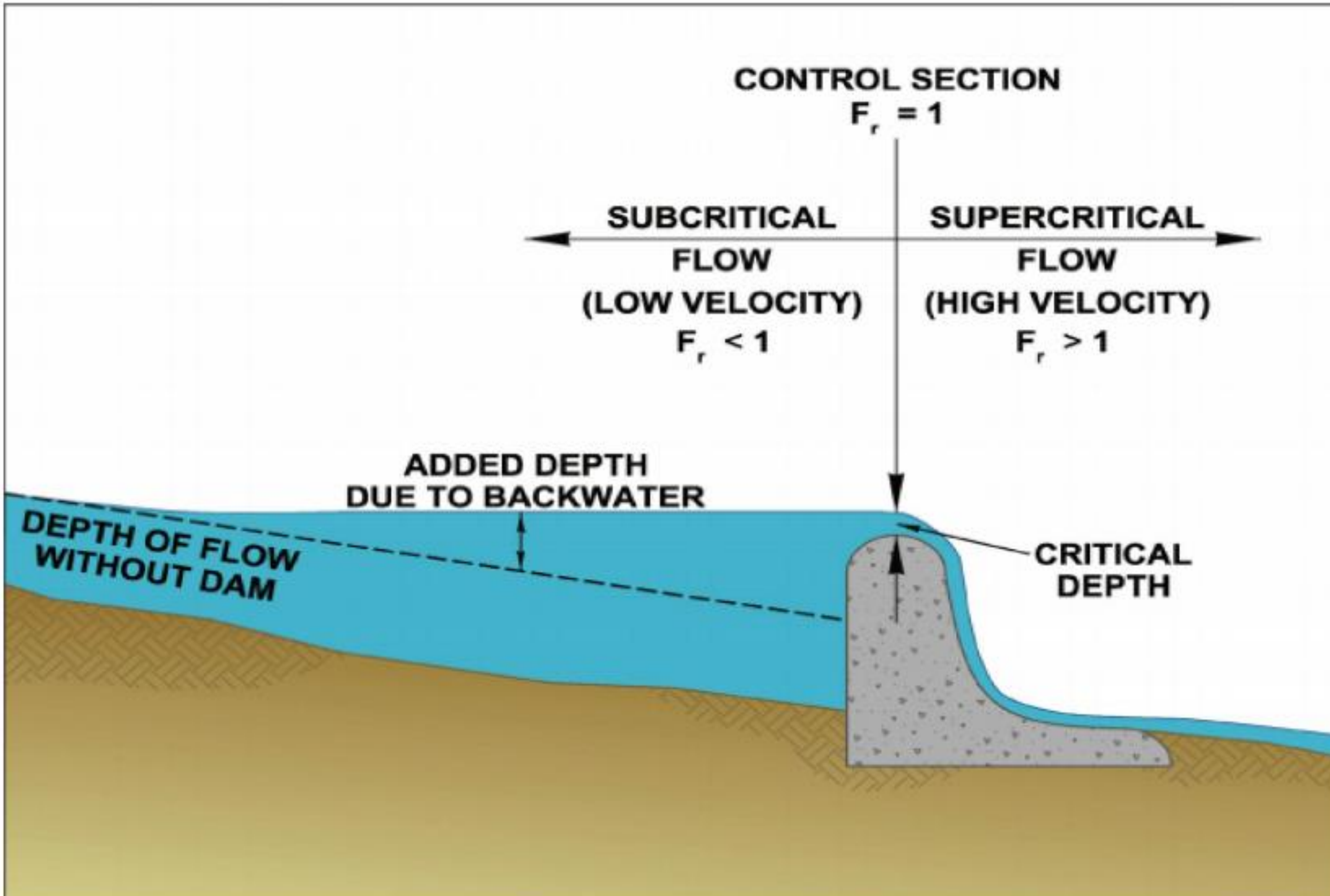


Typical inlet control flow section.



Flow conditions over a small dam.

2.3. Culvert flow controls (inlet and outlet control)

✓ **Types of flow control**

- Control section is the location where there is a unique relationship between the flow rate and upstream water surface elevation.
- The characterization of pressure, subcritical and supercritical flow regimes played an important role in determining the location of the control section.

Inlet control

- ✓ Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept.
- ✓ The inlet control section is located just inside the entrance.
- ✓ Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical.
- ✓ Downstream of the inlet control section do not affect the culvert capacity.
- ✓ The upstream water surface elevation and the inlet geometry are the major flow controls.

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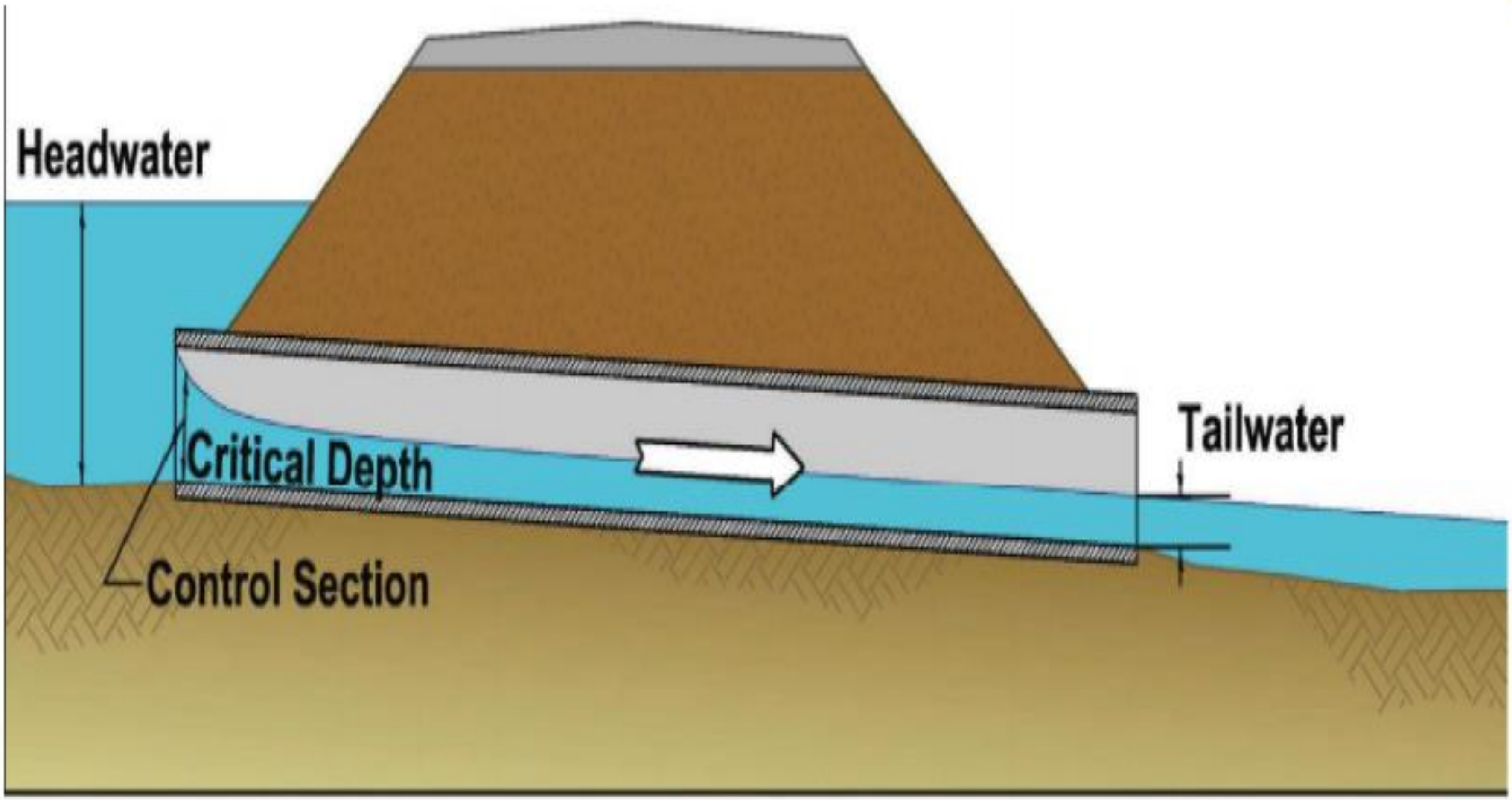

. Typical inlet control flow section.

Cont'd…

Outlet control

- ✓ Occurs when the culvert barrel is *not capable of conveying* as much flow as the inlet opening will accept.
- ✓ The control section for outlet control flow in a culvert is located at the barrel exit or further downstream.
- ✓ Either subcritical or pressure flow exists in the culvert barrel under these conditions.
- ✓ All of the geometric and hydraulic characteristics of the culvert play a great role in determining its capacity.

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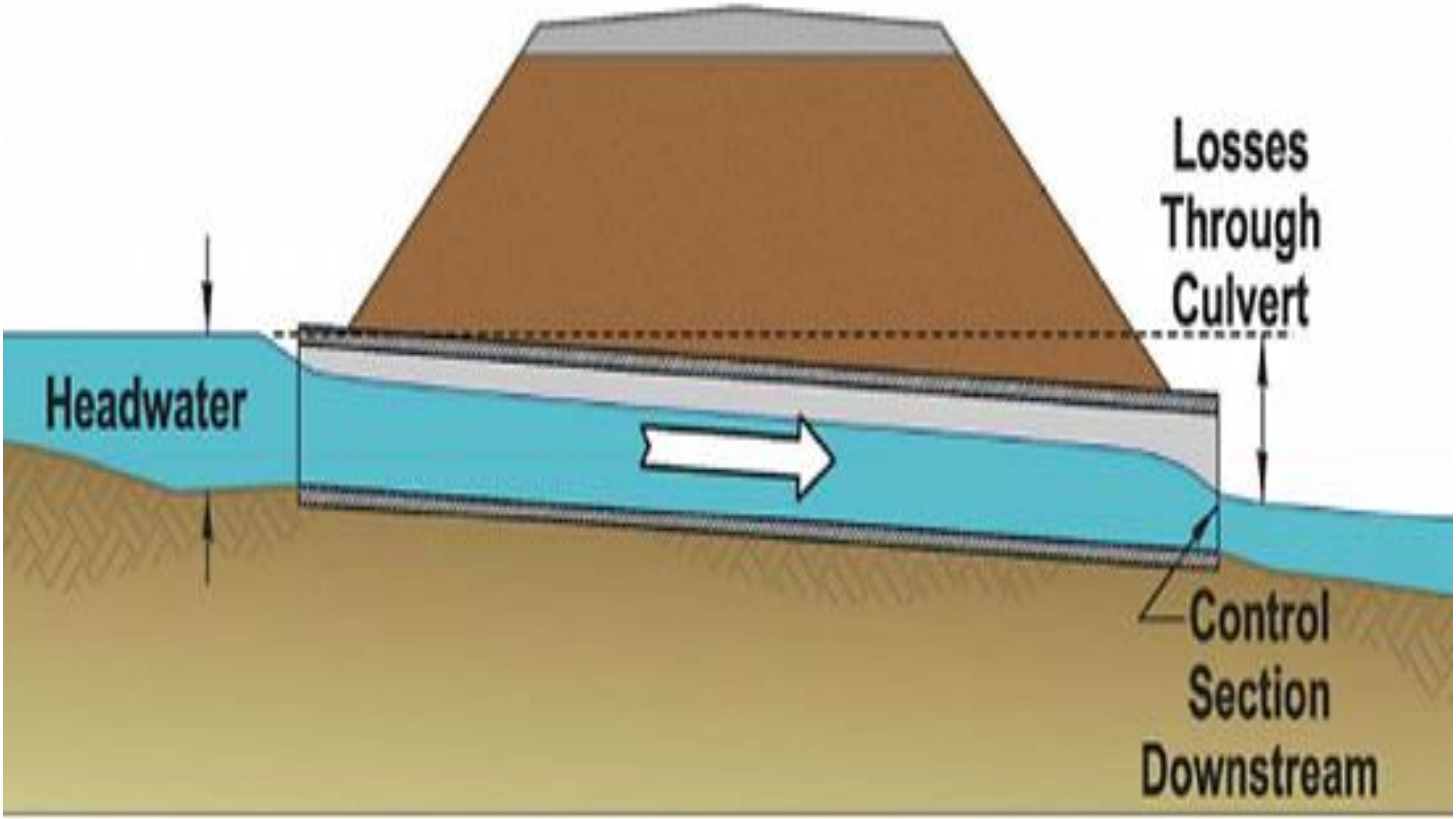


Typical outlet control flow conditions.

Factors influencing culvert performance

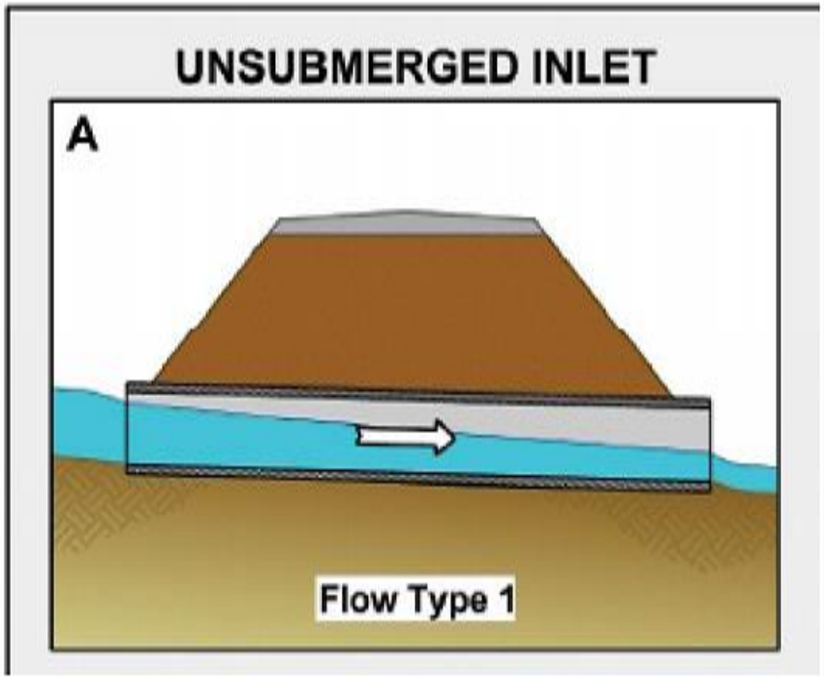
Factor	Inlet control	Outlet control
Headwater elevation	*X*	*X*
Inlet area	*X*	*X*
Inlet edge configuration	*X*	*X*
Inlet shape	*X*	*X*
Barrel roughness		*X*
Barrel area		*X*
Barrel shape		*X*
Barrel length		*X*
Barrel slope	**(to small extent or can be neglected)*	*X*
Tail-water elevation		*X*

Flow types

Flow type I: characteristics of the flow type-I are:

- Un-submerged inlet & free flow outlet
- Critical depth at inlet
- Partially full flow, steep slope
- Supercritical flow at barrel
- Inlet control.

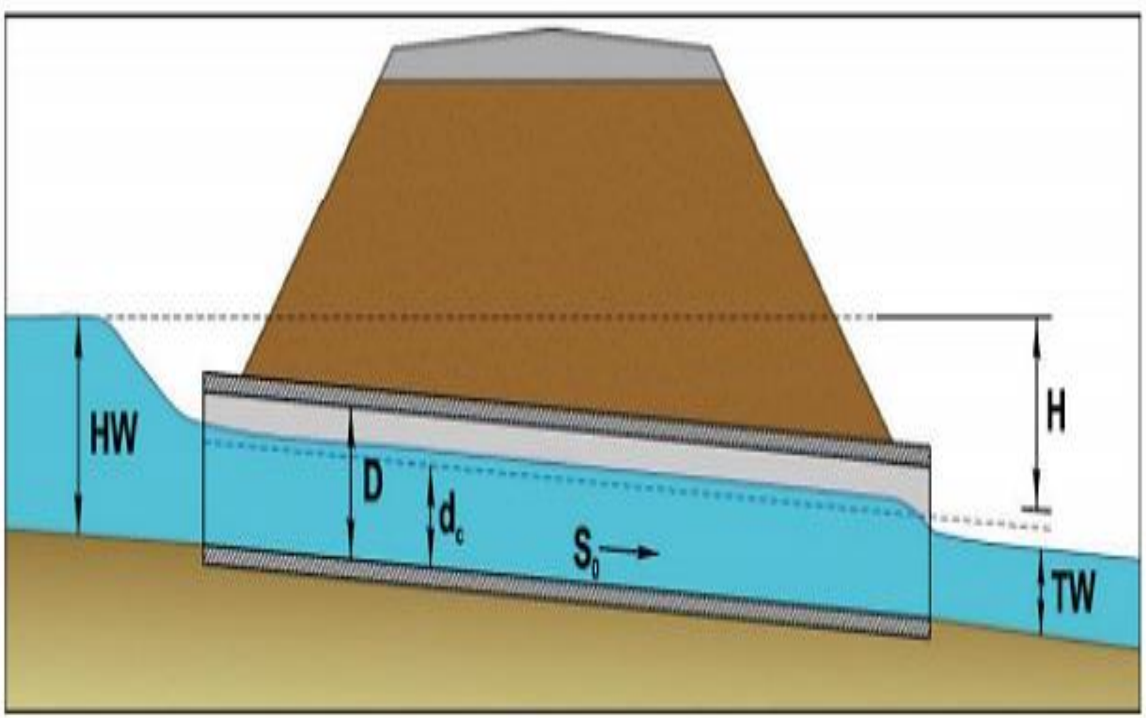
$$\frac{Y_1}{D} < 1.2 \qquad and \qquad \frac{Y_4}{Y_c} < 1$$



Flow type II: characteristics of the flow type-II are:

- Un-submerged inlet with low tail water
- Critical depth at outlet
- Partially full flow
- Mild slope
- Subcritical flow

  at barrel
- Outlet control



$$\frac{Y_1}{D} < 1.2\ and\ \frac{Y_4}{Y_c} < 1$$

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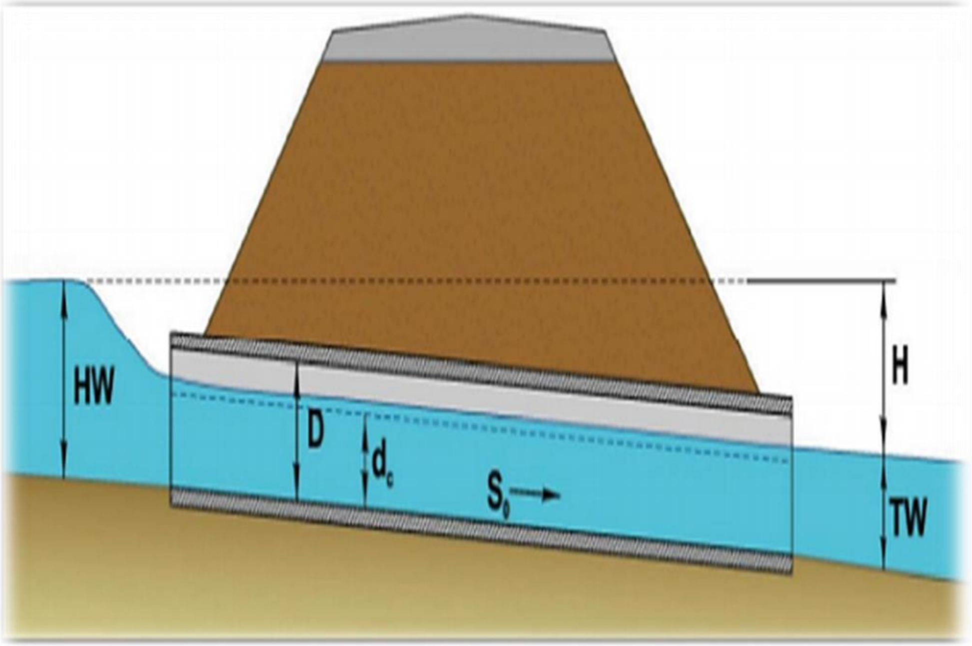
Flow type III: characteristics of the flow type-III are:

- Un-submerged inlet with tail water backing up in to outlet
- Partially full flow
- Mild slope
- Subcritical flow at barrel
- Outlet control

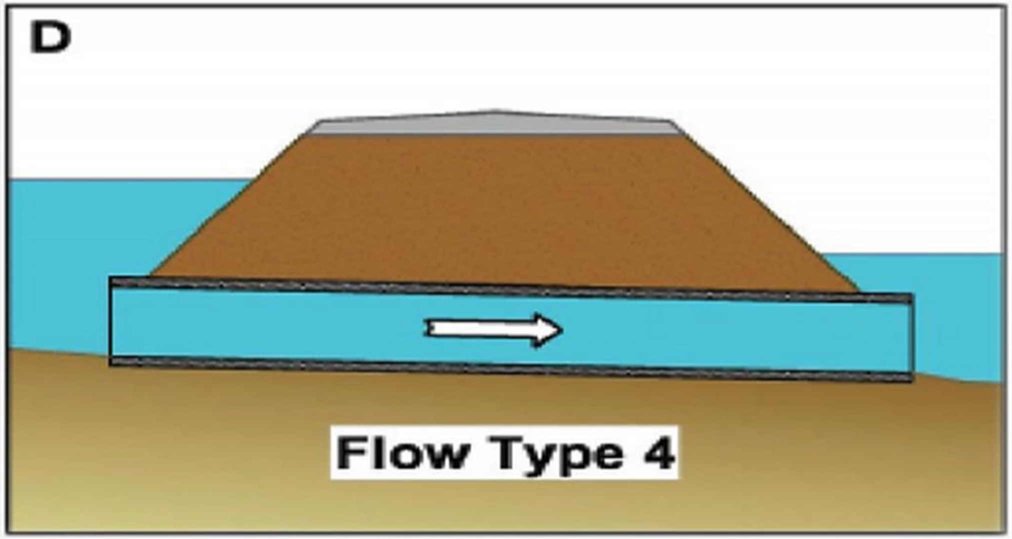


$$\frac{Y_1}{D} < 1.2 \ \ and \ \ \frac{Y_4}{Y_c} > 1$$

- Existence of GVF profile is the controlling factor, critical depth cannot occur, and the upstream water surface elevation is the function of the tail water elevation.

Flow type IV: characteristics of the flow type-IV are:

- Both inlet and outlet fully submerged
- Full flow/pipe flow
- Any slope
- Outlet control



$$\frac{Y_1}{D} > 1 \; and \; \frac{Y_4}{D} > 1$$

- The flow rate can be estimated directly from the energy equation. The tail water depth is assumed to be critical depth near the culvert outlet or in the downstream channel.

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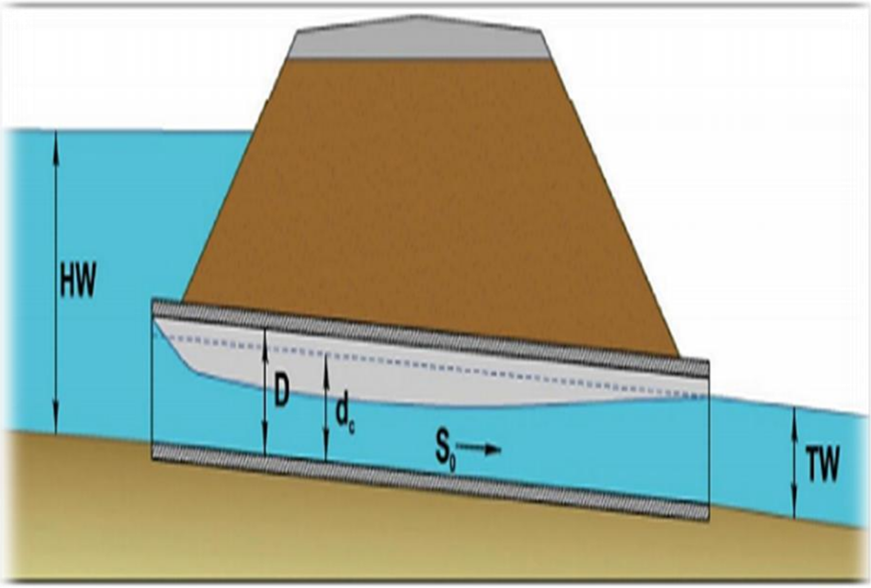
Flow type V: characteristics of the flow type-V are:

- Submerged inlet with low tail water
- Partially full flow
- Any slope
- Short length of culvert
- Orifice flow control at inlet.



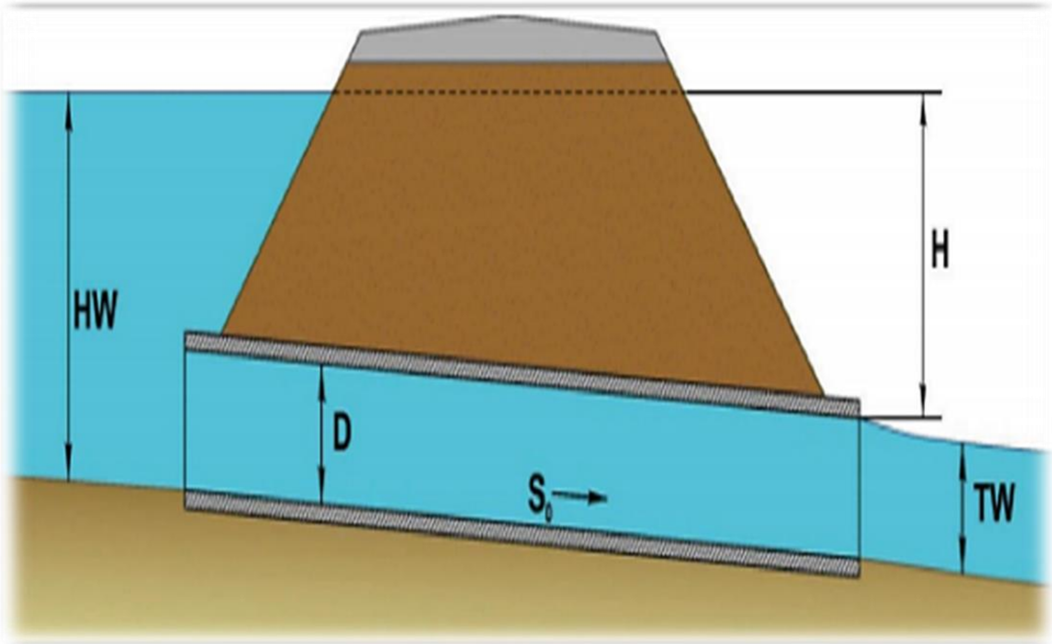
$$\frac{Y_1}{D} > 1.2 \quad and \quad \frac{Y_4}{Y_c} \leq 1$$

- An orifice is an opening, submerged on the upstream side and flowing freely on the downstream side.

Flow type VI: characteristics of the flow type-VI are:

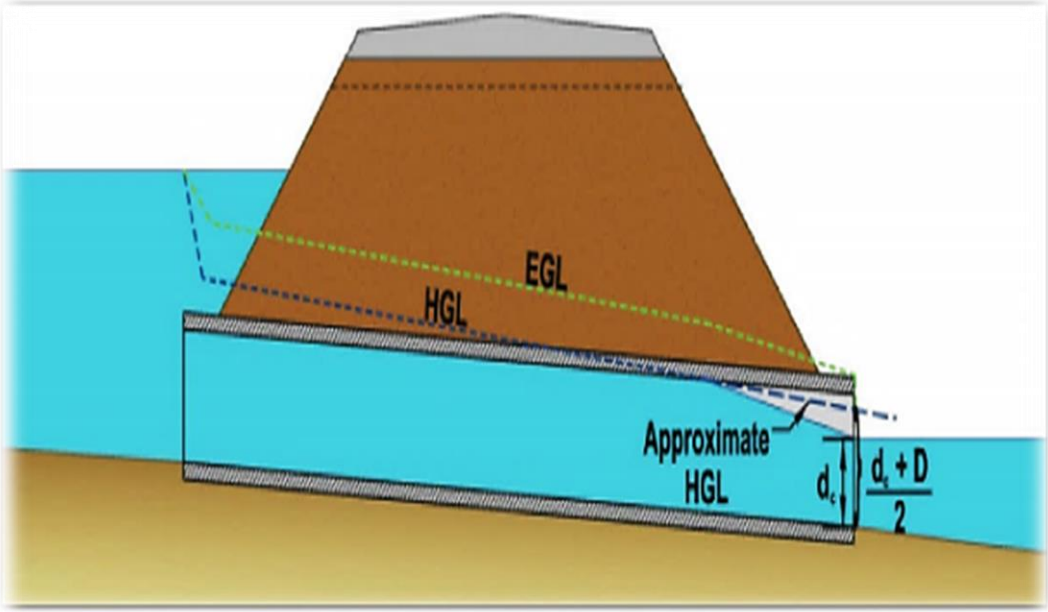
- Submerged inlet with low tail water
- Culvert flows full /pipe flow
- Outlet not submerged
- Any slope
- Long length of culvert
- Outlet control

$$\frac{Y_1}{D} > 1.2 \;\; and \;\; \frac{Y_4}{D} \leq 1$$



Flow type VII: characteristics of the flow type-VII are

- Submerged inlet with low tail water
- Partially full flow
- Outlet not submerged
- Steep slope
- Outlet control



$$\frac{Y_1}{D} > 1.2 \quad and \quad \frac{Y_4}{D} \leq 1$$

2.4. Principles and criteria of culvert design

Culvert design principles

- ✓ All culverts shall be hydraulically designed and survey information shall include:
  - – Topographic features
  - – Channel characteristics
  - – High water information
  - – Existing structures
  - – Other related site specific information.

Cont'd…

- ✓ Culvert location in both plan and profile shall be:
  - Investigated to avoid sediment build-up
  - Designed to accommodate debris
  - Minimum hazard to traffic and people
- ✓ Material selection shall include:
  - consideration of materials availability
  - service life (*abrasion* and *corrosion potentials*)

Design criteria of culvert

- ***Site Criteria***
  - Structure type selection, length and slope should be chosen to approximate existing topography
  - The culvert invert aligned with the channel bottom and the skew angle of the stream
  - The culvert entrance shall match with geometry of the roadway

Cont'd…

- ✓ ***Debris Control*** shall be needed for culverts:
  - Located in mountainous or steep regions
  - Those are under high fills
  - Where clean out access is difficult
- ✓ ***Design Limitations*** for allowable headwater:
  - Will not damage up stream property
  - Not higher than 300 mm below the edge of the shoulder
  - Equal to HW/D not greater than 1.5 (2 for $D < 1m$)
  - No higher than the low point in the road grade
  - Equal to the elevation where flow can be diverted around the culvert.

Cont'd…

- ✓ Tail water Relationship
  - – Evaluate the hydraulic conditions of the downstream channel to determine a tail water depth for a range of discharges.
- ✓ Calculate backwater curves at sensitive locations or use a single cross section analysis
- ✓ Use the critical depth and equivalent hydraulic grade line if the culvert outlet is operating with a free outfall
- ✓ Use the headwater elevation of any nearby if it is greater than the channel depth.

Cont'd…

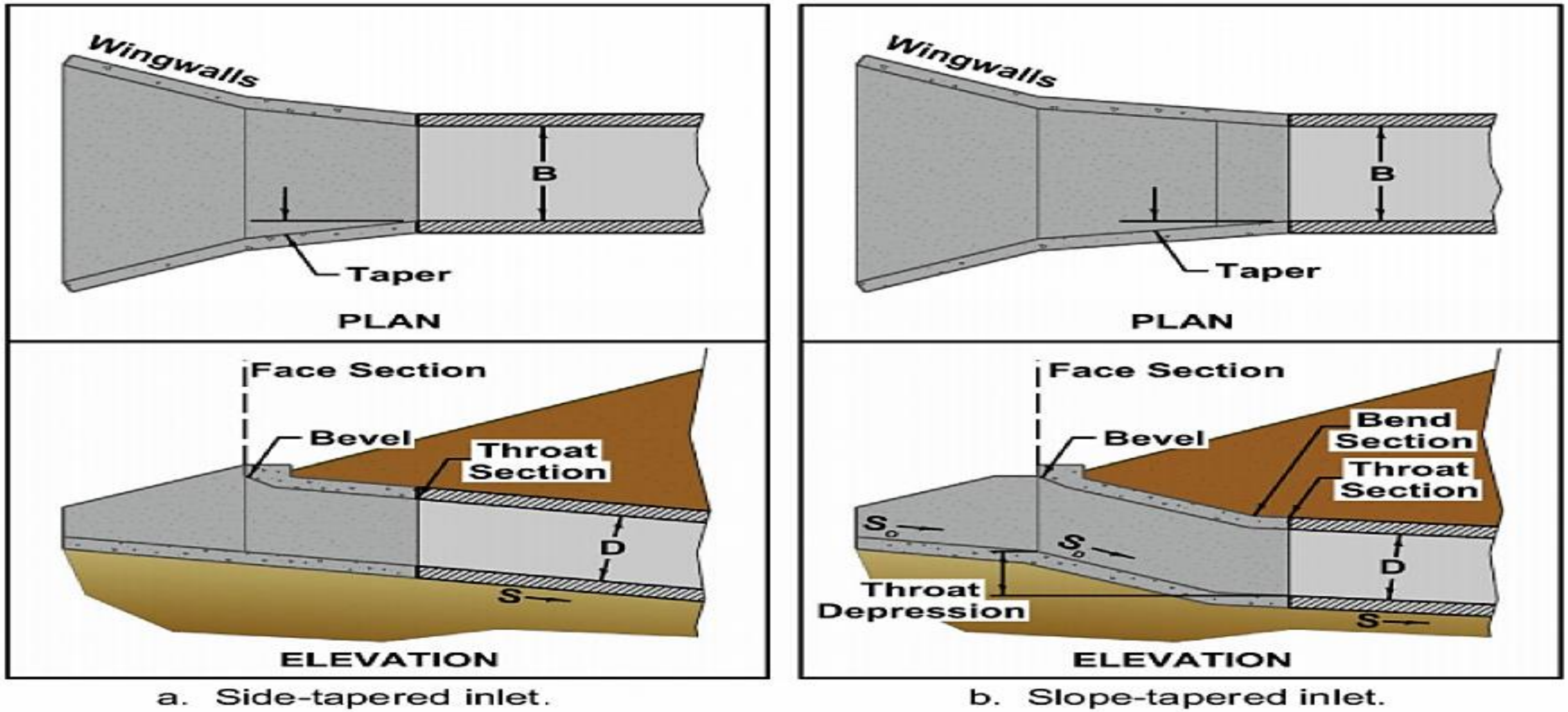
- ✓ *Maximum Velocity* at the culvert exit shall be consistent with the velocity in the natural channel or shall be mitigated with:
  - Channel stabilization
  - Energy dissipation
- ✓ *Minimum Velocity* in the culvert barrel should result in a tractive force ($\tau = \gamma dS$) greater than critical τ of the transported streambed material at low flow rates
- ✓ Use 0.8 m/s when streambed material size is not known.
- ✓ *Design Features*:- the culvert size and shape selected is to be based on engineering and economic criteria related to site conditions

Cont'd…

- ✓ The culvert size and shape selection process shall be based on:-
  - Durability (service life)
  - Cost, availability
  - Construction and maintenance ease
  - Structural strength
  - Traffic delays
  - Abrasion and corrosion resistance
  - Water tightness requirements
- ✓ *Culvert Skew* shall not exceed 45 degrees as measured from a line perpendicular to the roadway centerline.

Cont'd…

- *End Treatment* (Inlet or Outlet):-the culvert inlet or outlet type shall be selected from standard details
- ***Projected inlets or outlets*** are box or pipe culvert extensions beyond the embankment of the roadway



Cont'd…

- ✓ *Improved Inlets* may be considered for culverts that will operate in inlet control to increase the hydraulic performance.
- ✓ *Wing walls* are used to retain the roadway embankment to avoid a projecting culvert barrel and also used where the side slopes of the channel are unstable.
- ✓ Provide the best hydraulic efficiency if the flare angle is between 30°and 60°
- ✓ *Aprons* are used to reduce scour from high headwater depths or from approach velocity in the channel.

Cont'd…

- ✓ *Outlet Protection* (energy dissipaters): efficient energy dissipaters provided at culvert outlet to protect scour.
- ✓ Outlet protection for the selected culvert design flood shall be provided where the outlet scour hole depth computations indicate:
  - – The scour hole will damage the culvert outlet and the expected scour hole may cause costly property damage.

Cont'd…

The outlet velocities of this culvert are almost 4.5 m/sec. Erosion damage is minimal.

Cont'd…

Rip rap erosion protection was required below this culvert. If Fr <3 and Q < 12m3/s, design a riprap basin for each barrel).

Cont'd…

A simple energy dissipater at a culvert outlet. (If Fr > 3, design a stilling basin)

2.5. Hydraulic design of culverts

Important formula

Manning's Equation:

$$Q = (1/n) * A * (R)^{2/3} * (S)^{1/2}$$

$Q = Discharge\ (m3)$

$n = Hydraulic\ roughness\ coefficient$

$A = Area\ of\ the\ pipe\ (m\ 2)$

$R = Hydraulic\ radius\ (m)$

$S = Slope\ of\ the\ culvert$

Weir Equation (unsubmerged Inlet):-

$$Q = Cw\ (Lw)\ h^{3/2}$$

Where:

Q = *discharge (m3/s)*

Cw = *dimensionless weir coefficient (2.2 to 3.1 are typical values for a broad-crested weir)*

Lw = *the length of the weir perpendicular to the flow (m)*

h = *is the energy head (depth + velocity head) of the approaching flow above the crest of the spillway.* ***If the weir is substantially submerged by tailwater, a reduction factor is usually applied.***

- **Orifice Equation (Submerged Inlet):**

$$Q = C_o A_o (2g\ h_o)^{1/2}$$

Where:

Q = *discharge (m3/s)*

C_o = *dimensionless orifice coefficient (0.62 is a typical value for a sharp-edged orifice)*

A_o = *the area of the orifice perpendicular to the flow (m2)*

g = *gravitational constant of 9.81 m/sec 2*

h_o = *the orifice energy head (depth + velocity head) of the approaching flow from inlet centroid (m)*

- ***Energy Grade Line:-*** represents the total energy at any point along the culvert barrel. Equating the total energy at sections 1 and 2, upstream and downstream of the culvert barrel in Figure below, the following relationship results:

$$Hwo + SL + (V_u^2/2g) = TW + (V_d^2/2g) + H_L$$

Where:

HWo = headwater depth above the outlet invert, m

Vu = approach velocity, m/s

TW = tailwater depth above the outlet invert, m

V_d = downstream velocity, m/s

HL = sum of all losses

L = length of culvert

S = slope of culvert.

Head Loss through Submerged Tube :(Barrel Losses)

$$h_L = he + hf + ho$$

Where:

h_L = the total head loss (m)

he = entrance loss (m)

hf = friction loss through the pipe (m)

ho = exit loss (m)

$$he = ke\,(V^2 / 2g)$$

Where:

ke = Entrance loss coefficient

V = the flow velocity just inside the barrel inlet (m / sec)

g = gravitational constant of 9.81m / sec 2

..... $h_f = (V^2 n^2)(L) / (R^{4/3})$

Where:

h_f = *friction loss through the culvert*

V = *average barrel velocity, m/s*

L = *length of pipe*

A = *area of the pipe*

R = *hydraulic radius ($R = A / W_p$ - area divided by the wetted perimeter)*

n = *Hydraulic roughness coefficient (barrel roughness)*

$h_o = k_o [(V^2/2g) - (V_d^2/2g)]$, $k_o = 1$

Where:

V_d = *channel velocity downstream of the culvert, m/s (usually neglected,).*

88

$h_o = h_v = V^2/2g$

Total Loss:

$$h_L = [\ 1 + ke + (19.62\ n^2 L) / R^{4/3}\ ]\ V^2 / 2g$$

Critical Depth:

$$Q^2 / g = A_c^3 / Tc$$ ....*for circular shape*

Where:

Q = *discharge*

g = *the gravitational constant*

Ac = *cross sectional area at critical flow*

Tc = *top width of water surface at critical flow*

For rectangular sections, critical depth can be directly computed as:

$$Yc = (q^2 / g)1/\ 3$$

Where:

q = *discharge per unit width(Q/B)*

g = *the gravitational constant*

Design using nomographs

- ✓ Using nomographs provides a convenient and organized procedure for designing culvert. Its disadvantage is that it needs **trial and error**.
- ✓ *Inlet control during un-submerged inlet* (the entrance operates as a weir).

$$Form\ (1)\ \frac{HW_i}{D} = \frac{H_c}{D} + K\left[\frac{K_u Q}{AD^{0.5}}\right]^M - 0.5 * S^2$$

$$Form\ (2)\ \frac{HW_i}{D} = K\left[\frac{K_u Q}{AD^{0.5}}\right]^M$$

- ✓ *Inlet control during submerged inlet* (the entrance operates as an orifice).

$$\frac{HW_i}{D} = C\left[\frac{K_u Q}{AD^{0.5}}\right]^2 + Y - 0.5 * S^2$$

Where, HW_i = Head water depth above inlet control section invert, m

D = Interior height of culvert barrel, m

H_c= Specific head at critical depth ($d_c + V_c^2/2g$), m

Q = Discharge, m^3/s, A = Full cross-sectional area of culvert barrel, m^2

S = Culvert barrel slope, m/m, K, M, c, Y = Constants from Table 2.1

K_u = 1.811

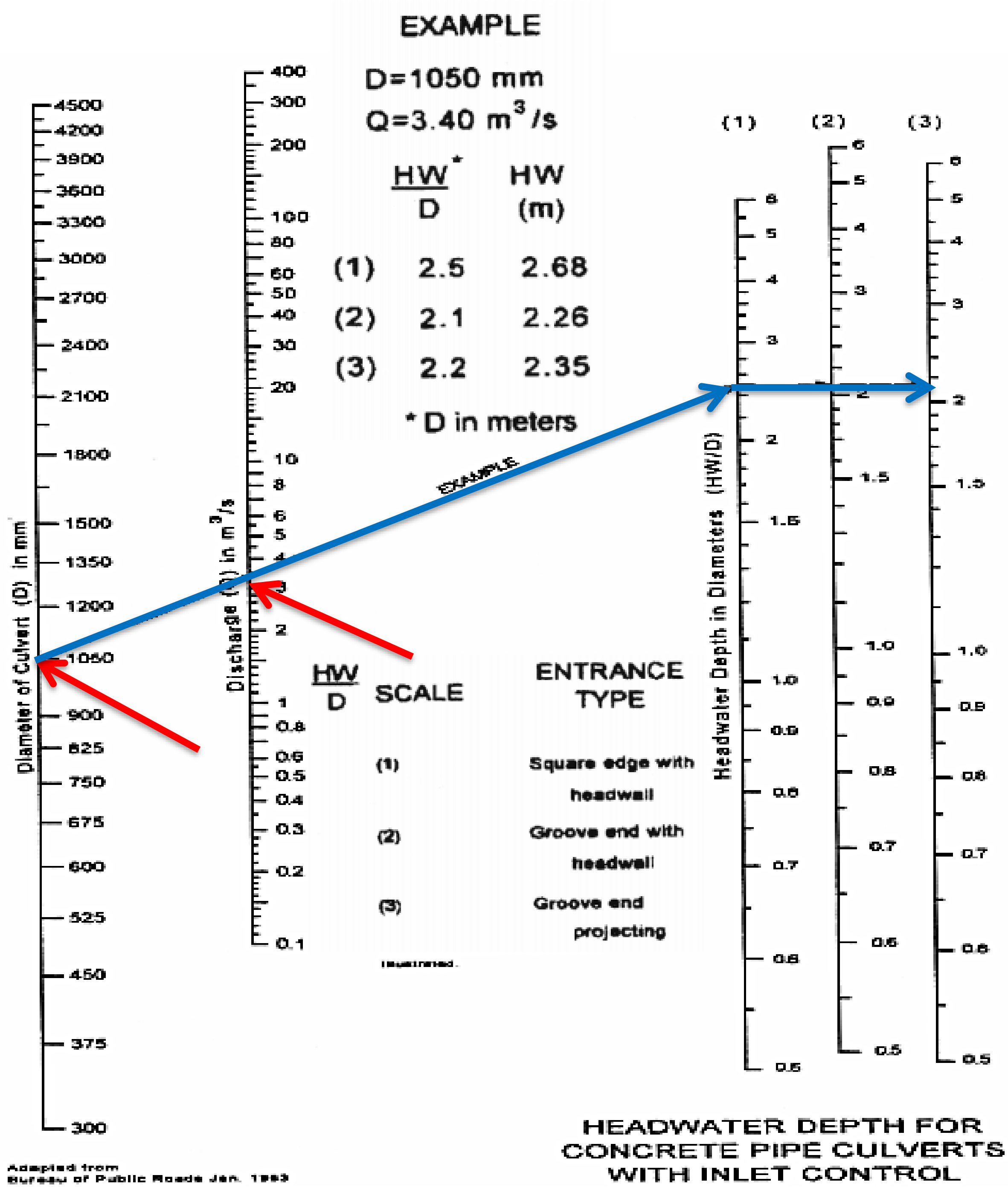
Cont'd…

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged K	Unsubmerged M	Submerged c	Submerged Y	References
1	Circular Concrete	1	Square edge w/headwall	1	.0098	2.0	.0398	.67	56/57
		2	Groove end w/headwall		.0018	2.0	.0292	.74	
		3	Groove end projecting		.0045	2.0	.0317	.69	
2	Circular CMP	1	Headwall	1	.0078	2.0	.0379	.69	56/57
		2	Mitered to slope		.0210	1.33	.0463	.75	
		3	Projecting		.0340	1.50	.0553	.54	
3	Circular	A	Beveled ring, 45° bevels	1	.0018	2.50	.0300	.74	57
		B	Beveled ring, 33.7° bevels*		.0018	2.50	.0243	.83	
8	Rectangular Box	1	30° to 75° wingwall flares	1	.026	1.0	.0347	.81	56
		2	90° and 15° wingwall flares		.061	.75	.0400	.80	56
		3	0° wingwall flares		.061	.75	.0423	.82	8
9	Rectangular Box	1	45° wingwall flare d = .043D	2	.510	.667	.0309	.80	8
		2	18° to 33.7° wingwall flare d = .083D		.486	.667	.0249	.83	
10	Rectangular Box	1	90° headwall w/3/4" chamfers	2	.515	.667	.0375	.79	8
		2	90° headwall w/45° bevels		.495	.667	.0314	.82	
		3	90° headwall w/33.7° bevels		.486	.667	.0252	.865	
11	Rectangular Box	1	3/4" chamfers; 45° skewed headwall	2	.545	.667	.04505	.73	8
		2	3/4" chamfers; 30° skewed headwall		.533	.667	.0425	.705	
		3	3/4" chamfers; 15° skewed headwall		.522	.667	.0402	.68	
		4	45° bevels; 10°-45° skewed headwall		.498	.667	.0327	.75	
12	Rectangular Box 3/4" chamfers	1	45° non-offset wingwall flares	2	.497	.667	.0339	.803	8
		2	18.4° non-offset wingwall flares		.493	.667	.0361	.806	
		3	18.4° non-offset wingwall flares 30° skewed barrel		.495	.667	.0386	.71	
13	Rectangular Box Top Bevels	1	45° wingwall flares - offset	2	.497	.667	.0302	.835	8
		2	33.7° wingwall flares - offset		.495	.667	.0252	.881	
		3	18.4° wingwall flares - offset		.493	.667	.0227	.887	
16-19	C M Boxes	2	90° headwall	1	.0083	2.0	.0379	.69	57
		3	Thick wall projecting		.0145	1.75	.0419	.64	
		5	Thin wall projecting		.0340	1.5	.0496	.57	

Cont'd…

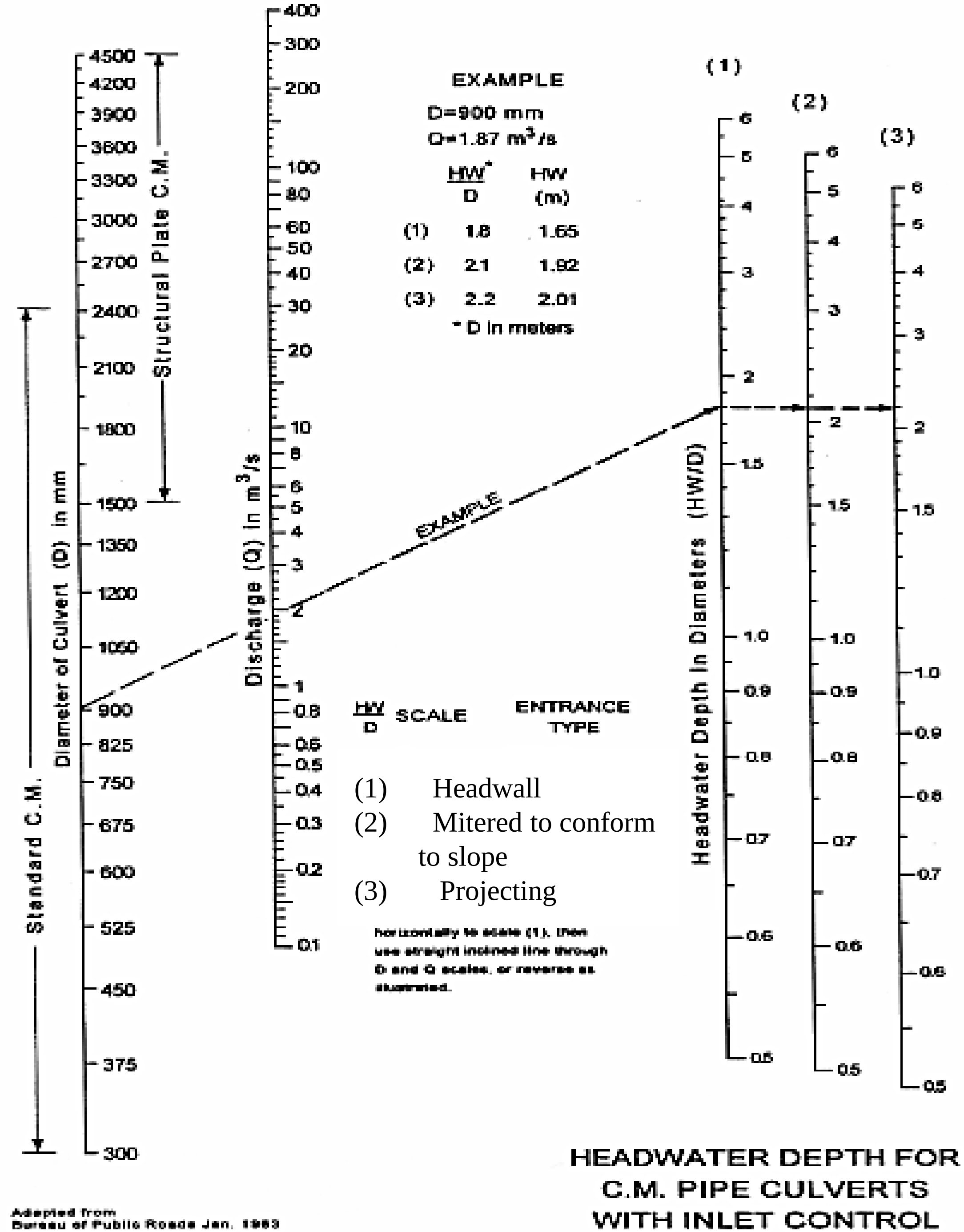
- ✓ In the inlet control design nomographs, *HW* is measured to the *total upstream energy grade line* including the approach velocity head.
- ✓ Inlet control nomographs are shown below as *Charts 2-1, 2-2 and 2-3, for concrete pipe culverts, corrugated metal culverts, and box culverts* respectively.

Cont'd…

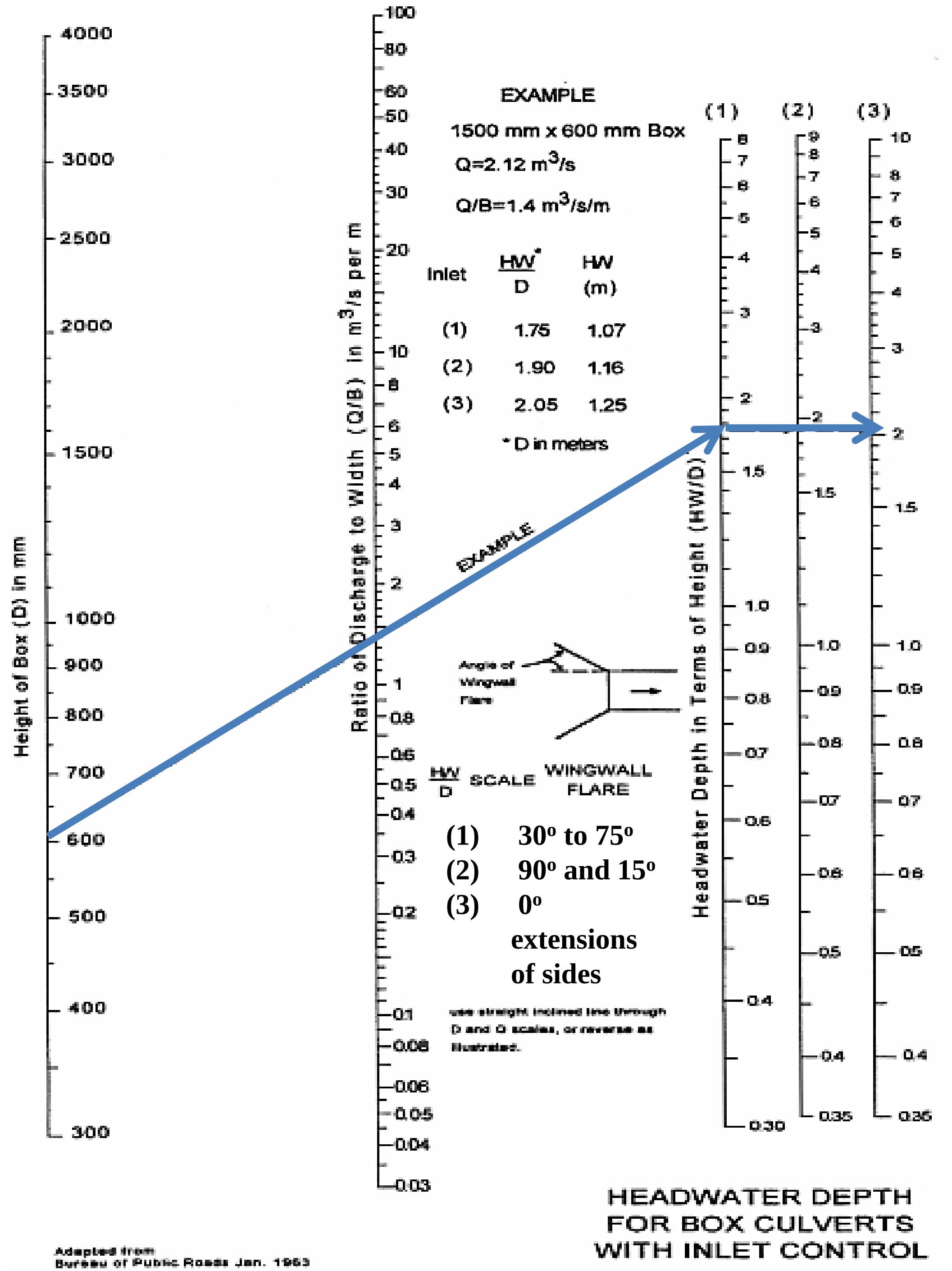


EXAMPLE
D=1050 mm
Q=3.40 m³/s
HW*/D HW (m)
(1) 2.5 2.68
(2) 2.1 2.26
(3) 2.2 2.35
* D in meters
EXAMPLE
Diameter of Culvert (D) in mm
Discharge (Q) in m³/s
Headwater Depth in Diameters (HW/D)
HW/D SCALE
ENTRANCE TYPE
(1) Square edge with headwall
(2) Groove end with headwall
(3) Groove end projecting
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL
Adapted from Bureau of Public Roads Jan. 1963

Cont’d…


EXAMPLE
D=900 mm
Q=1.87 m³/s
HW*/D HW (m)
(1) 1.8 1.65
(2) 2.1 1.92
(3) 2.2 2.01
* D in meters
Structural Plate C.M.
Standard C.M.
Diameter of Culvert (D) in mm
Discharge (Q) in m³/s
Headwater Depth in Diameters (HW/D)
EXAMPLE
HW/D SCALE
ENTRANCE TYPE
(1) Headwall
(2) Mitered to conform to slope
(3) Projecting
horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.
Adapted from Bureau of Public Roads Jan. 1963
HEADWATER DEPTH FOR C.M. PIPE CULVERTS WITH INLET CONTROL

Cont'd…



EXAMPLE
1500 mm x 600 mm Box
Q=2.12 m3/s
Q/B=1.4 m3/s/m
Inlet HW*/D HW (m)
(1) 1.75 1.07
(2) 1.90 1.16
(3) 2.05 1.25
* D in meters
Height of Box (D) in mm
Ratio of Discharge to Width (Q/B) in m3/s per m
Headwater Depth in Terms of Height (HW/D)
EXAMPLE
Angle of Wingwall Flare
HW/D SCALE WINGWALL FLARE
(1) 30º to 75º
(2) 90º and 15º
(3) 0º extensions of sides
use straight inclined line through D and Q scales, or reverse as illustrated.
HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL
Adapted from Bureau of Public Roads Jan. 1963

Cont'd…

Example # 2.2

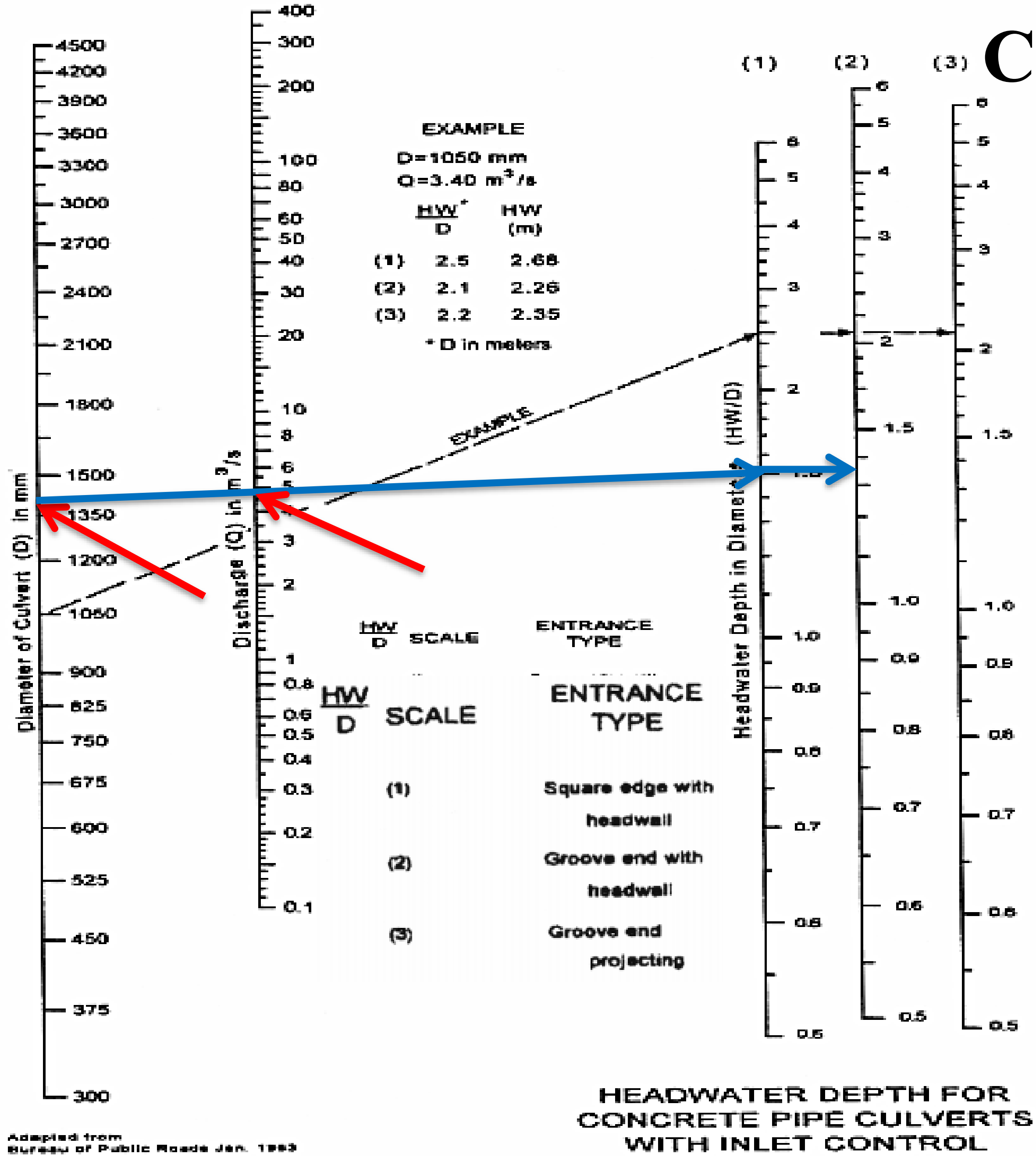
- ✓ The inlet of a 1.4m diameter reinforced concrete pipe (RCP) is submerged by a flow of $5m^3/s$. The inlet has a Groove end treatment with headwall. The slope of the culvert is 0.015 m/m. (a) What is the inlet control headwater depth above the inlet invert (flow line)? (b) Compute by using nomograph.

Example # 2.3

- ✓ Compute the inlet control headwater depth and elevation using the nomographs. The headwater needs to be determined for a discharge of $2.88m^3/s$ through a 1.2 m by 1.2m concrete box culvert. The culvert has an entrance with wing wall 45^o. The barrel has an upstream invert elevation of 30m at the inlet.

Cont'd…

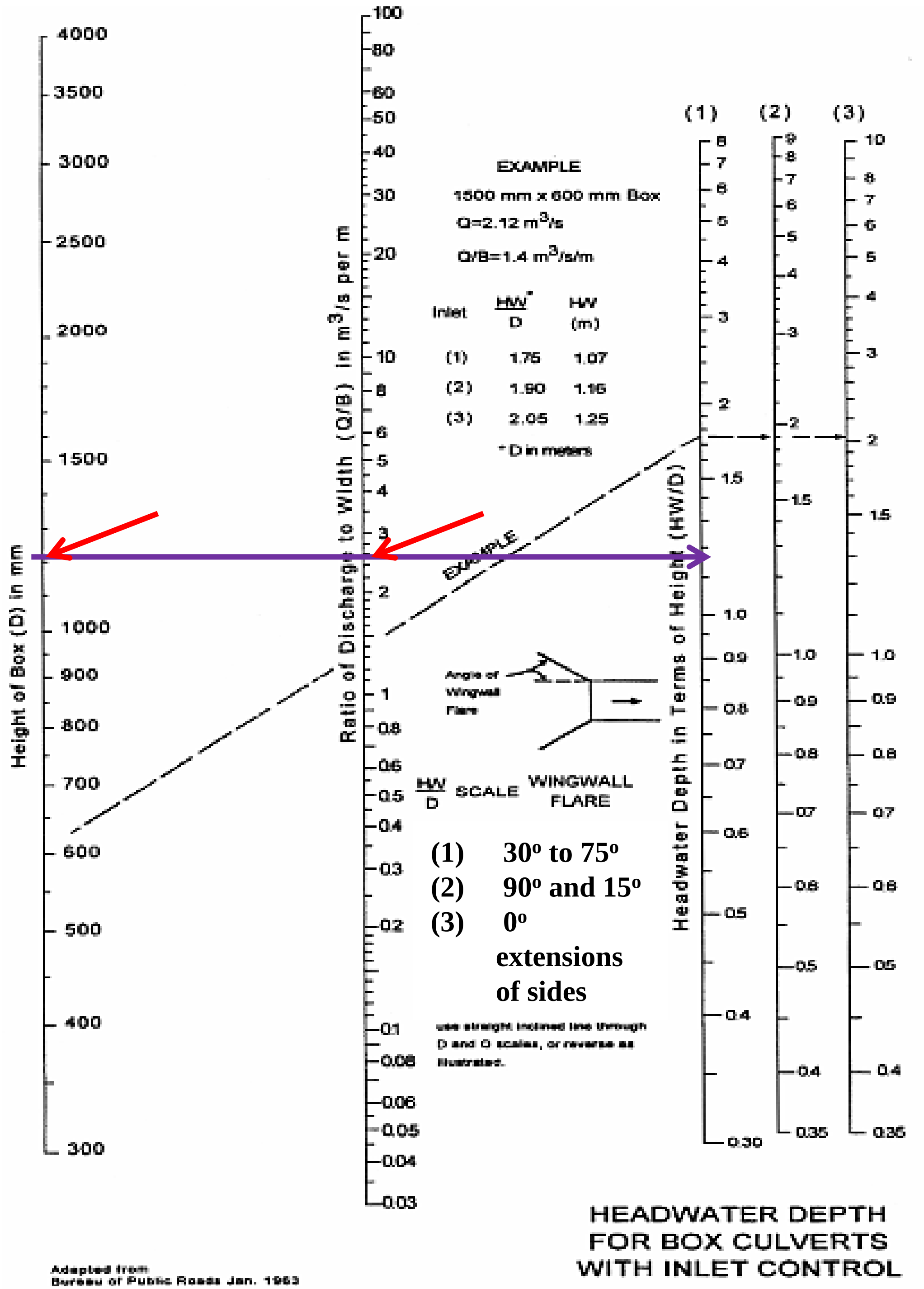
$Q = 5m^3/s$
$D = 1.4m$
$HW/D = 1.35$
$HW = 1.89m$



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

Cont’d…

$Q = 2.88 m^3/s$
$B = 1.2 m$
$Q/B = 2.4 m^2/s$
$HW/D = 1.2$
$HW = 1.44 m$



Cont'd…

- *Outlet Control Nomographs (full flow):* the nomographs were developed assuming that the culvert barrel is flowing full and:
  - TW > D, Flow Type IV
  - dc > D, Flow Type VI
- V_u is small and its velocity head can be considered as a part of the available headwater (HW) used to convey the flow through the culvert.
- V_d is small and its velocity head can be neglected.

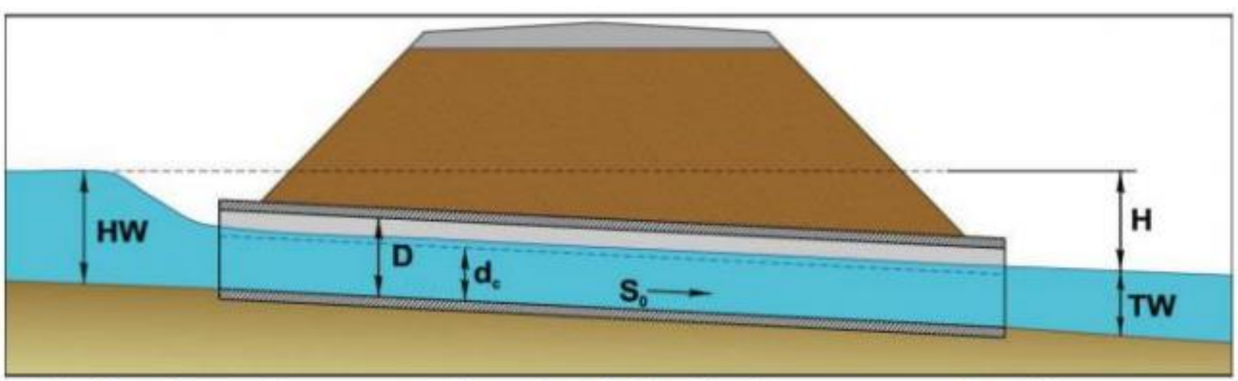
Energy Equation becomes:

$HW = TW + H - S_oL$

- HW = depth from the inlet invert to the energy grade line, m
- H = is the value read from the nomographs, m
- S_oL = drop from inlet to outlet invert, m

Cont'd…

- *Outlet Control Nomographs (Partly full flow):-* for flow type 2 & 3, approximate method. The following equation should be used:- $HW = h_o + H - S_oL$
  - h_o = the larger of TW or (dc+ D)/2, m
  - HW = depth from the inlet invert to the energy grade line, m
  - H = is the value read from the nomographs, m
  - S_oL = drop from inlet to outlet invert, m
- If the headwater depth falls below 0.75D, the approximate method shall not be used.



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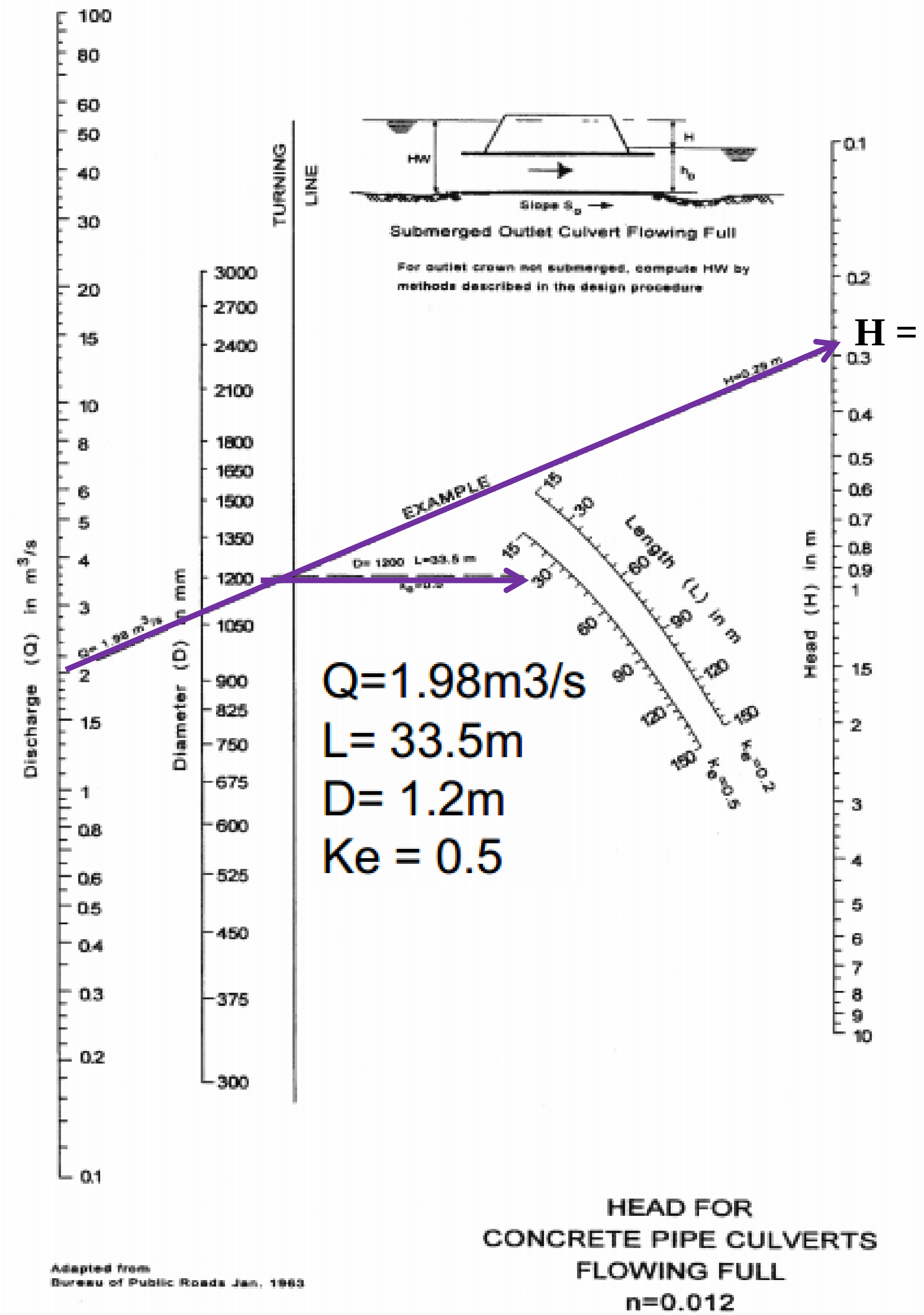
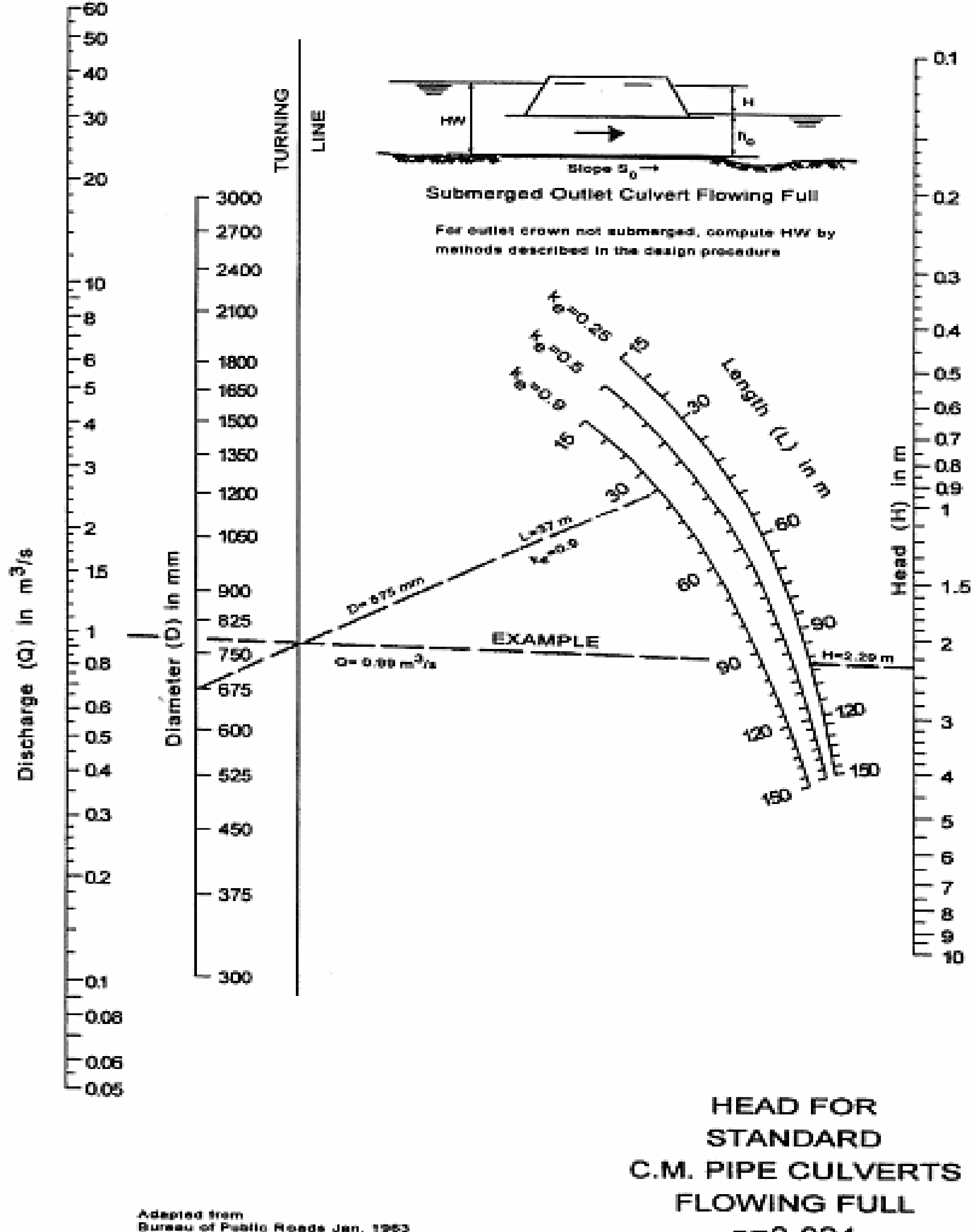
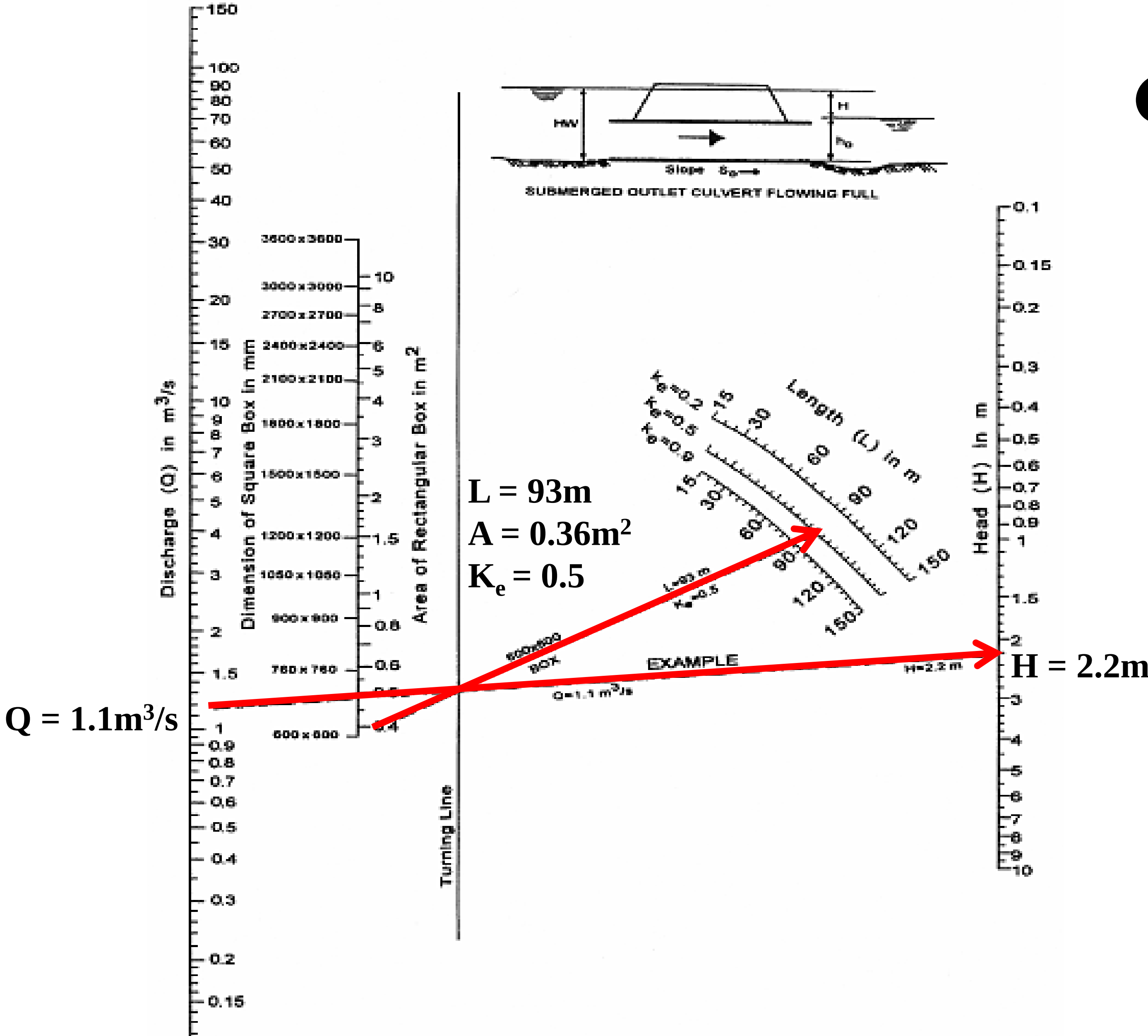


H = 0.29m
Q=1.98m3/s
L= 33.5m
D= 1.2m
Ke = 0.5
Submerged Outlet Culvert Flowing Full
For outlet crown not submerged, compute HW by methods described in the design procedure
Discharge (Q) in m3/s
Diameter (D) in mm
TURNING LINE
EXAMPLE
Length (L) in m
Head (H) in m
HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n=0.012
Adapted from
Bureau of Public Roads Jan. 1963

Cont'd…


Submerged Outlet Culvert Flowing Full
For outlet crown not submerged, compute HW by methods described in the design procedure
TURNING LINE
Discharge (Q) in m³/s
Diameter (D) in mm
Head (H) in m
Length (L) in m
Ke=0.25
Ke=0.5
Ke=0.9
EXAMPLE
D=675 mm
L=37 m
ke=0.9
Q=0.99 m³/s
H=2.29 m
Slope S_o
HW
H
h_o
HEAD FOR STANDARD C.M. PIPE CULVERTS FLOWING FULL n=0.024
Adapted from Bureau of Public Roads Jan. 1963

Cont'd…



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n=0.012

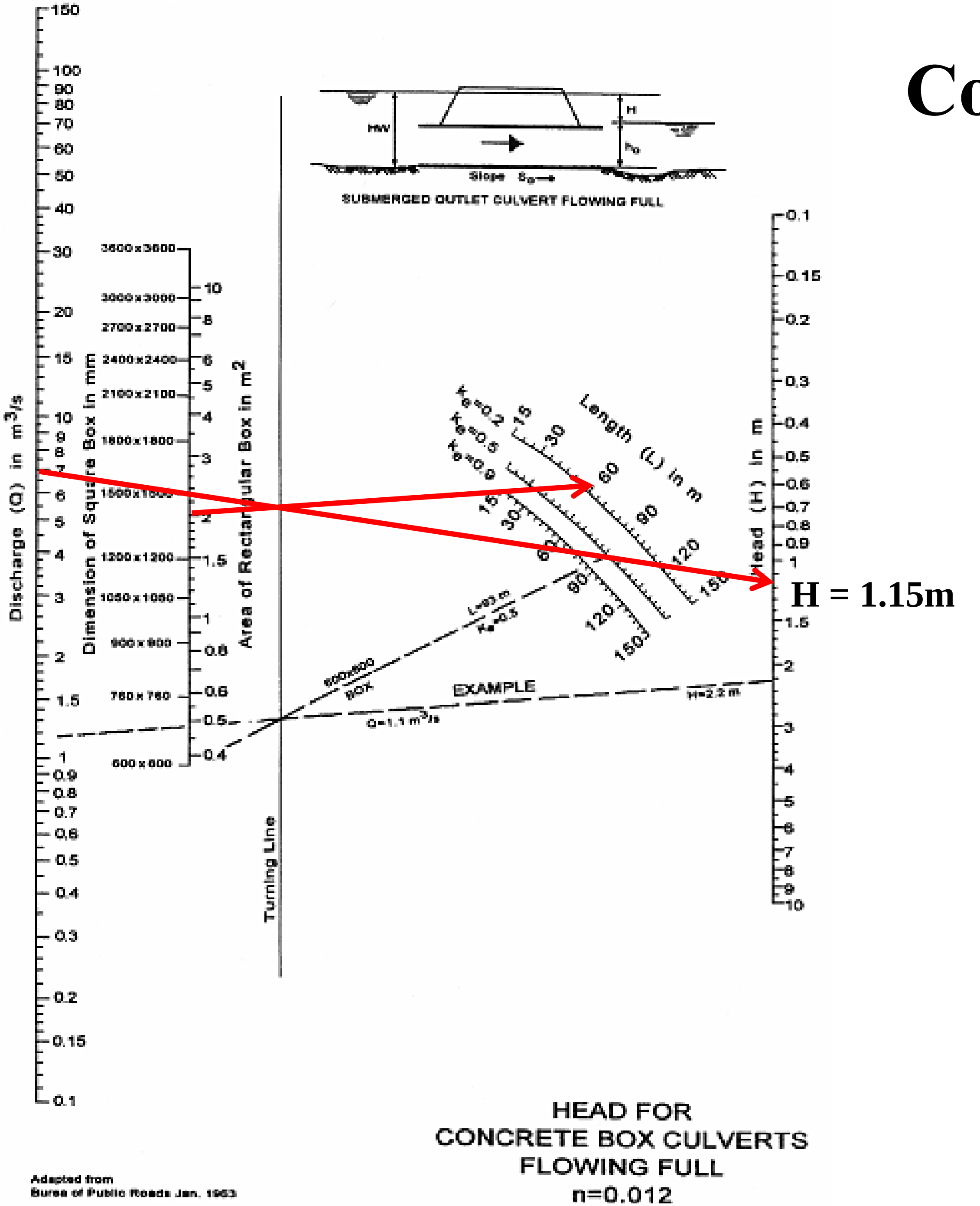
Adapted from
Burea of Public Roads Jan. 1963

Example #2.4

- A 1.4m by 1.4m concrete box culvert has a length of 60m, an entrance loss coefficient Ke of 0.2, and a roughness coefficient of 0.012. The inlet invert elevation “Eli” is 29.5m. The outlet invert elevation “Elo” is 27.5m. The tailwater depth at the outlet is 2.5m and submerges the outlet. Calculate the outlet control headwater elevation “$ELHw_o$” and depth for a flow of $7m^3/s$.

Cont'd…

Given
Q = 7m³/s
L = 60m
Ke = 0.2
D = 1400 * 1400



Design procedure

- The following design procedure provides a convenient method for designing culverts for a constant discharge, considering ***inlet*** and *outlet control.*
- Step 1:-Assemble Site Data and Project File

a. Hydrographic Survey - Data include:
   - Topographic and location maps,
   - Embankment cross section, roadway profile
   - Field visit (sediment, debris) and
   - Design data of nearby structures

b. Studies by other agencies including Ministry of Water Resources.

c. Design criteria: Standards and departures from standards for applicable criteria

Cont'd…

- *Step 2:-Determine Hydrology*, Minimum data required:- drainage area maps and discharge frequency plots.
- *Step 3:-Design Downstream Channel*, Minimum data are cross section of channel and the rating curve for channel.
- *Step 4:-Summarize Data on Design Form*, Collect all data from the preceding steps and record on a single design form.
- *Step 5:-Select Design Alternative*

  a. See design features.

  b. Choose culvert material, shape, size, and entrance type.

Cont’d…

- *Step 6:- Select Design Discharge Q_d*, see
  a. Design limitations
  b. Determine flood frequency from criteria
  c. Determine Q from discharge-frequency plot (Step 2)
  d. Divide Q by the number of barrels
- *Step 7:- Determine Inlet Control Headwater Depth (HWi)*

Use the inlet control “nomograph”

  a. Locate the size or height on the scale
  b. Locate the discharge
    - For a circular shape use discharge(Q)
    - For a box shape use Q/B
  c. Locate HW/D ratio using a straight edge.
  d. Calculate headwater depth (HW)

Cont'd…

- *Step 8:-Determine Outlet Control Headwater Depth at Inlet (HWoi)*
  a. Calculate "TW" depth *using the design flow rate and normal depth (single section)* or *using a water surface profile*.
  b. Calculate critical depth (dc), dc cannot exceed D
  c. Determine (h_o) = the larger of TW or (dc + D)/2
  d. Determine entrance loss coefficient (Ke) from Table
  e. Determine losses through the culvert barrel (H)
  f. Calculate outlet control headwater (HW)
- Use $HWoi = H + h_o - S_oL$, (if V_u and V_d are neglected)
- Use energy equation at *approach section* & *tailwater section* to include V_u and V_d, if these values are significant.

Cont'd…

- *Step 9:-Determine Controlling Headwater (HWc)*
  - Compare **HWi** and **HWoi**, use the higher
  - HWc = HWi, if HWi > HWoi
  - *The culvert is in inlet control*
  - HWc = HWoi, if HWoi > HWi
  - *The culvert is in outlet control.*
- *Step 10:-Compute Discharge Over the Roadway (Qr)*
- a. Calculate *depth above the roadway* (HWr)
  - HWr = HWc - HWov
  - HWov *is height of road above inlet invert*
- b. If HWr ≤ 0, Qr = 0 *(no overtopping)*
  - If HWr > 0, determine Qr *(overtopping)*

Cont'd…

- ***Step 11***:- *Compute Total Discharge (Qt)*
  - Qt = Qd + Qr
- ***Step 12***:- *Calculate Outlet Velocity (Vo)*
- ***Step 13***:- *Review Results*

Compare alternative design with constraints and assumptions, *if any of the following will be exceeded*, repeat Steps 5 through 12.

- The barrel must have adequate cover
- The length should be close to the approximate length
- The headwalls and wing-walls must fit site conditions
- The allowable headwater should not be exceeded and
- The allowable overtopping flood frequency should not be exceeded

Cont'd…

- *Step 14*:- *Plot Performance Curve*
- A performance curve is a plot of *headwater depth* or *elevation* versus *flow rate*.

a. Repeat Steps 6 through 12 with a *range of discharges*

b. Use the following *upper limit for discharge*

   - Q_{100} if $Q_d \leq Q_{100}$
   - Q_{500} if $Q_d > Q_{100}$
   - Q_{max} if no overtopping is possible

Cont'd…

- *<u>Step 15</u>:- Related Designs*
- Consider the following options (*design features* and *related design*):
  - *Tapered inlets* if culvert is in inlet control and has limited available headwater.
  - *Flow routing* if a large upstream headwater pool exits.
  - *Energy dissipaters* if V_o is larger than the normal V in the downstream channel.
  - *Sediment control storage* for sites with sediment concerns such as alluvial fans.
- *<u>Step 16</u>:- Documentation*
- Prepare report and file with background information

Example (follow the Design Procedure Steps)

- Step 1: Assemble site data and project file

Site survey project file contains:

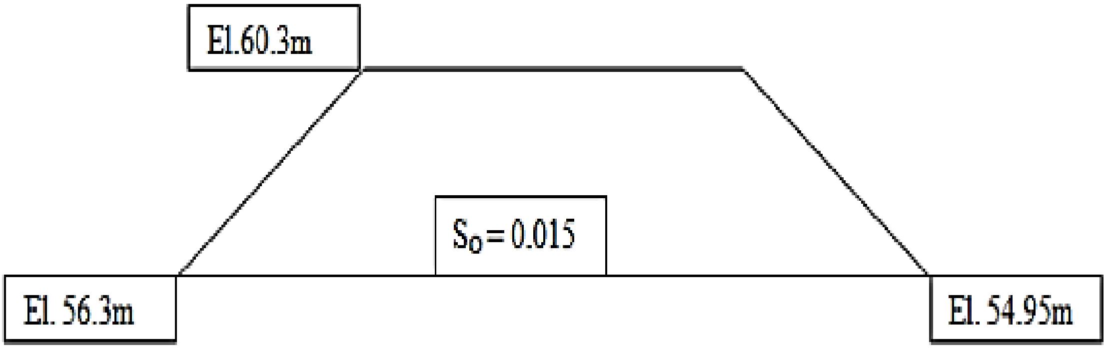


Figure Cross-Section

- Roadway profile and
- Embankment cross-section

- Design criteria
  - 50-year frequency for design and
  - 100-year frequency for check

Cont'd…

- Step 2: Determine Hydrology
  - $Q_{50} = 11.33\ m^3/s$
  - $Q_{100} = 14.16 m^3/s$
- Step 3: Design Downstream Channel

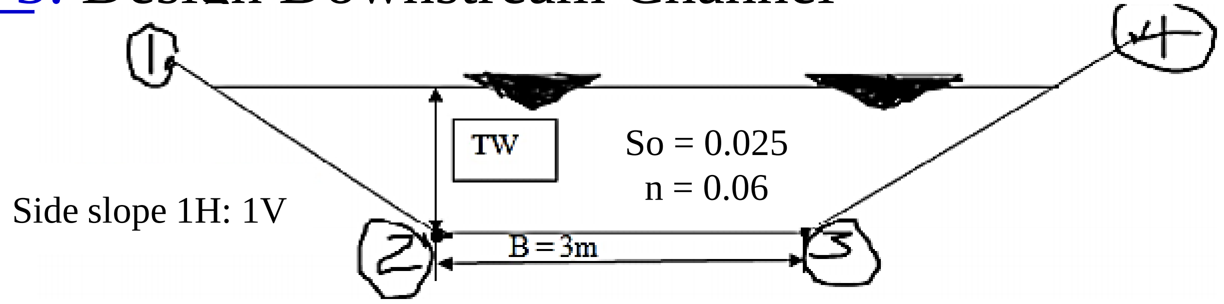


Figure Cross-Section of Channel

Points	Elevation, m
1	56.86
2	54.62
3	54.62
4	56.86

Cont'd…

- The rating curve for the channel calculated by normal depth yields: ($Q = (1/n)AR^{2/3}S^{1/2}$)

Q (m^3/s)	TW (m)	V (m/s)
2.80	0.72	1.04
5.60	0.97	1.45
8.50	1.08	1.93
11.33	1.2	2.24
14.16	1.36	2.39

Step 4: Summarize Data on Design Form

Step 5: Select Design Alternatives

- Shape: - box Size -2100 mm by 2100 mm
  - Material: - concrete
  - Entrance: - Wing walls, 45º bevel, rounded

Cont'd…

- Step 6: Select Design Discharge

  $Q_d = Q_{50} = 11.33\ m^3/s$

- Step 7: Determine Inlet Control Headwater Depth (HWi)

  Use inlet control monograph Chart 7-6

  – a. D = 2.1 m

  – b. Q/B = 11.33/2.1 = 5.4m^2/s

  – c. HW/D = 1.1 for 45^o bevel (Chart 7.6)

  – d. HWi = (HW/D)*D = (1.1)*2.1m = 2.3m (Neglect the approach velocity)

- Step 8: Determine Outlet Control Headwater Depth at
- Inlet (HWoi)

  – a. TW = 1.2 m for Q_{50} = 11.33 m3/s

  – b. dc = 1.44 m from (Yc = (q^2/ g)1/3)

Cont’d…

- c. (dc + D)/2 = (1.44 + 2.1)/2 = 1.77 m
- d. h_o = the larger of TW or (dc + D)/2

  ho = (dc + D)/2 = 1.77 m
- e. Ke = 0.2 from Table for Wing-walls, 45^o bevel, rounded
- f. Determine (hL) -use Chart 7-8

  KE scale = 0.2, culvert length (L) = 90 m

  n = 0.012 same as on chart, area = $4.41m^2$

  H = 0.55 m
- g. HWoi = H + ho -S_oL = 0.55 + 1.77 - (0.015)*90 = 0.97m

• HWoi is less than 1.2D, but control is inlet control, outlet control computations are for comparison only.

Cont'd…

- Step 9: Determine Controlling Headwater (HWc)
  - HWc = HWi = 2.3m > HWoi = 0.97 m
  - The culvert is in inlet control
- Step 10: Compute Discharge over the Roadway (Qr)
  - a. Calculate depth above the roadway:

    HWr = HWc - HWov = 2.3m – 4m = -1.7m
  - b. If HWr $\leq$ 0, Qr = 0
- Step 11: Compute Total Discharge (Qt)
  - Qt = Qd + Qr = 11.33 m3/s + 0 = 11.33 m^3/s

Cont'd…

- Step 12: Calculate Outlet Velocity (V_o) and Depth (d_n) Inlet Control

  a. Calculate normal depth (d_n):

  $Q = (1/n)A\ R^{2/3}S^{1/2} = 11.33\ m^3/s$

  $= (1/0.012)*(2.1*d_n)[(2.1*d_n/(2.1+2d_n)]^{2/3} *(0.015)^{0.5}$

  use $d_n = 0.87$ m,

  b. A = 2.1m*0.87m = 1.827 m

  c. $V_o = Q/A = 11.33/1.827 = 6.2$ m/s

- Step 13: Review Results
- Compare alternative design with constraints and assumptions, if any of the following are exceeded repeat, steps 5 through 12.

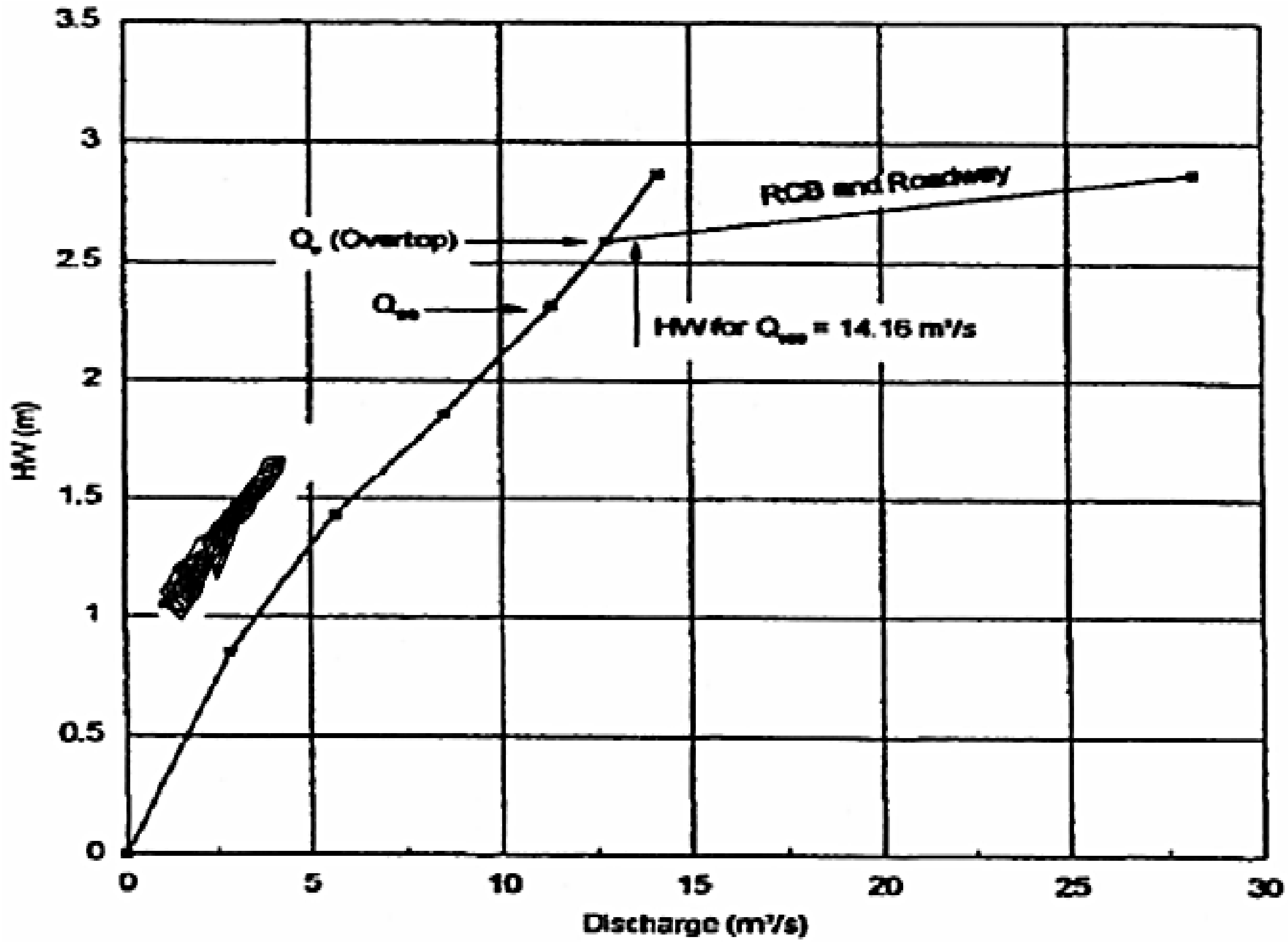
Cont'd…

- barrel has (4m – 2.1m) = 1.9m of cover
- L = 90m
- headwalls and wing-walls fit site
- allowable headwater (3.7m) > 2.3 m is ok and
- overtopping flood frequency > 50-year

- Step 14: Plot Performance Curve

Use Q_{100} for the upper limit, Steps 6 through 12 should be repeated for each discharge used to plot the performance curve

Cont'd…



RCB and Roadway
Q_o (Overtop)
Q_{50}
HW for Q_{100} = 14.16 m³/s
HW (m)
Discharge (m³/s)
3.5
3
2.5
2
1.5
1
0.5
0
0
5
10
15
20
25
30

Cont'd…

- Step 15: Related Designs

Consider the following options (Design Features, and Related Designs)

a) Consider tapered inlets, culvert is in inlet control and has limited available headwater

- No flow routing, a small upstream headwater pool exists
- Consider energy dissipaters since V_o = 6.2m/s > 2.24 m/s in the downstream channel
- No sediment problem

- Step 16: Documentation

Report prepared and background filed

Thank YOU!!!

Chapter Three: Bridge Hydraulics

3.1. Introduction

- ✓ Studying the bridge hydraulics is important to:
  - – Check the effect of constructing the bridge on the upstream reach
  - – Hydraulically inefficient bridge could cause flooding upstream and extremely damaging a large number of properties.
  - – A properly designed bridge is one that balances the cost of the bridge with concerns of safety, impacts to the environment and regulatory requirements.
  - – Proper hydraulic analysis and design of bridges is as vital as structural design.

Cont'd…

Hydraulic analysis of bridge involves the following:

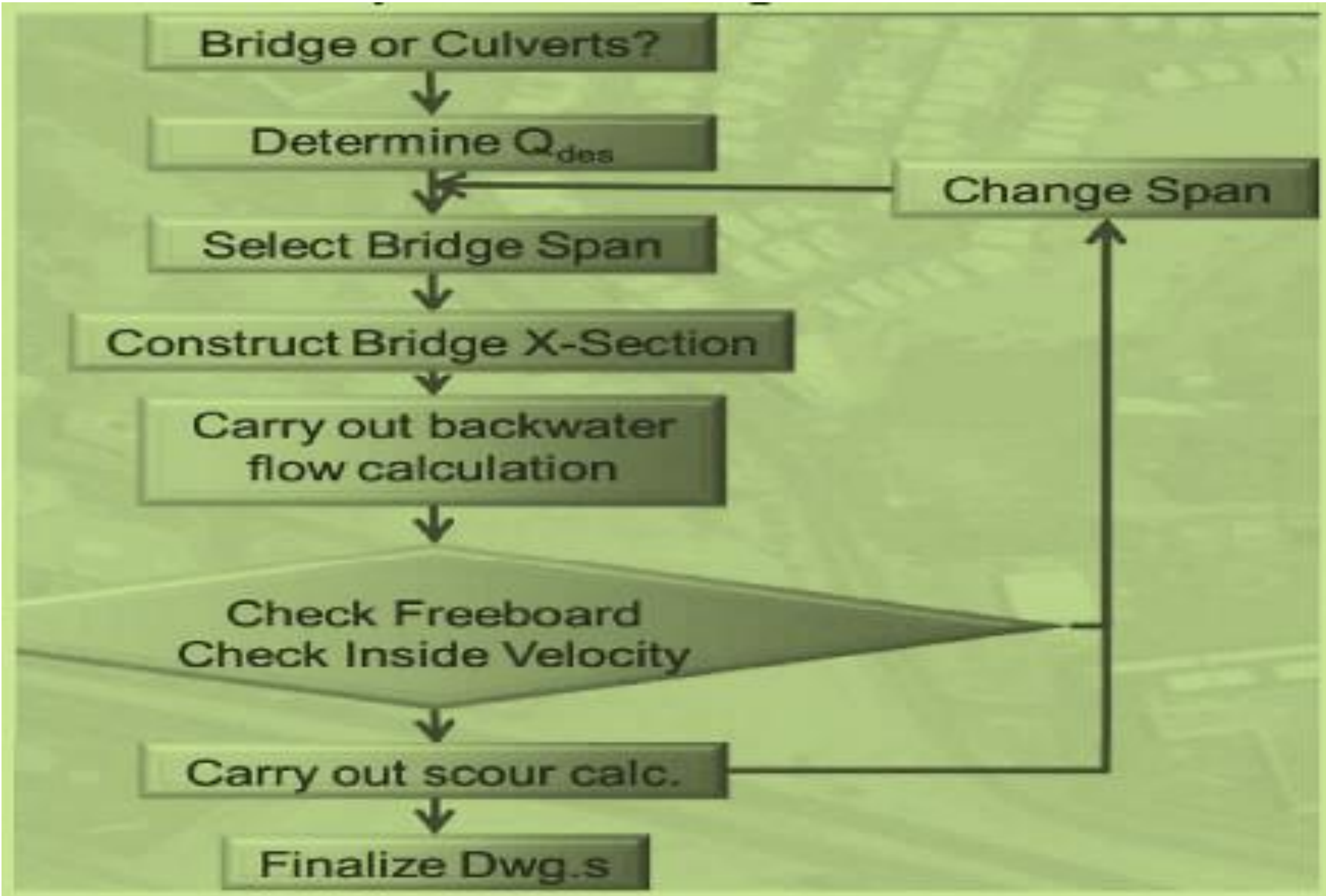
- ✓ Determining the backwater surface profile
- ✓ Determining the effects on flow distribution and velocities
- ✓ Estimating scour potential

The deliverables of the hydraulic analysis of bridge involves the following:

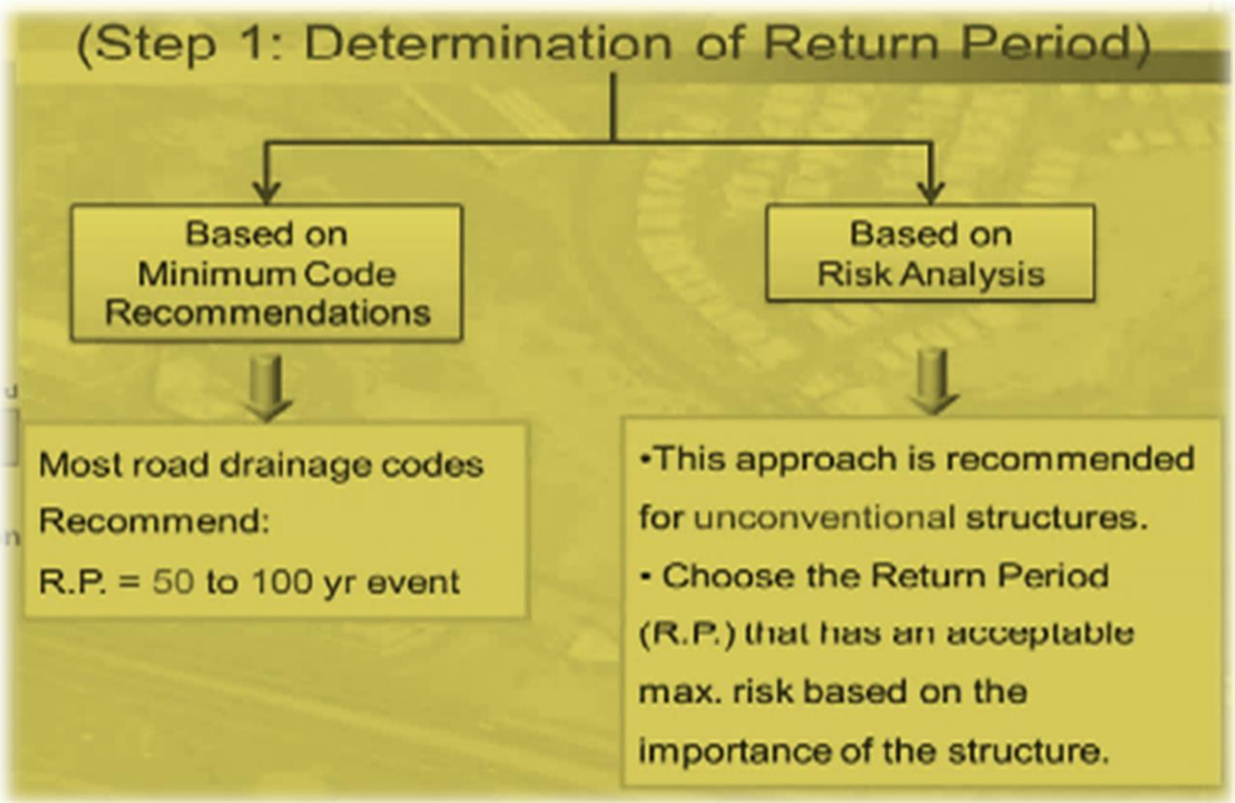
- ✓ Location and orientation of the bridge
- ✓ Bridge span and piers spacing if any
- ✓ Type/shape of piers and abutment
- ✓ Bridge profile and lower chord invert level
- ✓ Recommended foundation levels (scour calculations)
- ✓ Protection extent (upstream and downstream).

Cont’d…

Hydraulic design flow chart



- ✓ Design flow calculations
  - Determine the *design return period*
  - Pick out the *design flow* corresponding to the obtained design return period



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Determination of return period based on risk analysis

Risk:- is simply defined as the probability of failure

- Within n years we have two events only: occurrence of *failure* or *no failure*
- Probability of failure + Probability of no failure at all = 1

i.e Probability of failure in n years = 1 – probability of no failure at all.

- *Probability of no failure* at all in n years = $(1 – 1/R.P.)^n$

$$\text{Risk} \sim 1 - (1 - 1/R.P.)^n$$

Pick out the design flow corresponding to the obtained design return period

Example:- Estimate the risk related to adopting a design return period of 50 years during the first 30 year period.

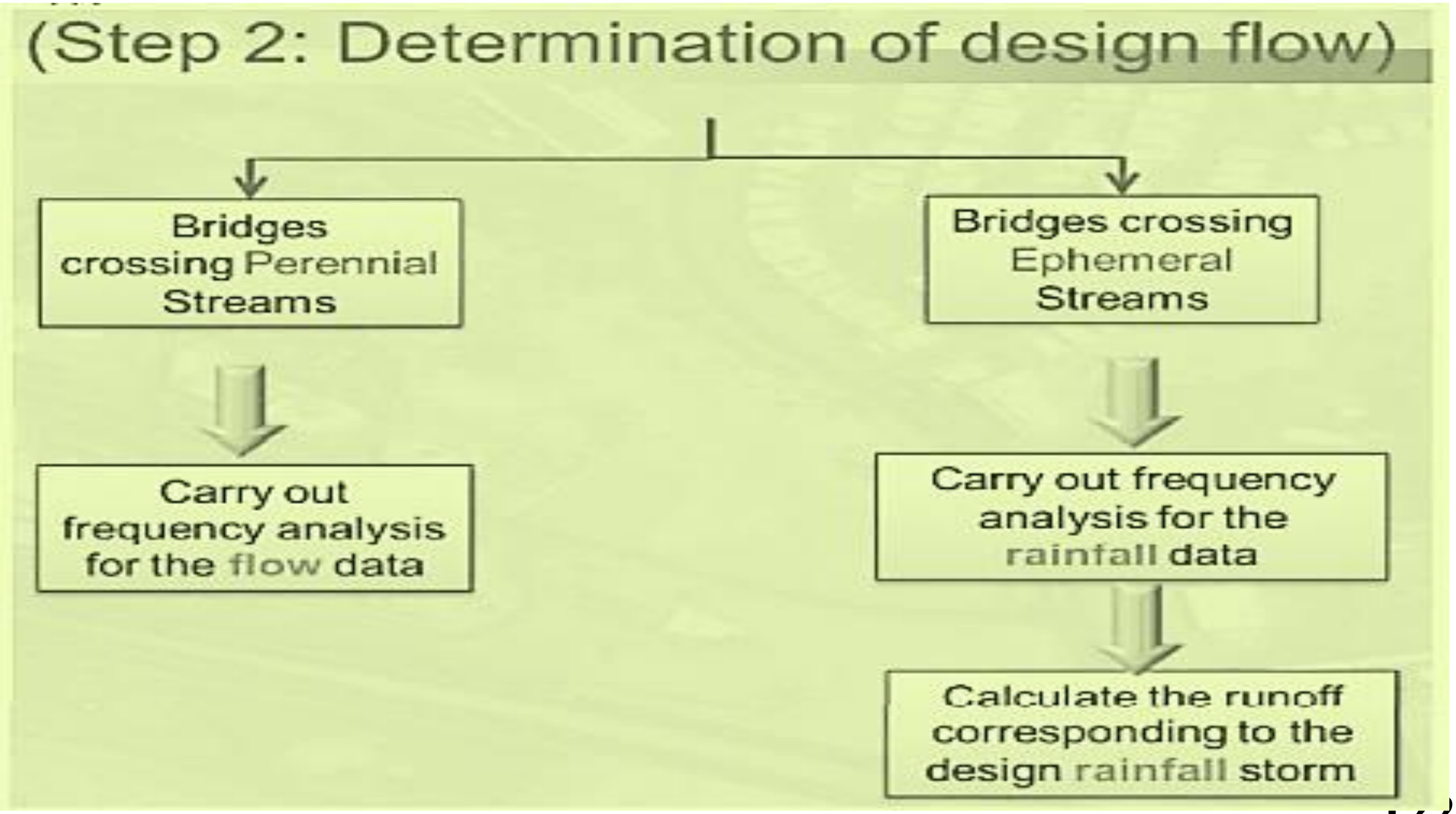
$$\begin{aligned} \text{Risk} &\sim 1 - (1 - 1/R.P.)^n \\ &\sim 1 - (1 - 0.02)^{30} \\ &\sim 0.455 \text{ or } 46\% \end{aligned}$$

Cont'd…

- ✓ If this is too large risk, then increase design level to the 100 year where p = 0.01

Risk ~ $1 - (0.99)^{30}$

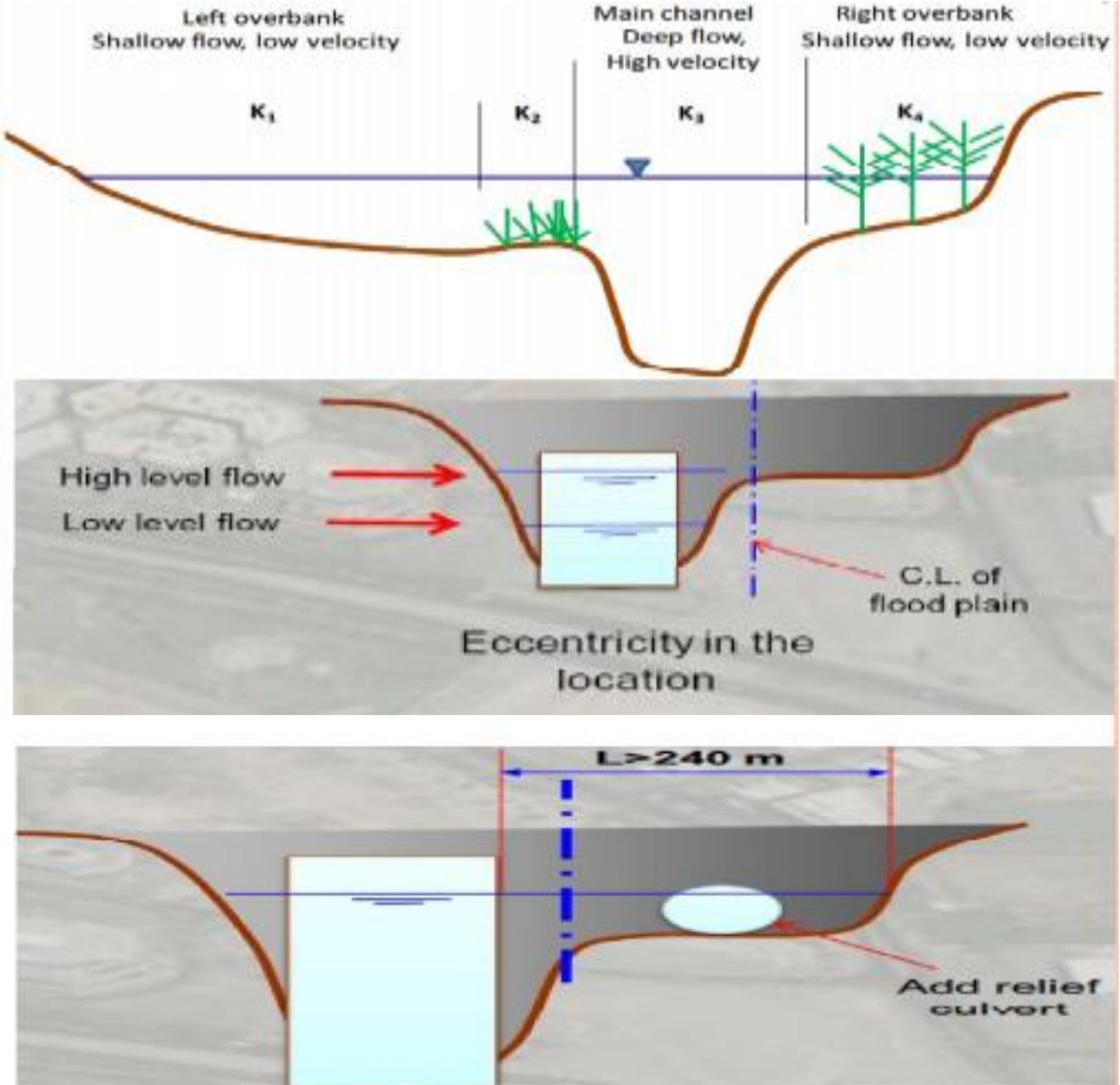
~ 0.26 or 26%



Bridge location

- ✓ Generally, the stream crossing location is primary selected during the planning phase of a highway project.
- ✓ But the final location should not be confirmed unless you obtain the detailed survey information and after completing the preliminary hydraulic studies.
- ✓ *The crossing should*: minimize skew, be located at the narrowest portion of the flood plain, be located on a stable reach of stream, minimize impacts of meander migration. and have appropriately located auxiliary/relief openings (if needed).
- ✓ Locate and center the bridge on the main channel portion of the entire floodplain.

Cont’d…



Left overbank
Shallow flow, low velocity
Main channel
Deep flow,
High velocity
Right overbank
Shallow flow, low velocity
K1
K2
K3
K4
High level flow
Low level flow
C.L. of
flood plain
Eccentricity in the
location
L>240 m
Add relief
culvert

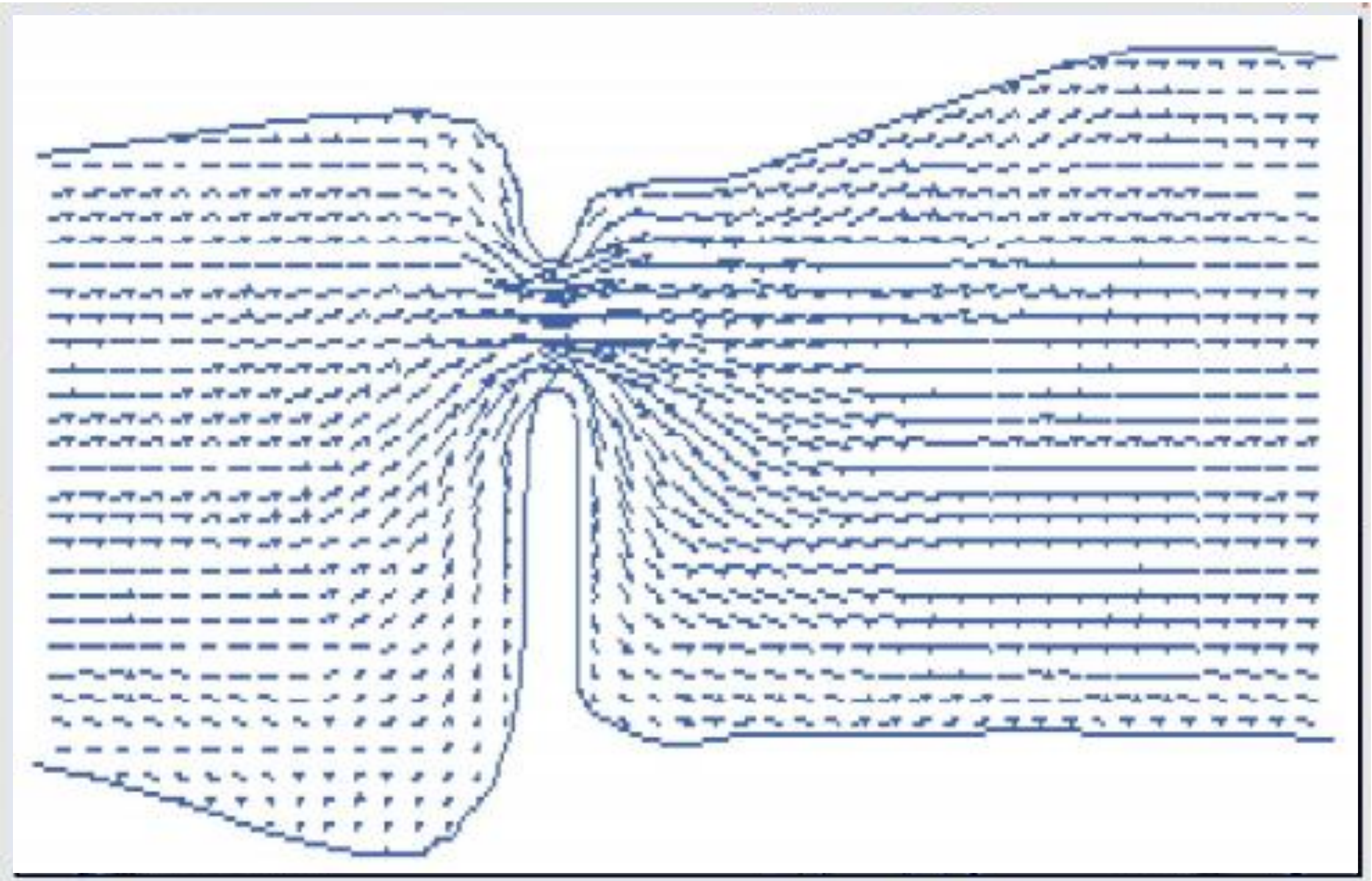
Cont'd…

- ✓ The need for auxiliary waterway openings, or relief openings, arises on streams with wide floodplains.
- ✓ The purpose of the relief openings is to *pass a portion of the flood flow that travels in the floodplain when the stream reaches a certain stage.*
- ✓ The openings *do not provide relief for the principal waterway opening as an emergency spillway of a dam does*, but it has predictable capacity during flood events.

3.2. The effect of bridge on a river flow

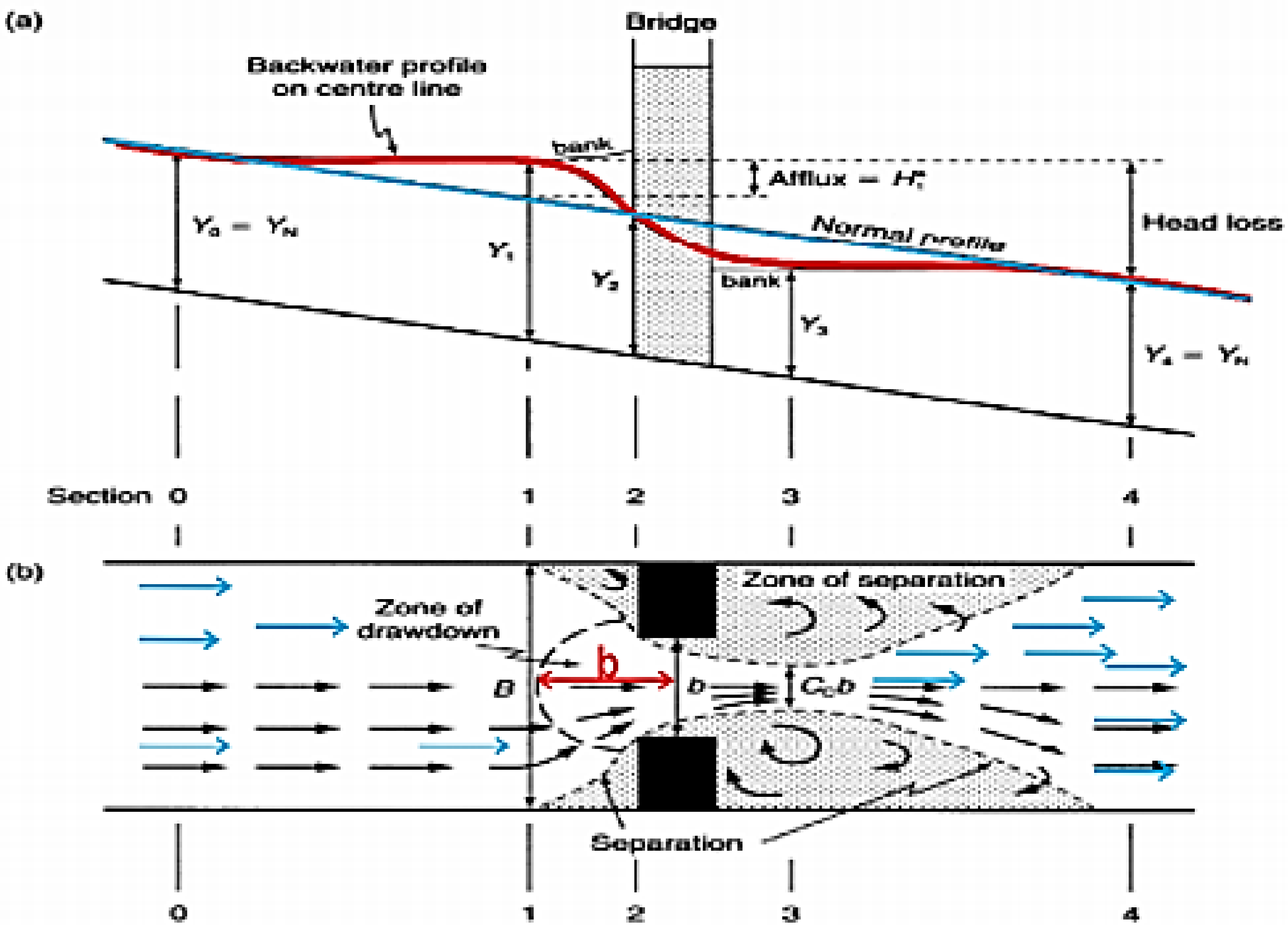
- ✓ When a bridge is placed in a river it forms a narrowing of the natural channel and an obstacle to the flow.
- ✓ This results in a *loss of energy as the flow contracts, passes through the bridge and then re-expands back* to the full channel width.
- ✓ Additional head is necessary to overcome the energy loss, the upstream water level increases above that level.
- ✓ *This additional head is called the afflux,* and its variation with distance upstream is called the ***backwater profile***.
- ✓ If the narrowing is very severe, the flow is usually subcritical with *GVF upstream and downstream of the structure* and *RVF at the bridge.*

Cont'd…



Cont'd…

- ✓ When investigating, the hydraulic capacity of the main *river channel without the bridge (Q_R)* should be compared with the *capacity of the bridge waterway (Q_{WB})* and the design flood (Q_{DF}).
- ✓ Then as a rough guide:
  - If $Q_R < Q_{WB}$ the bridge is relatively blameless
  - If $Q_R < Q_{DF}$ flooding of the floodplains would occur without the bridge
  - If $Q_{WB} < Q_R$ the bridge forms is under an obstacle to flow and may cause flooding
  - If $Q_{WB} < Q_{DF}$ the waterway is under designed
  - If $Q_{WB} > Q_{DF}$ the waterway is over designed or has a margin of safety

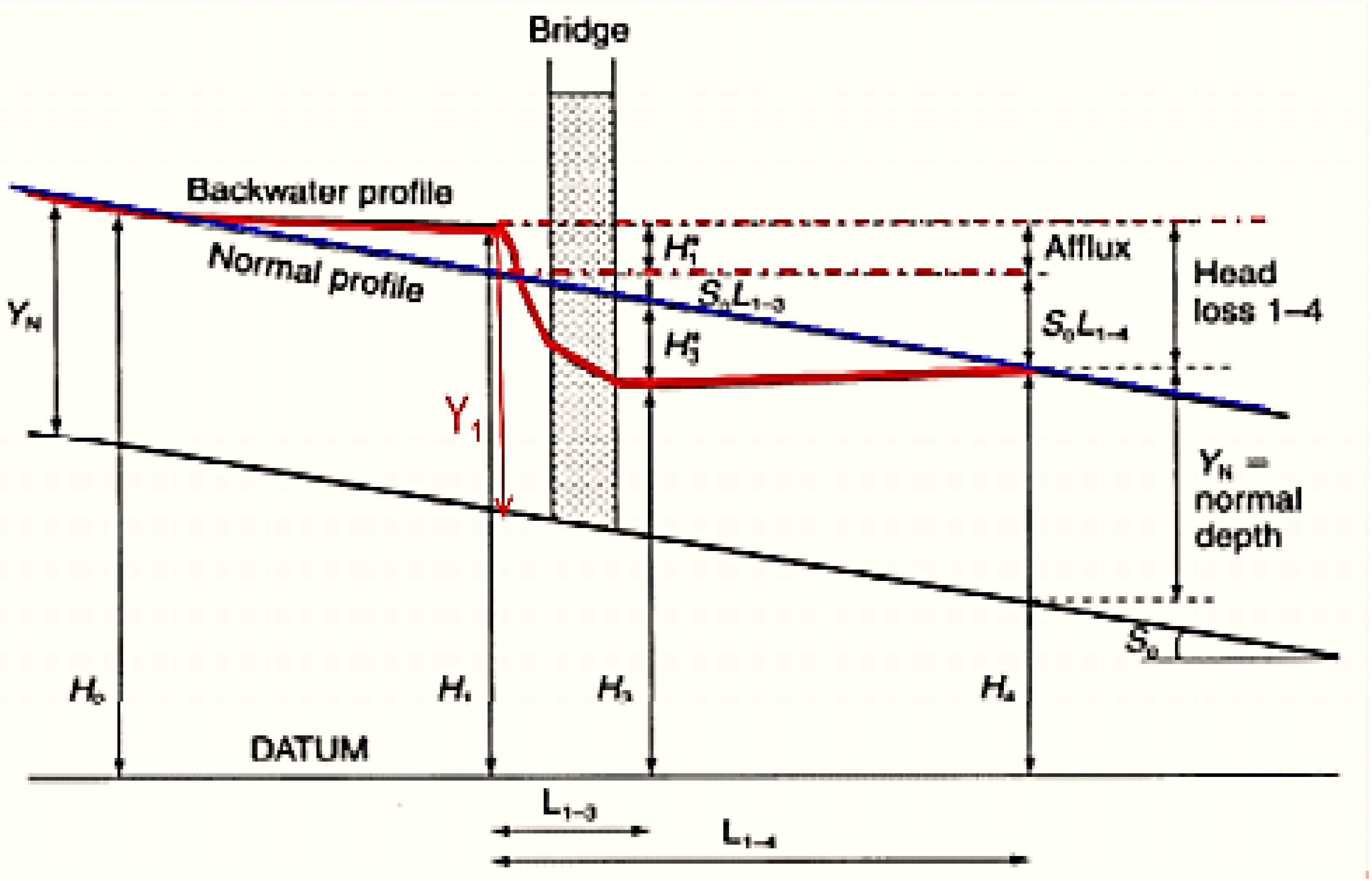

(a)
Bridge
Backwater profile
on centre line
bank
Afflux = H_1^*
Head loss
$Y_0 = Y_N$
Y_1
Y_2
Normal profile
bank
Y_3
$Y_4 = Y_N$
Section 0
1
2
3
4
(b)
Zone of separation
Zone of
drawdown
b
B
b
$C_c b$
Separation
0
1
2
3
4

- ✓ Fig.(a) longitudinal section of uniform flow at normal depth (Y_N) in a river channel with (superimposed) the surface profile arising due to provision of a bridge,
- ✓ Fig.(b) Plan view showing how the flow separates and forms a vena contracta of width $C_c b$.
- ✓ Zone of drawdown approximates a semicircle of radius “b” radiating from the Centre line of the opening at the upstream face.
- ✓ The maximum afflux is generally assumed to occur on the center line of the channel at one opening width (span) of the upstream face of the constriction. (And at this point lets say section-1).

Cont'd…

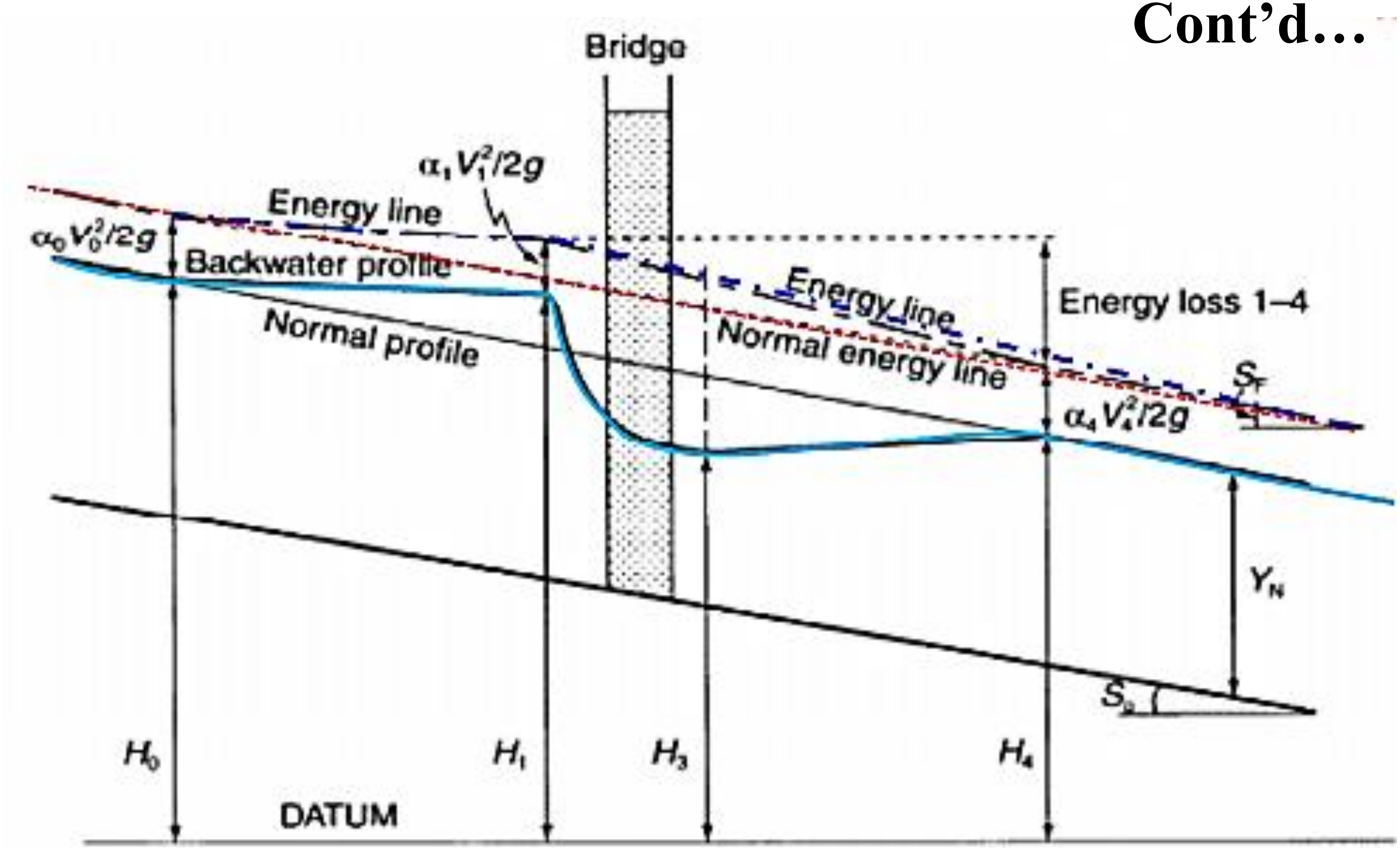
- With wide flooded valleys (of water surface width B) ***section-1*** may be better located around 0.5 (B – b) upstream of the constriction.
- After passing through ***section-1*** the water surface is drawn down and passing through normal depth at ***section-2*** at (or near) the upstream face of the bridge.
- ***Section-3*** (at minimum width and depth) may be located either in the opening or some distance downstream of it.
- ***Section-4*** is far enough downstream for normal depth conditions to have been re-established and for the flow to be unaffected by the bridge.

Cont'd…


Bridge
Backwater profile
Normal profile
H_1^*
S_0L_{1-3}
H_3^*
Afflux
S_0L_{1-4}
Head loss 1–4
Y_N
Y_1
Y_N = normal depth
S_0
H_0
H_1
H_3
H_4
DATUM
L_{1-3}
L_{1-4}

- ✓ Maximum afflux ($H_1^* = Y_1 – Y_N$) or $H_1^* = H_1 – (H_4 + S_oL_{1-4})$.
- ✓ ***Piezometer head loss*** is the difference in the elevation of the water surface between two points.
  - For example between sections 1 & 3:
    ($\Delta h = H_1^* + S_oL_{1-3} + H_3^*$).
  - head loss across the constriction is measured between section 1 and 4, i.e.
  - Head $\text{loss}_{1-4} = H_1 – H_4 = H_1^* + S_oL_{1-4}$
- ✓ ***Energy loss*** or total head loss, is the difference in the elevation of the energy line between two points,
  - ✓ Energy $\text{loss}_{1-4} = (H_1 + \alpha_1V_1^2/2g) – (H_4 + \alpha_4V_4^2/2g)$
- ✓ *Where* α is the dimensionless velocity head coefficient (kinetic energy correction factor) to allow for the non-uniform velocity.

Cont'd…



<u>*Slope variation of the energy line*</u> near a bridge & the definition of the energy loss for uniform flow at normal depth.

Cont'd…

- ✓ The *increased depth upstream* of the bridge *results* in *lower velocities* and *shallower energy* or *friction gradient* (S_F) than downstream, where the reverse happens.
- ✓ The *energy loss caused by a bridge* can be assumed to rise from three main things:
  - *Contraction of the flow* caused by the abutments, noses of the piers and when the opening is submerged the soffit or deck of the bridge (15%);
  - *Friction* between the water and the surface of the piers, abutments and when the opening is submerged, the soffit of the bridge (20%);
  - *Expansion* of the live stream downstream of the bridge (65%)

Definition of terms

- ✓ *Normal crossings*: is one with alignment at approximately 90^o to the general direction of flow during high water level.
- ✓ *Eccentric crossings*: is one where the main channel and the bridge are not in the middle of the flood plain.
- ✓ *Skewed crossings*: is one that is other than 90^o to the general direction of flow during flood stage
- ✓ *Conveyance (K)*: is a measure of the ability of a channel to transport flow. Conveyance can be expressed either in terms of *flow factors or geometric factors*.
- ✓ In bridge waterway computations, conveyance is used as a means of approximating the distribution of flow in the natural river channel upstream from a bridge.

$$K_i = \frac{Q_i}{(S_{Fi})^{1/2}} = \frac{A_i R_i^{2/3}}{n_i}$$

- ✓ *Width of Constriction, b*: b is simply the horizontal distance between abutment faces.
- ✓ In the more usual case involving spill-through abutments, where the cross-section of the constriction is irregular, it is suggested that the irregular cross-section be converted to a regular trapezoid of equivalent area.

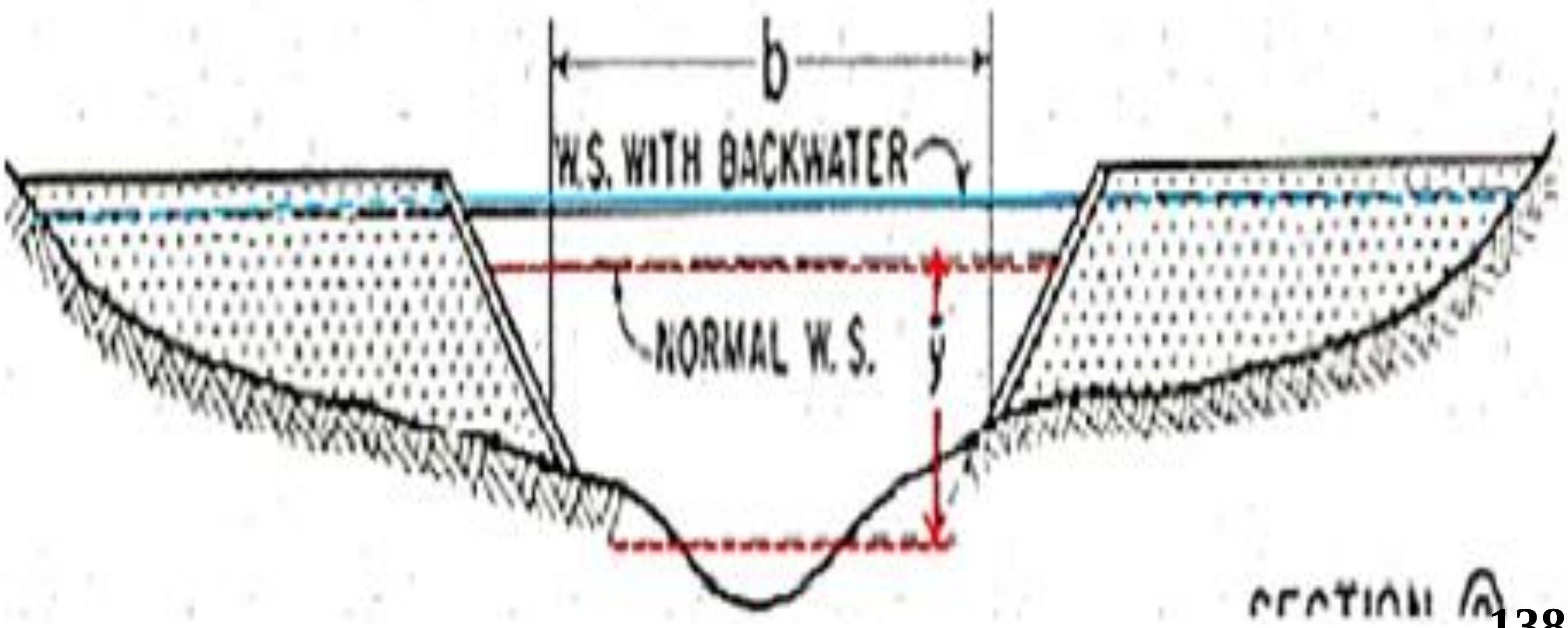
$$\boxed{b = \frac{A_{n2}}{\bar{Y}}}$$

  – A_{n2} = Gross area of flow in constriction below normal water surface at section-2.

  – $\bar{Y}$ = Mean depth of flow under bridge.

Cont'd…

- ✓ *Kinetic Energy (Velocity head) coefficient*: as the velocity distribution in the river varies from a *maximum at the deeper portion* of the channel to *essentially zero along the banks*.
- ✓ The *average velocity head* computed as **$(Q/A_1)^2/2g$** for the stream at *section-1*, does not give a true measure of the kinetic energy of the flow.



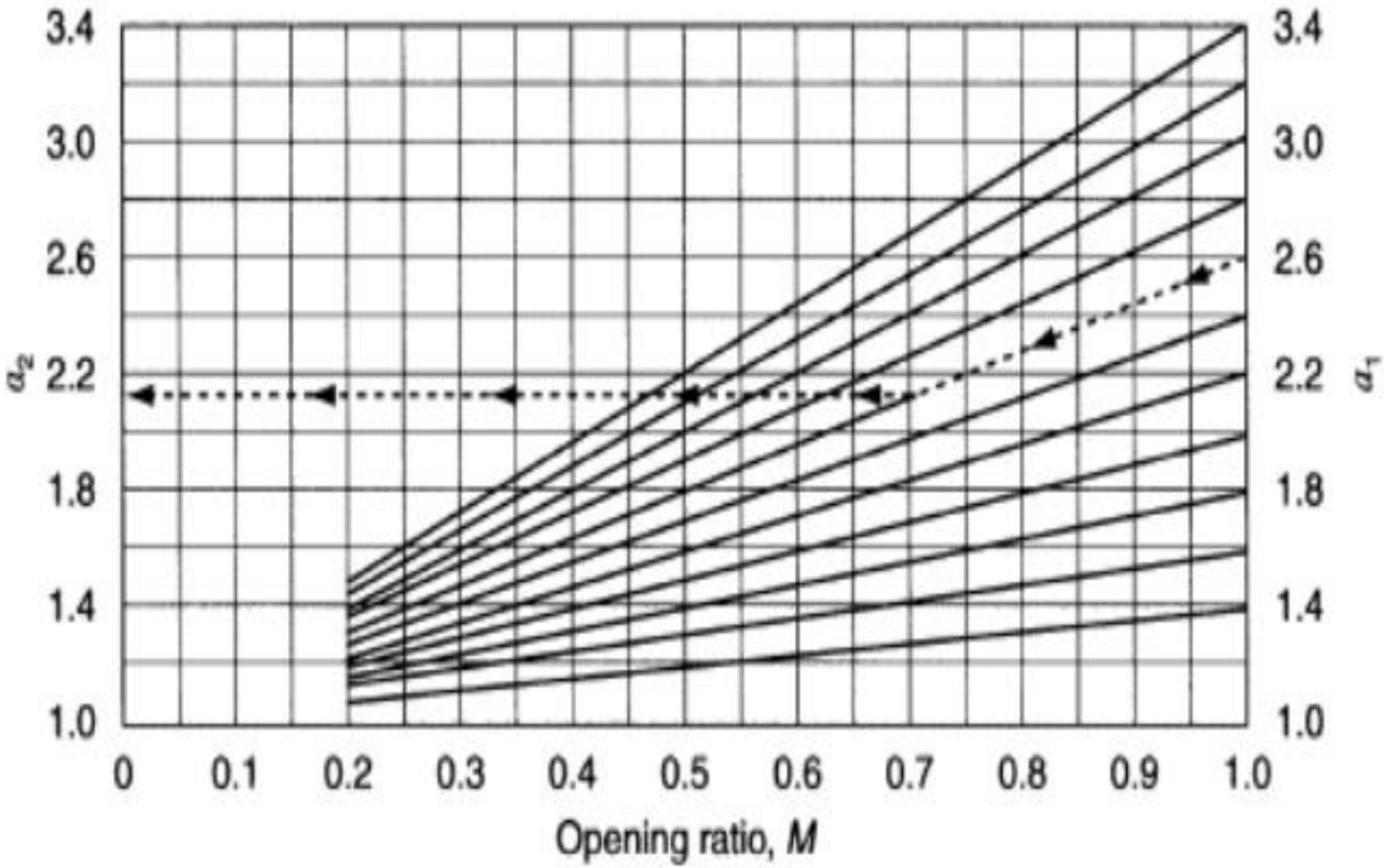
- ✓ *A weighted average* value of the kinetic energy is obtained by multiplying the average velocity head by a kinetic energy coefficient, α_1, defined as: $$\alpha_1 = \frac{\sum(Q_i V_i^2)}{Q V_1^2}$$

Where,

- V_i = Average velocity in a subsection (Q_i/A)
- Q_i = Discharge in same subsection
- Q = Total discharge in river
- V_1 = Average velocity in river section-1 or Q/A_1.

- ✓ A second coefficient, α_2, is required to correct the velocity head for non-uniform velocity distribution under the bridge. $$\alpha_2 = \frac{\sum(Q_i V_i^2)}{Q V_2^2}$$

Cont'd…



3.4
3.0
2.6
2.2
1.8
1.4
1.0
α_2
α_1
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
Opening ratio, M

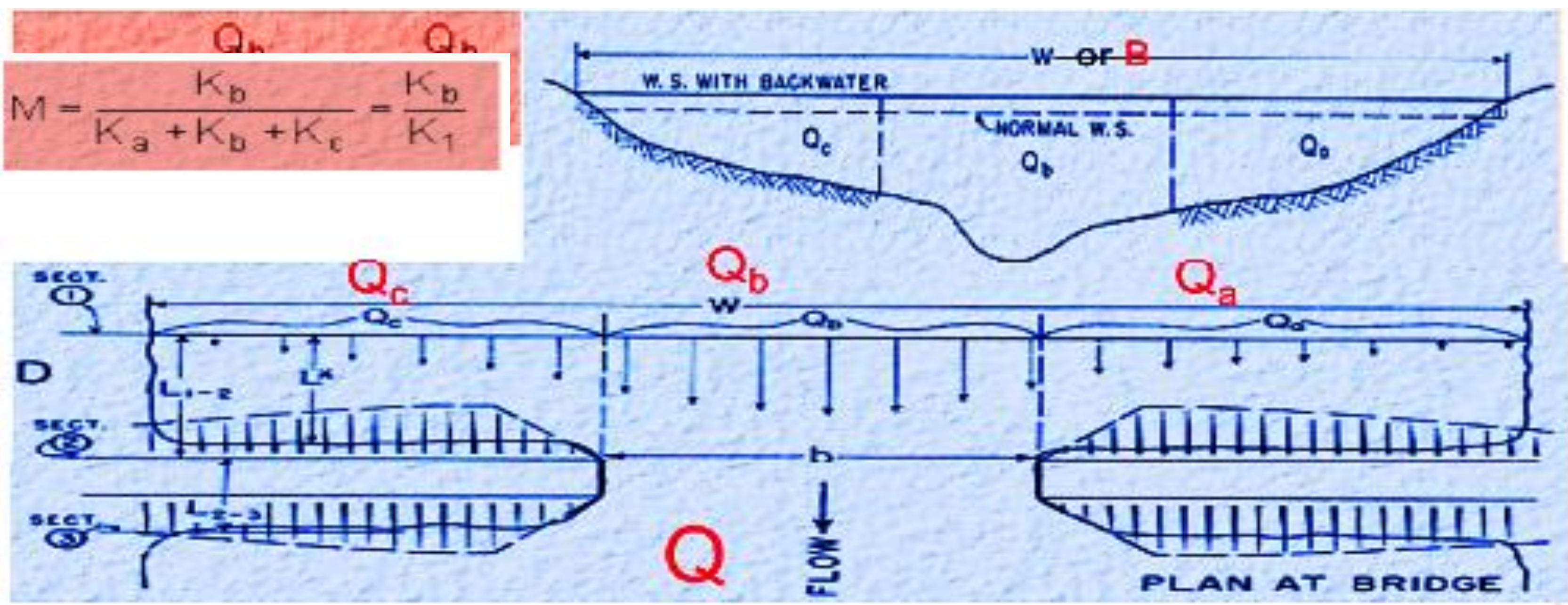
3.3. Hydraulic performance of a bridge

Factors that affect the hydraulic performance of a bridge

Bridge opening ratio (M): expressed as the *ratio of the flow which can pass unobstructed through the bridge constriction to the total flow of the river.*

✓ Since conveyance is proportional to discharge, assuming all subsections to have the same slope, M can be expressed also as:

$$M = \frac{K_b}{K_a + K_b + K_c} = \frac{K_b}{K_1}$$



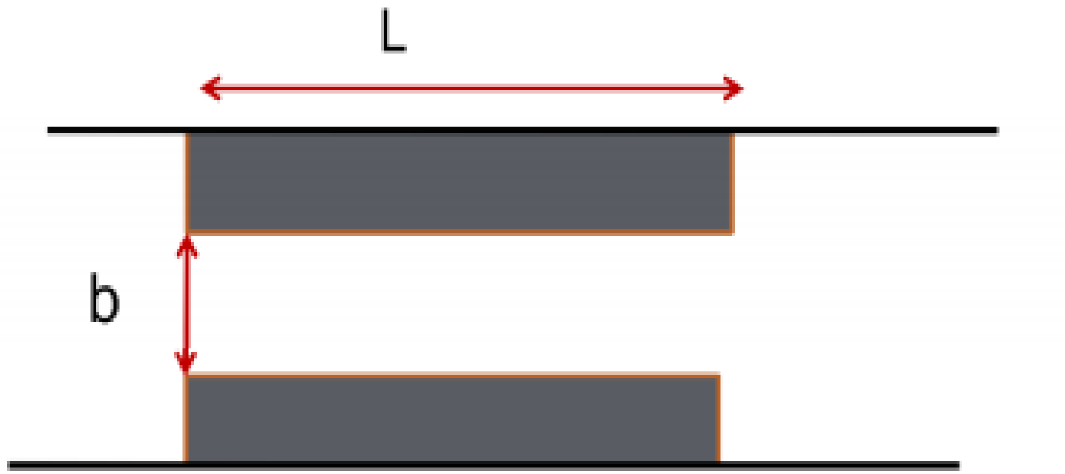
- If at section-1, it is assumed that both the normal depth (Y_N) and the associated mean velocity (V_N) are constant across the full width of the channel,
  - Q_b = Flow in portion of channel within projected length of bridge opening at section-1
  - Q_a, Q_c = Flow over that portion of the natural flood plain obstructed by the roadway embankments
  - $Q = Q_a + Q_b + Q_c$ = Total discharge
  - K_b = Conveyance of portion of channel within projected length of bridge at section-1
  - K_a, K_c = Conveyance of that portion of the natural flood plain obstructed by the roadway embankments.
  - K_1 = Total conveyance at section-1

$$M = \frac{b}{B}$$

  - B = Width of the channel, b = Width of constriction

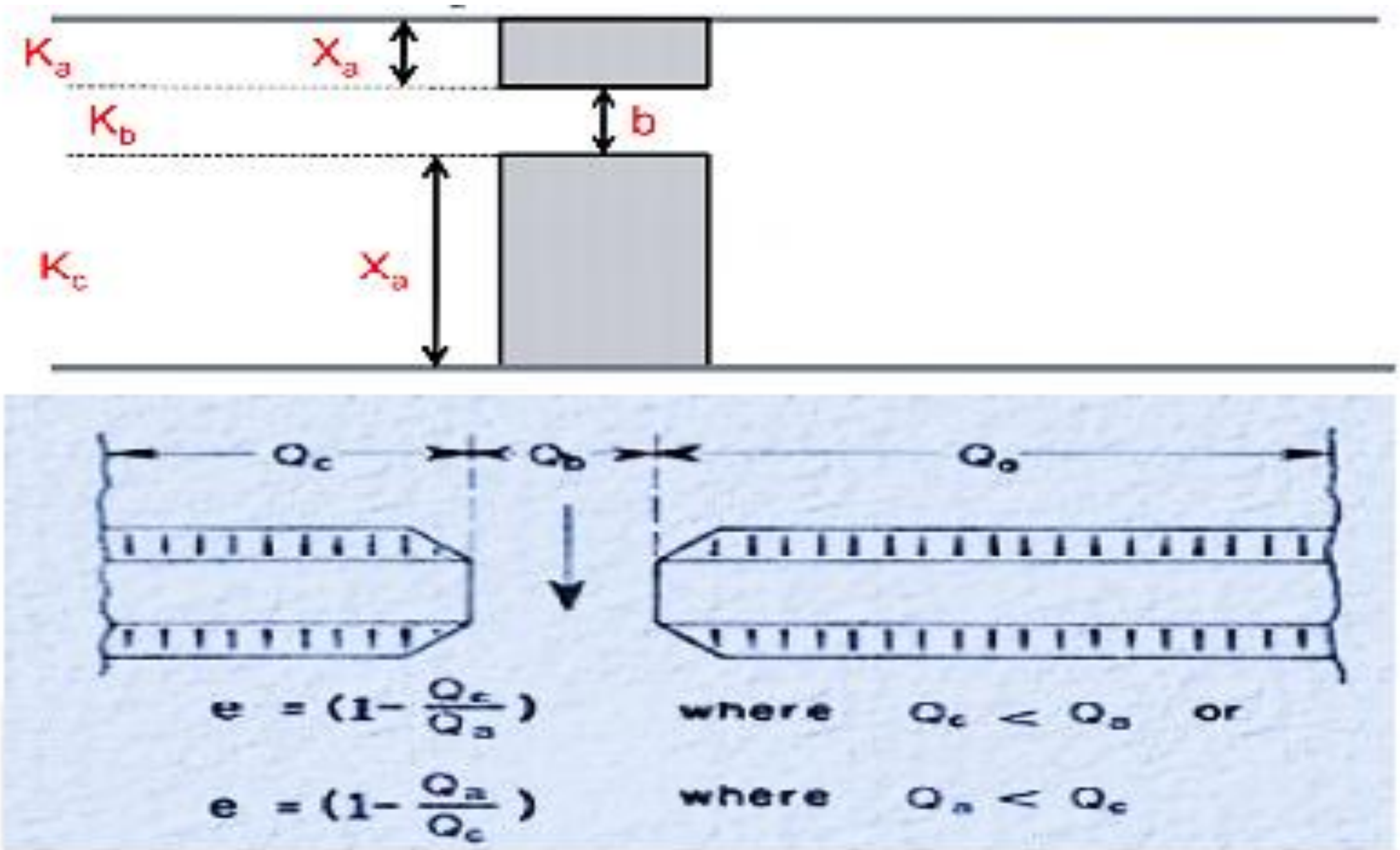
Ratio of waterway length to span, L/b

- ✓ In general, *long waterways are more efficient than short ones*, so the length of the openings is another factor that affects the hydraulic performance of a bridge.
- ✓ This is assessed in terms of the length ratio (L/b), which is the ratio of the *water way length between the upstream and downstream faces of the constriction, L, to the width or span of the openings, b*



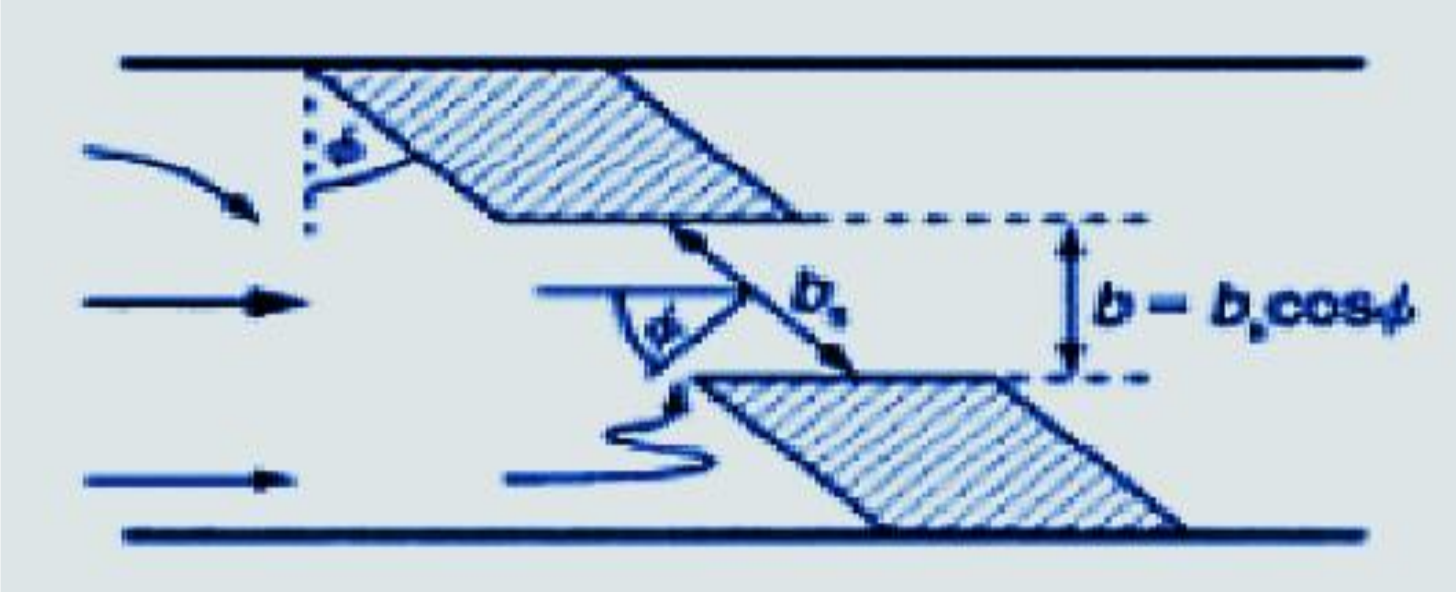
Eccentricity, (e)

- ✓ If a bridge openings is eccentricity located in the river channel, as shown in figure below this can affect the flow through the constriction.



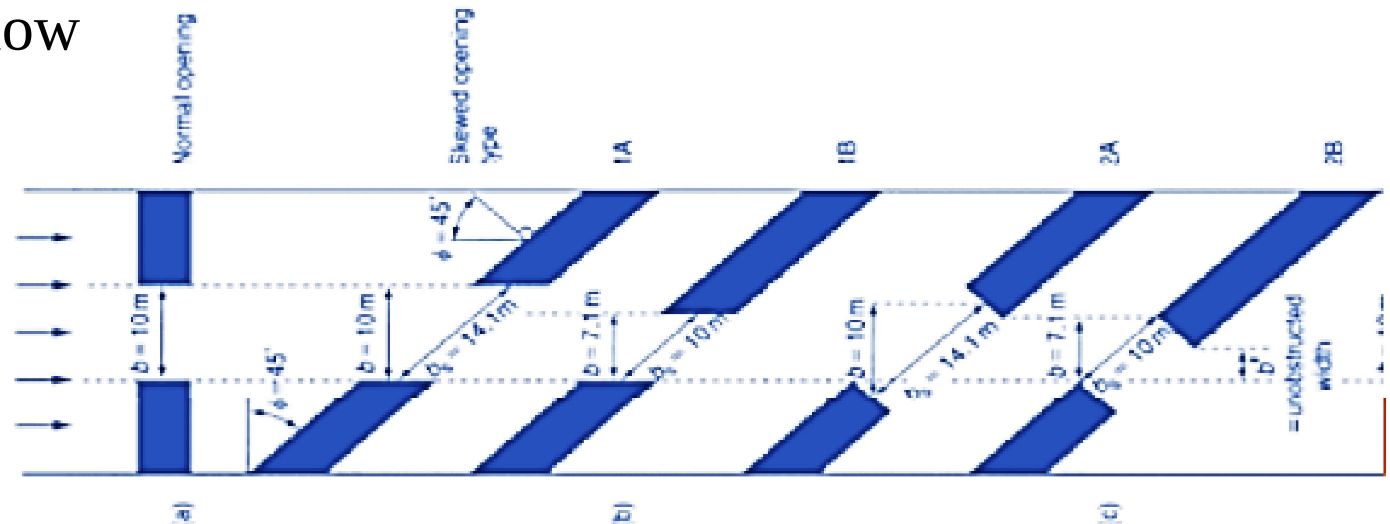
Skew, (ϕ):- A simple definition of skew is shown in figure below, in this example the longitudinal centerline of the bridge and its approach embankments is at an angle ϕ to the banks of the channel and the direction of flow, although the waterway opening itself is parallel to the flow.

✓ For a normal or perpendicular crossing $\phi = 0^o$.



✓ There are two possible types of skew in addition to a normal crossing

- *Normal crossing*:- embankments perpendicular to the flow, waterway parallel to the flow
- *Skew-1*:- embankments skewed to the flow, water parallel to the flow
- *Skew-2*:- embankments skewed to the flow, waterway skewed to the flow



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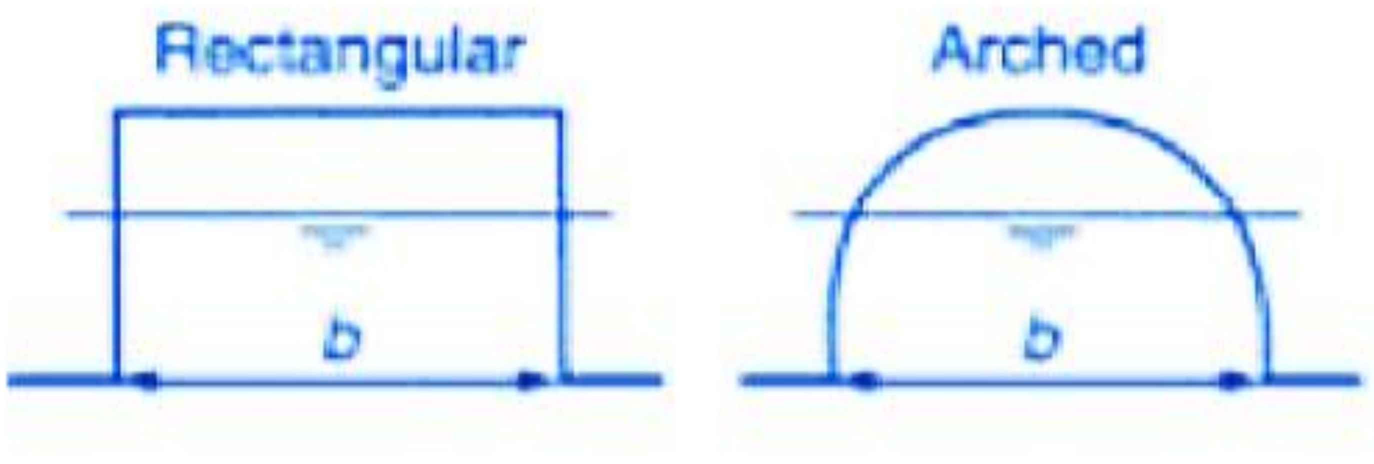
Depth of flow, Y:- For a given discharge, flow can occur over a wide range of depths depending upon the slope and geometry of the channel and the flow is uniform or non-uniform.

- ✓ The values of many variables, such as the Froude number (Fr), conveyance (K) and opening ratio (M) are functions of the depth.
- ✓ Additionally, the depth of flow relative to the height of bridge opening can influence both the type of flow that occurs at a bridge site and the hydraulic performance of the structure.

Shape of the waterway opening:- may affect the hydraulic performance of a bridge.

- ✓ For instance, a rectangular opening with a width twice its height (b = 2Z) has a 27% larger cross-sectional area than the equivalent semicircular arch.
- ✓ This means that at any given stage a rectangular opening will probably have a larger discharge and a smaller afflux than an arch of the same span.



Froude number (Fr), subcritical and supercritical flow:

- ✓ In open channel flow Fr also affects the discharge through the bridge opening.
- ✓ Flow at the critical depth (Fr = 1.0) can be used to *optimize the performance of a waterway, so a knowledge of the critical contraction* that will cause this condition is important.
- ✓ *A waterway narrower than the critical contraction may result in an unexpectedly large afflux* due to the phenomenon known as *choking*.
- ✓ Yarnell (1934) and Chow (1981) (F_3=1.0)

$$F = \frac{V}{(gY)^{1/2}} \qquad M_L^2 = \frac{27F_1^2}{(2+F_1^2)^3} \qquad M_L^2 = \frac{27\varepsilon^3 F_4^2}{(2+F_4^2)^3}$$

Where,

- M_L - Limiting opening ratio (if a wide waterway opening is made progressively narrower, there will be a limiting width at which the flow in the opening will no longer be subcritical but critical, which is called the *limiting or critical contraction*).
- ε – is equated to the energy at *section-3* between the piers to that at *section-4*, to represent the proportion of energy recovered.

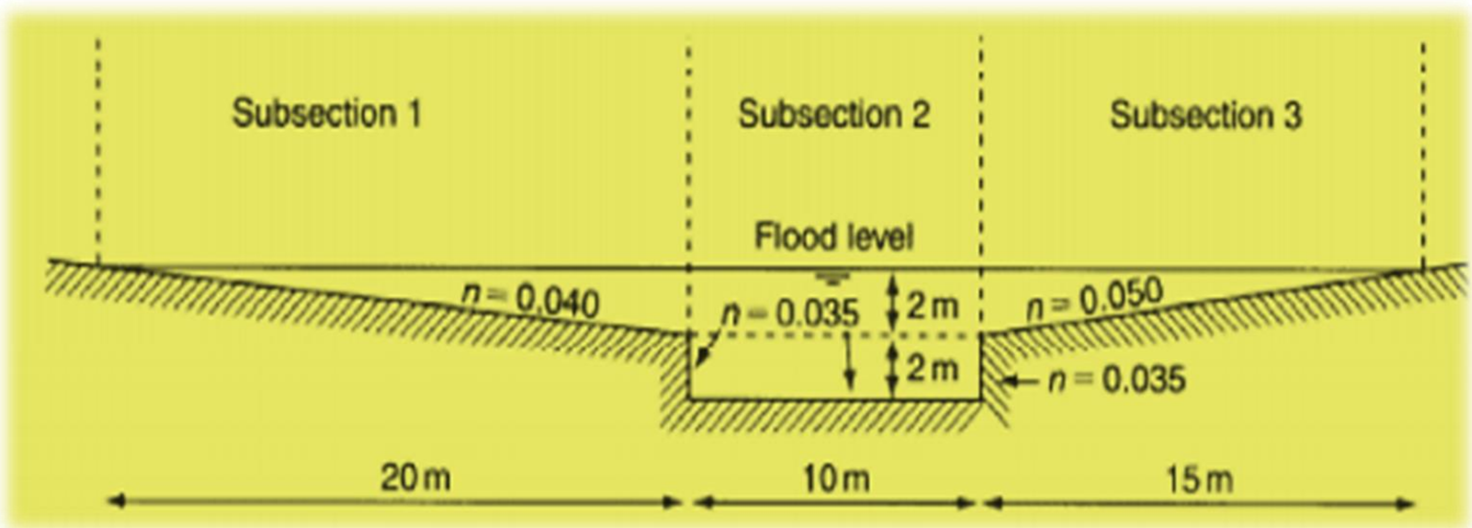
$$\varepsilon\left(Y_3+\frac{V_3^2}{2g}\right)=\left(Y_4+\frac{V_4^2}{2g}\right)$$

- ✓ *The Froude number* is important to indicate *where a backwater analysis should begin.*
- ✓ If $(gY)^{1/2} > V$ (subcritical flow through a bridge) the upstream reach is in hydraulic communication with the downstream reach and the *control is downstream* of the constriction and this would be the starting point for a backwater analysis.
- ✓ If $V > (gY)^{1/2}$ (supercritical flow through a bridge) the upstream reach is not in hydraulic communication with the downstream reach and the *control is upstream* so the calculations for the backwater analysis proceed in a downstream direction.

Entrance Rounding: entrance rounding reduces the contraction of the live stream and increases the width of the vena-contracta and hence increases the coefficient of discharge, such as C_d.

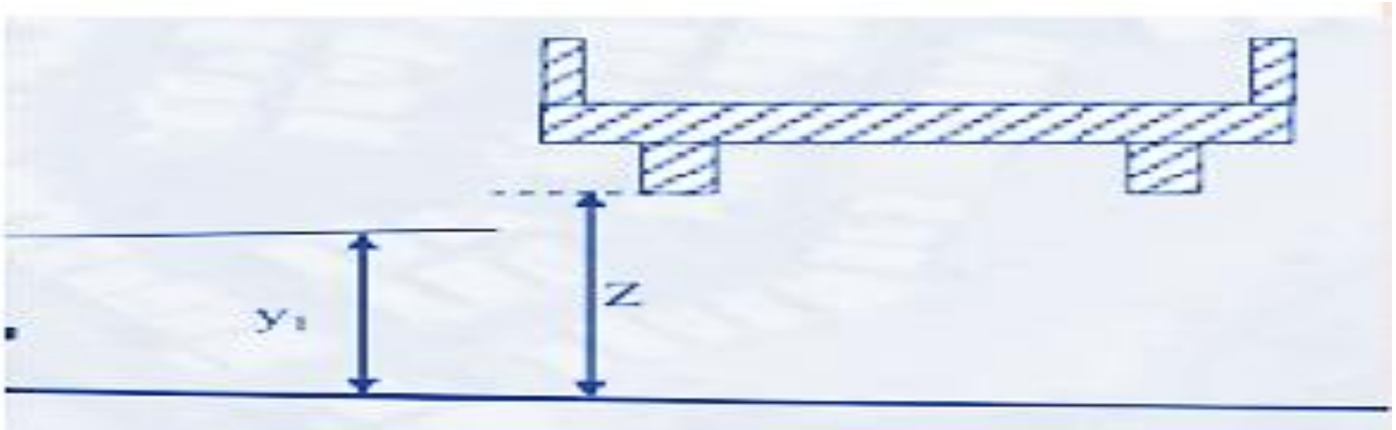
Cont'd…

Example 3.1 (L Hamill on page111): Abridge has a single rectangular opening 10m wide, which is the same width as the main river channel at low stages (Fig. below). However during flood the bridge obstructs the flow over the floodplains. The dimensions and Manning roughness coefficients are shown in the diagram. Assuming uniform flow, that the longitudinal slope of the channel and floodplains is 1 in 1000, and a depth of 4m in the main channel, estimate the following: (a) the conveyance of the upstream cross-section, K; (b) the velocity distribution coefficient, α; (c) the bridge opening ratio, M.

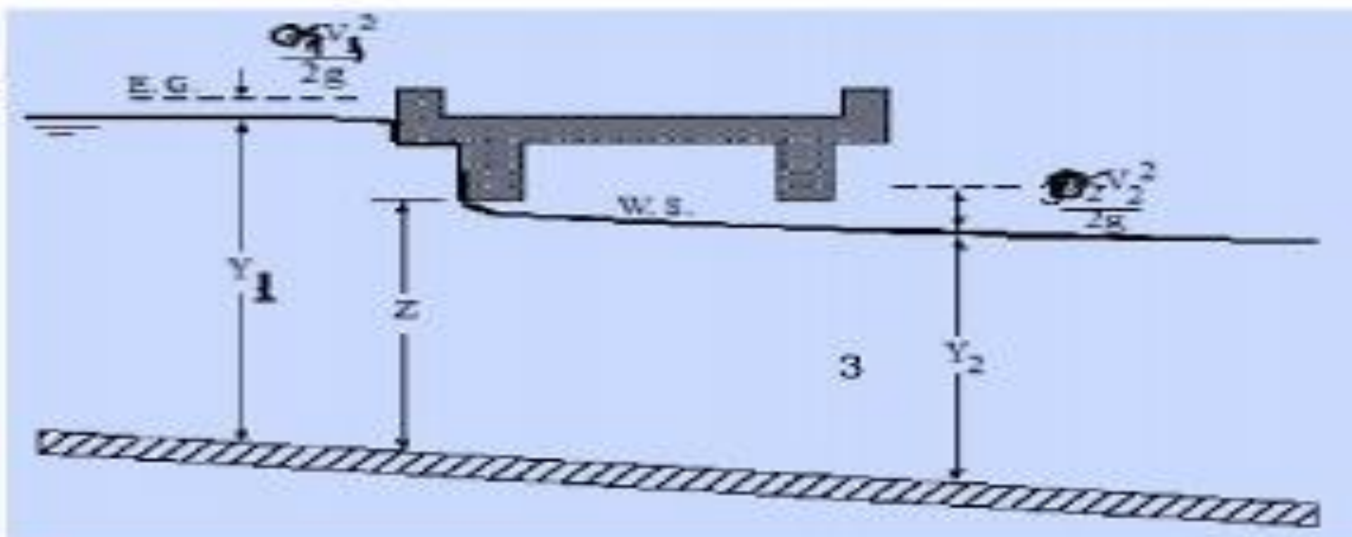


3.4.Types of flow under bridge waterway

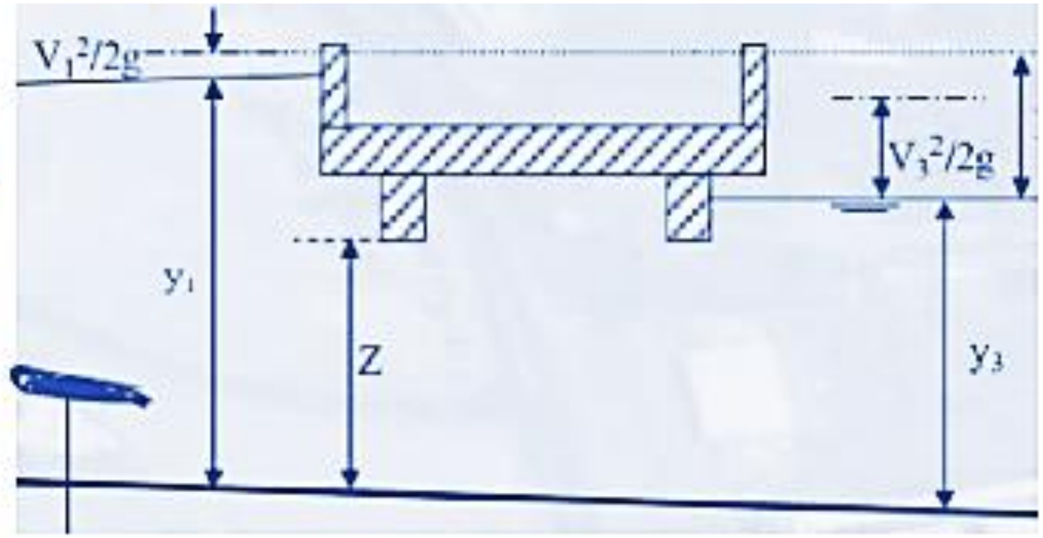
A. Low flow case (waterway opening is not submerged) ***(Type I, II or III)***



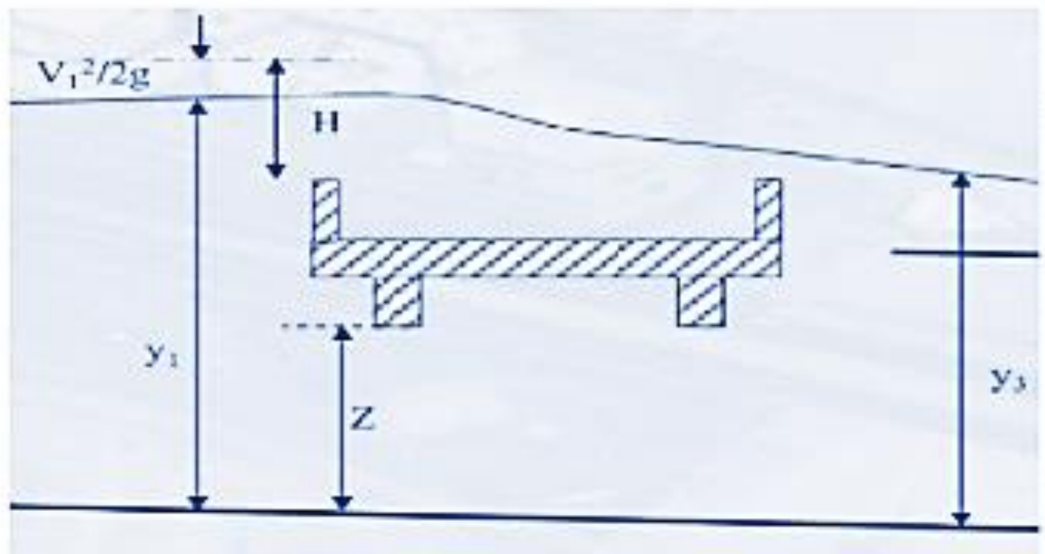
B. High flow case (waterway opening is submerged) ***Orifice flow***



Pressure flow case.



Weir flow case.



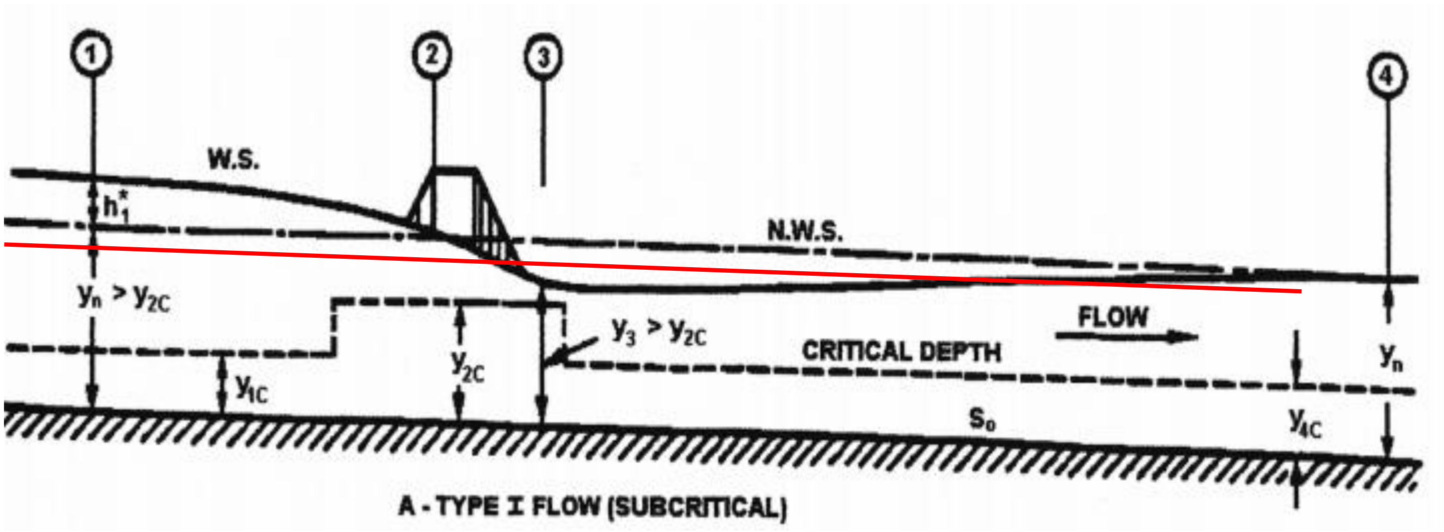
Types of flow in bridge waterway in low flow case

There are three types of flow in bridge waterway at flow low case.

Type I Flow (sub-critical flow)

- *Normal water surface* is everywhere *above critical depth*
- *Actual water surface* is everywhere *above critical depth*
- This type *usually* encountered in practice

Backwater expression for *type-I flow* is obtained by applying the *conservation of energy principle between sections*-1 *and 4.*



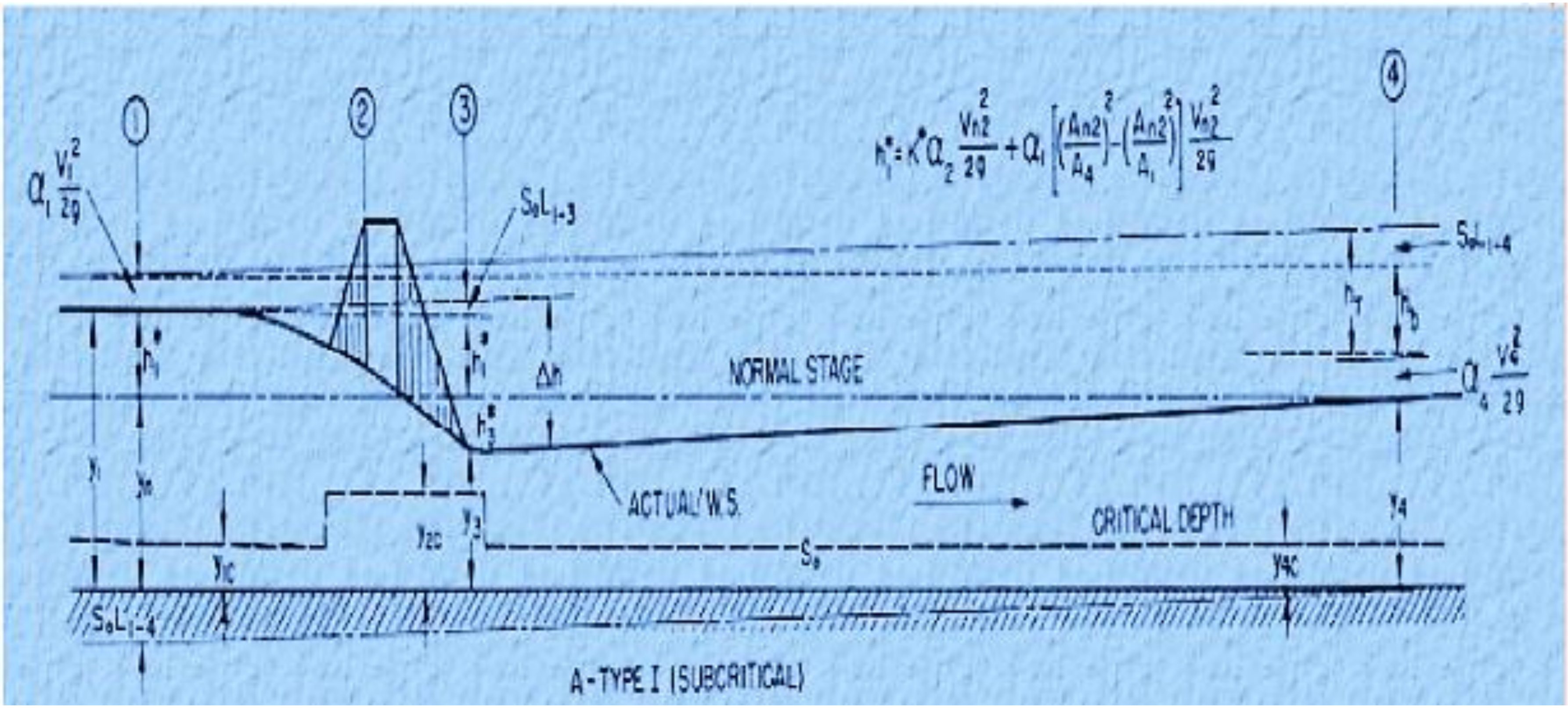
$$S_0 L_{1-4} + y_1 + \frac{\alpha_1 V_1^2}{2g} = y_4 + \frac{\alpha_4 V_4^2}{2g} + h_T \quad \text{............(eq.\#1)}$$

Where, h_T is the total energy loss between sections-1 and 4.

✓ *This method was developed on the basis that the*:

- Channel in the *vicinity of the bridge is essentially straight*.
- Cross-sectional *area of the stream is reasonably uniform* and *the same at sections-1 and 4*.
- *Gradient of the bottom is constant* between sections-1 & 4.
- Applies *only to steady subcritical flow*
- There is *no appreciable scour of the bed in the constriction*
- The *flow is free to contract and expand*.
- Energy loss caused *by constriction* ($h_b = h_T - S_o L_{1-4}$)

Cont'd…



Expression equation #1 can be written as:

$$y_1 - y_4 = \frac{\alpha_4 V_4^{\,2}}{2g} - \frac{\alpha_1 V_1^{\,2}}{2g} + h_b \quad \ldots\ldots\ldots\ldots\ldots(\text{eq.\#2})$$

- ✓ ***Energy loss*** caused by constriction (h_b) also can be expressed as the product of a loss coefficient, K*, and a velocity head.

$$h_b = K^* \frac{\alpha_2 V^2_{n2}}{2g} \quad \text{............(eq.\#3)}$$

Where: V_{n2} - is average velocity *in the contracted section* based on the flow area below NWS.

- ✓ Replacing ($y_1 - y_4$) with h_1* and h_b with $K^*\alpha_2 V^2_{n2}/2g$, equation #2 becomes:

$$h^*_1 = K^* \frac{\alpha_2 V^2_{n2}}{2g} + \left[\frac{\alpha_4 V_4^2}{2g} - \frac{\alpha_1 V_1^2}{2g}\right] \quad \text{............(eq.\#4)}$$

- ✓ Since the analysis is based on the assumption that the cross *sectional areas at sections-1 and 4 are essentially the same*, α_4 can be replaced by α_1.
- ✓ Also from the equation of continuity $A_1V_1 = A_4V_4 = A_{n2}V_{n2}$, velocities can be expressed as areas. So the expression for backwater becomes:

Cont'd…

$$h_1^* = K^* \alpha_2 \frac{V_{n2}^2}{2g} + \alpha_1 \left[\left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \frac{V_{n2}^2}{2g} \quad \text{............(eq.\#5)}$$

Where,

- $h_1^* = H_1^*$ = total backwater (m)
- K^* = total backwater head loss coefficient
- α_1= velocity head correction coefficient at sections-1 and 4
- α_2= velocity head correction coefficient at constriction
- A_{n2} = gross water area in constriction measured below normal stage (m^2)
- V_{n2} = average velocity in constriction for flow at normal stage or Q/A_{n2} (m/s)
- A_4 = water area at section-4
- A_1 = total water area at section 1, including backwater (m^2)

✓ If piers are present in the constriction, these are ignored in the determination of A_{n2}. (including the area occupied by any piers)

Cont'd…

- ✓ The expression represents the difference in kinetic energy between sections-4 and 1.

$$\alpha_1\left[\left(\frac{A_{n2}}{A_4}\right)^2-\left(\frac{A_{n2}}{A_1}\right)^2\right]\frac{V^2_{n2}}{2g}$$

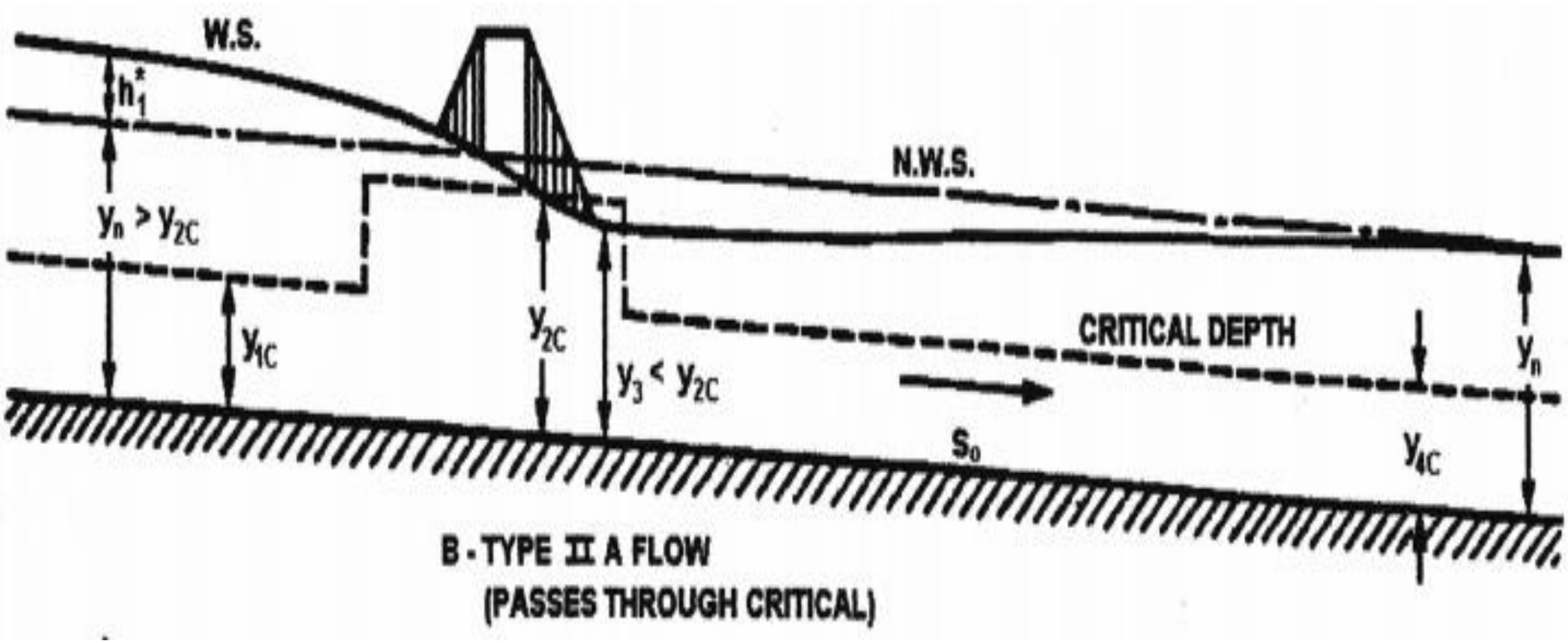
- ✓ The backwater coefficient, K* can be:

$$K^*=\frac{h_1^*}{\alpha_2 V^2_{n2}/2g}-\frac{\alpha_1}{\alpha_2}\left[\left(\frac{A_{n2}}{A_4}\right)^2-\left(\frac{A_{n2}}{A_1}\right)^2\right] \quad \text{............(eq.\#6)}$$

Type-II flow (water surface passes through critical depth)

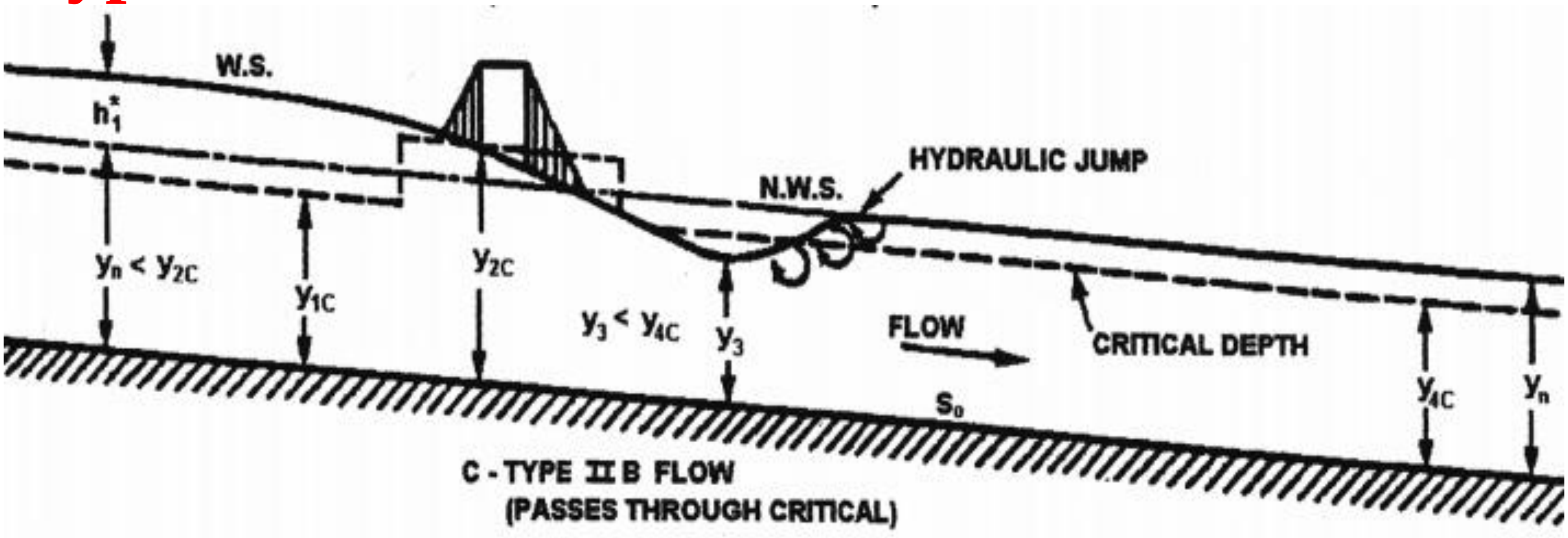
- ✓ There are at least two variations of type II flow which will be described here under types IIA and IIB.

Type-IIA Flow:



- *Normal water surface* is everywhere *above critical depth.*
- *Actual water surface passes* through *critical depth in the constriction.*

Type-IIB Flow



- *Normal water surface* is *above critical depth* at the upstream and at the downstream (section-4).
- Both *actual water surface and normal water surface* are *below critical depth* at the constriction and passes through critical depth in the constriction.

- There is *poor hydraulic jump before it returns to normal depth.*
- *Once critical depth is reached, the water surface upstream from* the constriction is no longer influenced by conditions downstream.
- This is true even though the water surface may dip below critical depth, Y_{2c}, in the constriction and then return to subcritical flow as in Type-IIA.

- Type-IIB flow is similar *except the water surface not only dips below Y_{2c} but also Y_{4c} downstream from the constriction.*
- Both types of flow are subject to the same analysis since the criterion here is that the flow passes through critical depth.
- *A backwater expression* applicable to both types-IIA and IIB flow has been developed by equating the *total energy surface between section-1 and the point at which the water surface passes through critical stage* in the constriction.

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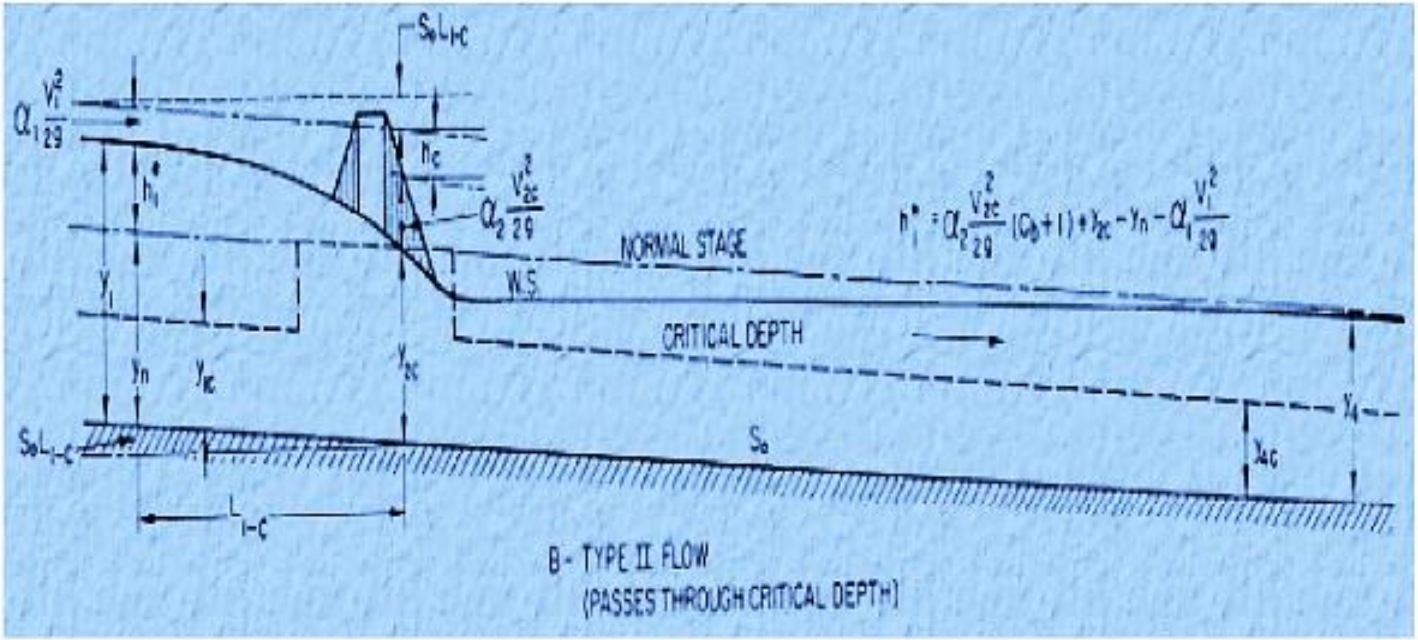


$$S_0L_{1-c} + y_n + h_1^* + \alpha_1 \frac{V_1^2}{2g} = Y_{2c} + \alpha_2 \frac{V^2_{2c}}{2g} + h_T \quad \text{............(eq.\#7)}$$

- *<u>Energy loss</u>* caused by constriction (h_c) can be expressed as

$h_c = h_T - S_oL_{1-c} = c_b\alpha_2(V^2_{2c})/(2g)$

$$y_n + h_1^* - Y_{2c} = \alpha_2 \frac{V^2_{2c}}{2g} + C_b\alpha_2 \frac{V^2_{2c}}{2g} - \frac{\alpha_1 V_1^2}{2g} \quad \text{............(eq.\#8)}$$

✓ Solving for backwater:

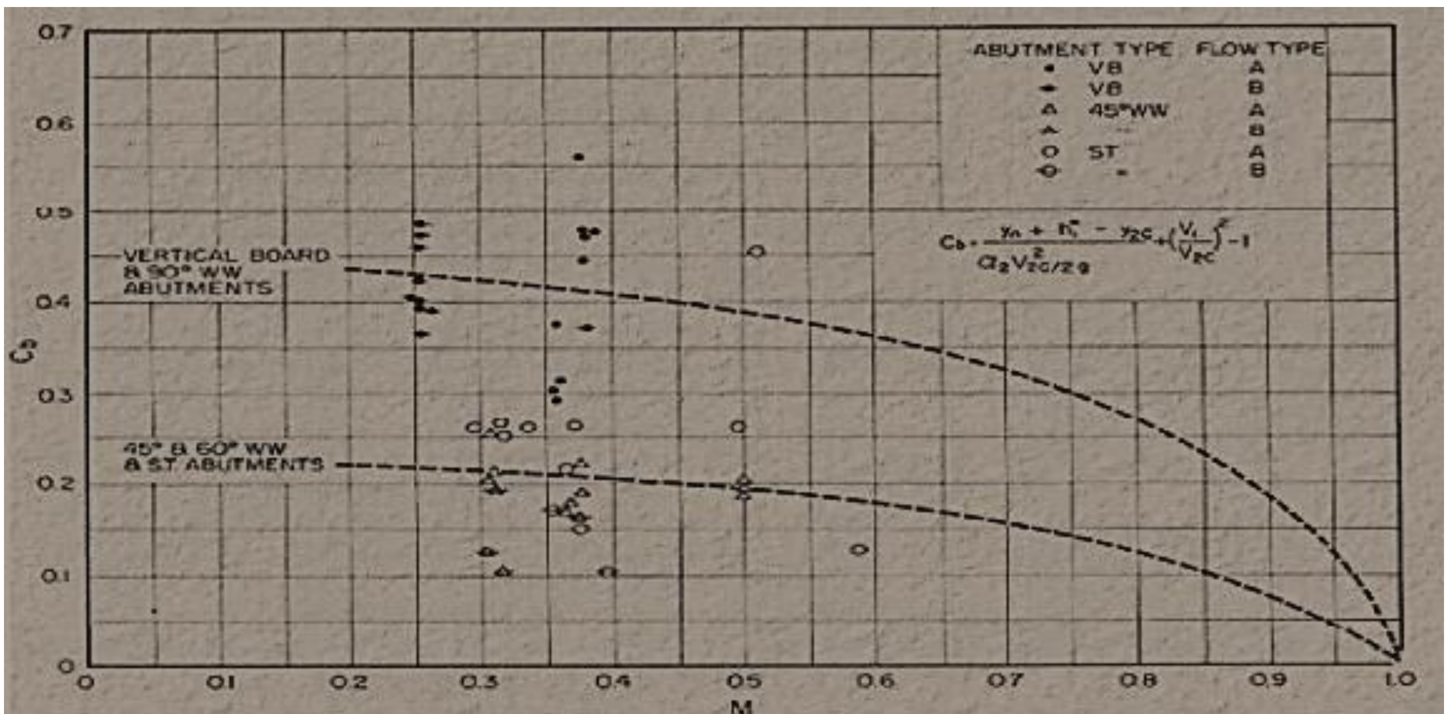
$$h_1^* = \frac{\alpha_2 V_{2c}^2}{2g}(C_b + 1) + Y_{2c} - y_n - \frac{\alpha_1 V_1^2}{2g}, \quad \ldots\ldots\ldots\ldots(\text{eq.\#9})$$

Where,

- h_1^*= total bridge backwater (m)
- Y_n = normal flow depth (m) (model)
- Y = normal flow depth or $[A_{n2}/b]$ (m) (prototype)
- Y_{2c} = critical depth in constriction or $[(Q^2/(b^2g))^{1/3}]$ (m)
- V_{2c} = critical velocity in constriction or $[Q/(Y_{2c}*b)]$ (m/s)
- V_1 = velocity at section 1 or $[Q/A_1]$ (m/s)
- α_1, α_2 = velocity head correction coefficients at section-1 and in the constriction respectively
- C_b = backwater coefficient for type-II flow (constriction loss only) should the backwater coefficient be desired

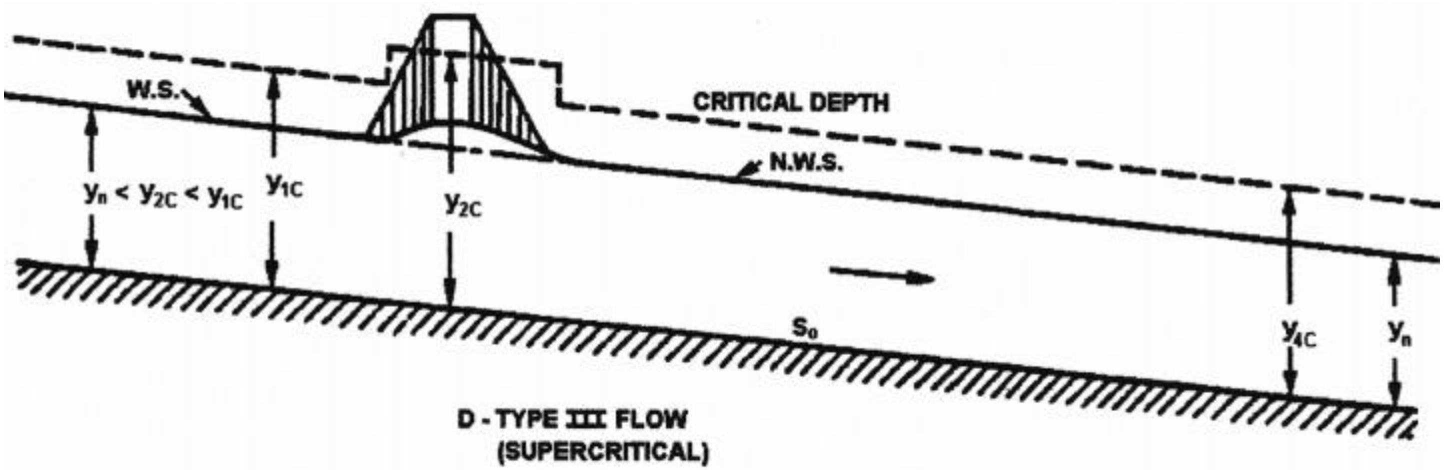
Cont'd…

$$C_b = \frac{h_1^* + y_n - Y_{2c}}{\alpha_2 V^2_{2c}/2g} + \frac{\alpha_1}{\alpha_2}\left(\frac{V_1}{V_{2c}}\right)^2 - 1 \quad \text{……………(eq.\#10)}$$



Backwater coefficient curve for type II flow.

Type-III Flow



- In the type-III flow, figure above the *normal water surface is everywhere below critical depth and flow throughout is supercritical.*
- *This is an rare case requiring a steep gradient but such conditions do exist, particularly in mountainous regions.*
- Theoretically *backwater should not occur for this type, since the flow throughout is supercritical.*

3.5. Hydraulic design of bridge

Computation of Backwater

- ✓ An expression for backwater has been formulated by applying the <u>*principle of conservation of energy between the point of maximum backwater in upstream (section-1)*</u> and a <u>*point downstream from the bridge at which normal stage has been re-established (section-4)*</u>.
- ✓ The backwater at upstream from a bridge constricting the flow is:

$$h_1^* = K^* \alpha_2 \frac{V_{n2}^2}{2g} + \alpha_1 \left[\left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \frac{V_{n2}^2}{2g} \quad \text{............(eq.\#11)}$$

- ✓ To compute backwater, it is necessary to obtain the approximate value of h_1^* by using the first part of the expression (eqn. #11).

Cont'd…

$$h_1^* - K^* \alpha_2 \frac{V^2_{n2}}{2g}$$

- ✓ The value of A_1 is the second part of expression (eqn. #11), which depends on h_1*, can then be determined and the second term of the expression evaluated:

$$\alpha_1 \left[\left(\frac{A_{n2}}{A_4} \right)^2 - \left(\frac{A_{n2}}{A_1} \right)^2 \right] \frac{V^2_{n2}}{2g}$$

Backwater Coefficient (K* and K_b)

- ✓ K_b - is the backwater coefficient for a bridge in which *only the bridge opening ratio, M, is considered*.
- ✓ This is known as a *base coefficient* as shown in the Figure below from *base curves.*

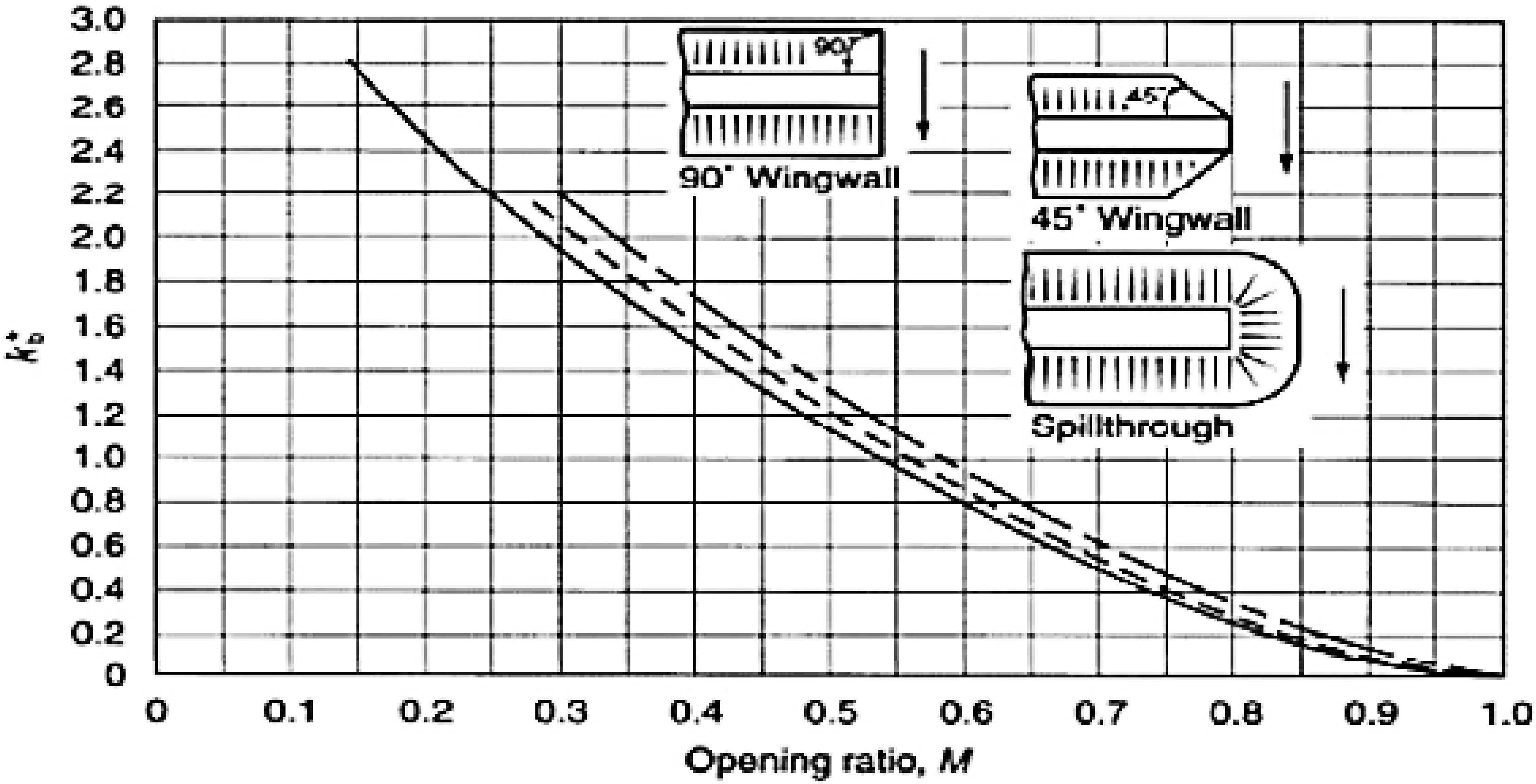


Figure Backwater coefficient base curves (subcritical flow)

- ***Top curve:-*** 90^o wing-wall abutments, b < 60m (high energy loss)
- ***Middle curve:-*** 30^o wing-wall abutments with b < 60m
- ***Lower curve:-*** all 45^o and 60^o wing-wall abutments (low energy loss).

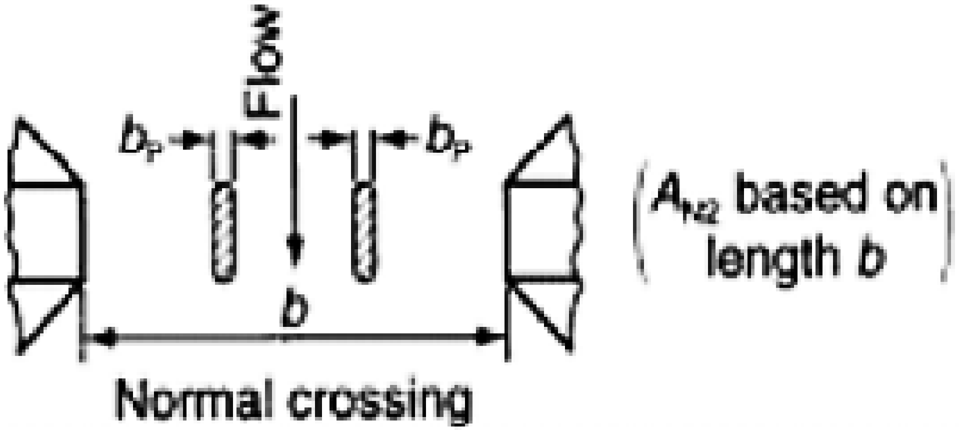
- ✓ The value of the *overall backwater coefficient, K*,* is similarly *dependent on the value of M* but also affected by:
  a) Number, size, shape and orientation of piers in the constriction
  b) Eccentricity or asymmetric position of bridge with respect to the valley cross section
  c) Skew (bridge crosses stream at other than 90^o angle).
- ✓ K* consists of a base curves coefficient, K_b, *coefficients* to account for the *effect of piers, eccentricity and skew.*

Effect of Piers (Normal Crossings), ΔK_p

- ✓ Backwater *caused by introduction of piers in a bridge constriction* has been treated as an incremental backwater coefficient (ΔK_p), *which is added to the base curve coefficient K_b* when piers are present in the waterway.
- ✓ ΔK_p is depends on:
  - The ratio that the *area of the piers* bears to the *gross area of the bridge opening* ($J = A_p/A_{n2}$)
  - The type of piers
  - The value of the bridge opening ratio (M)
  - The angularity of the piers with the direction of flood flow.

Cont'd…

- A_p, A_{n2} both based on the normal water surface.



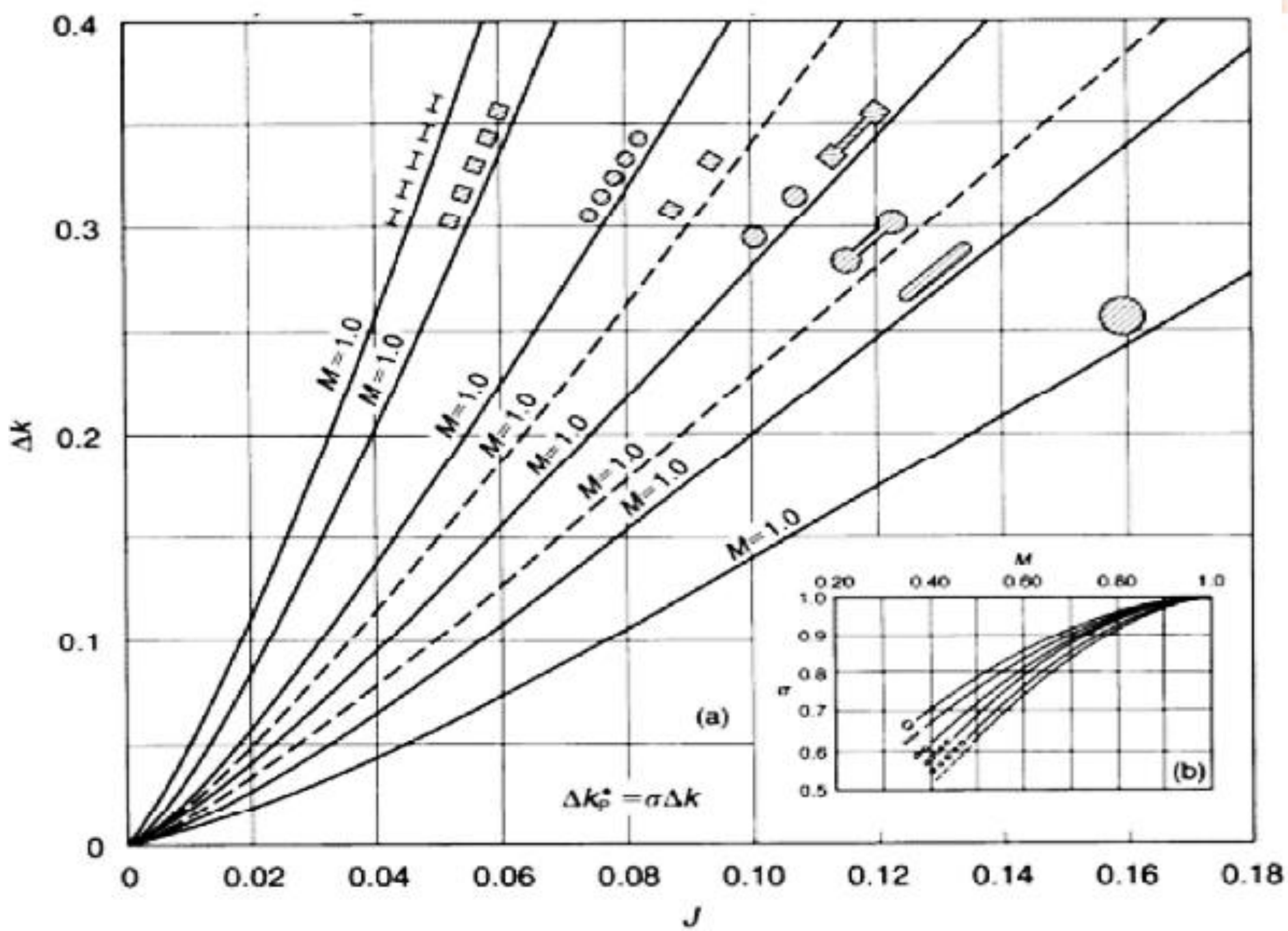
i.e. $A_{n2} = b*Y_{n2}$,

$$A_p = \sum (b_p * h_{n2}),$$

- A_p = total projected area of piers normal to flow, h_{n2} = height pier exposed to flow, b_p = width of pier normal to flow.

$$\boxed{\Delta K_p = \sigma \Delta K}$$

Cont'd…

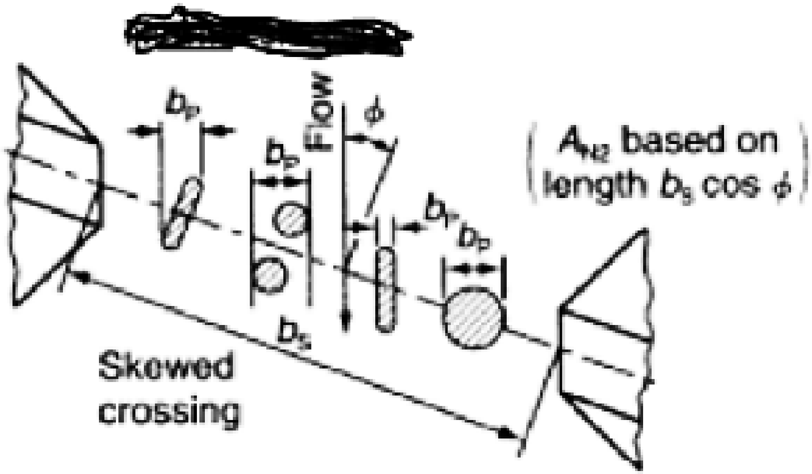


- For a normal crossing with piers, the total backwater coefficient becomes:

$$K^* = K_b + \Delta K_p$$

Effects of piers (skewed crossings)

- ✓ In the case of skewed crossings, the effect of piers is treated as explained for normal crossings except for the computation of J, A_{n2} and M.
- ✓ The pier area for a skewed crossing, A_p, is the sum of the individual pier areas normal to the general direction of flow.
- ✓ The area of the constriction, A_{n2}, for skewed crossings is based on the projected length of bridge, $b_s \cos\phi$.
- ✓ A_{n2} is a gross value and includes the area occupied by piers.
- ✓ The computation of M for skewed crossings is also based on the projected length of bridge.



$A_p = \Sigma^n b_p h_{N2}$ = total projected area of piers normal to flow

A_{N2} = Gross water cross-section in constriction based on normal water surface (Use projected bridge length normal to flow for skew crossings)

$$J = \frac{A_p}{A_{N2}}$$

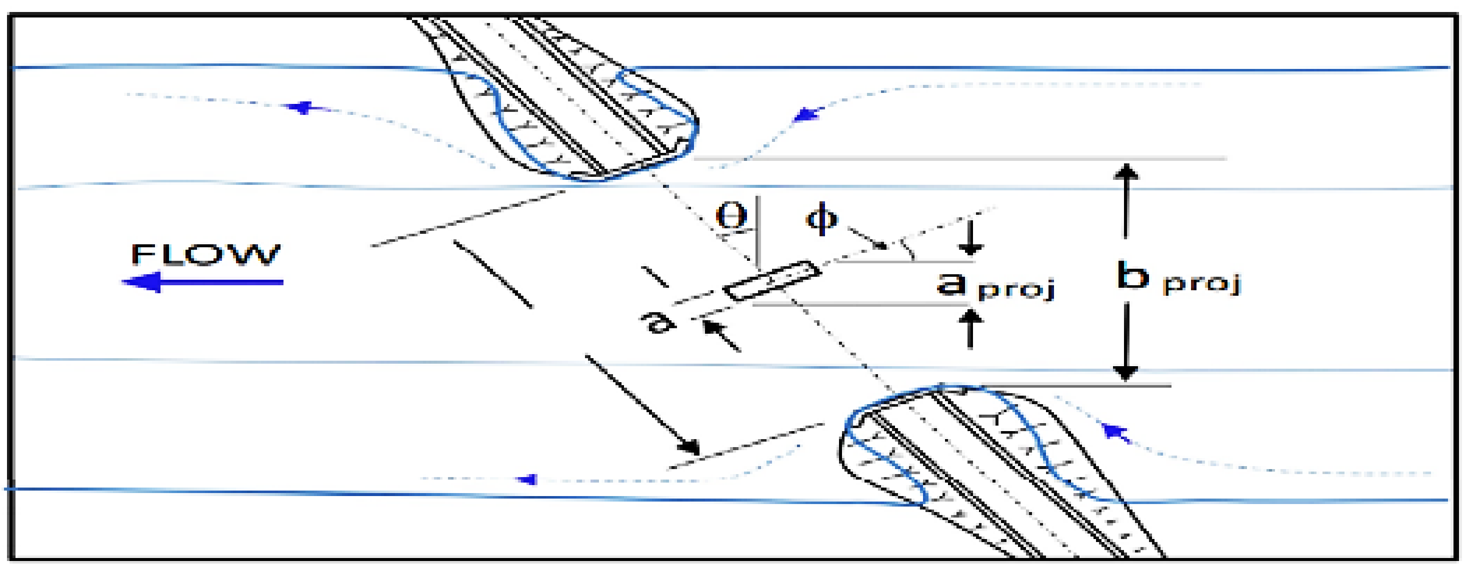
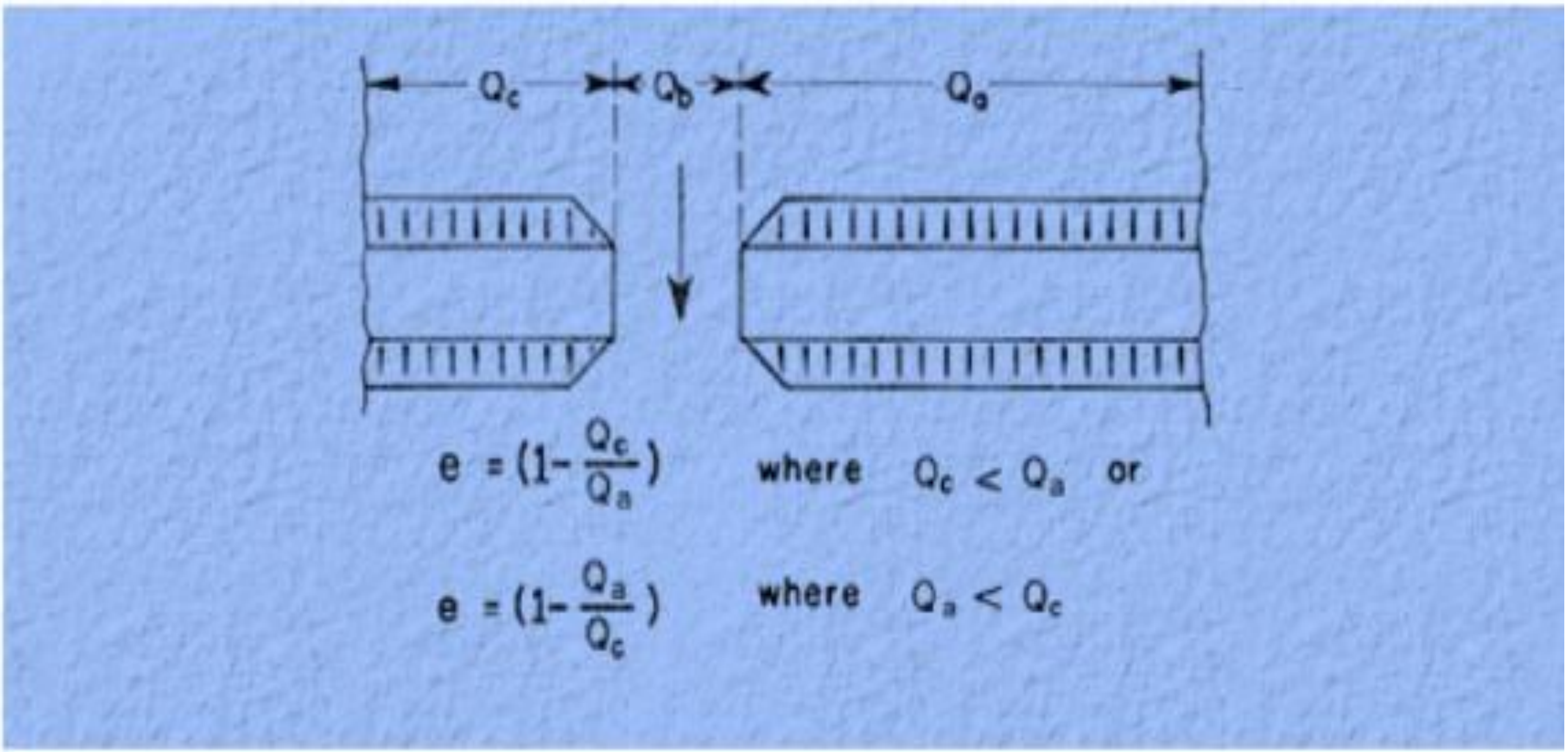


Figure illustration of a skewed bridge crossing

Effect of eccentricity (ΔK_e)

- ✓ For an extremely eccentric crossing with wing-wall or spill-through abutments and piers will be:

$$K^* = K_b + \Delta K_p + \Delta K_e$$



Effect of eccentricity (ΔK_e)

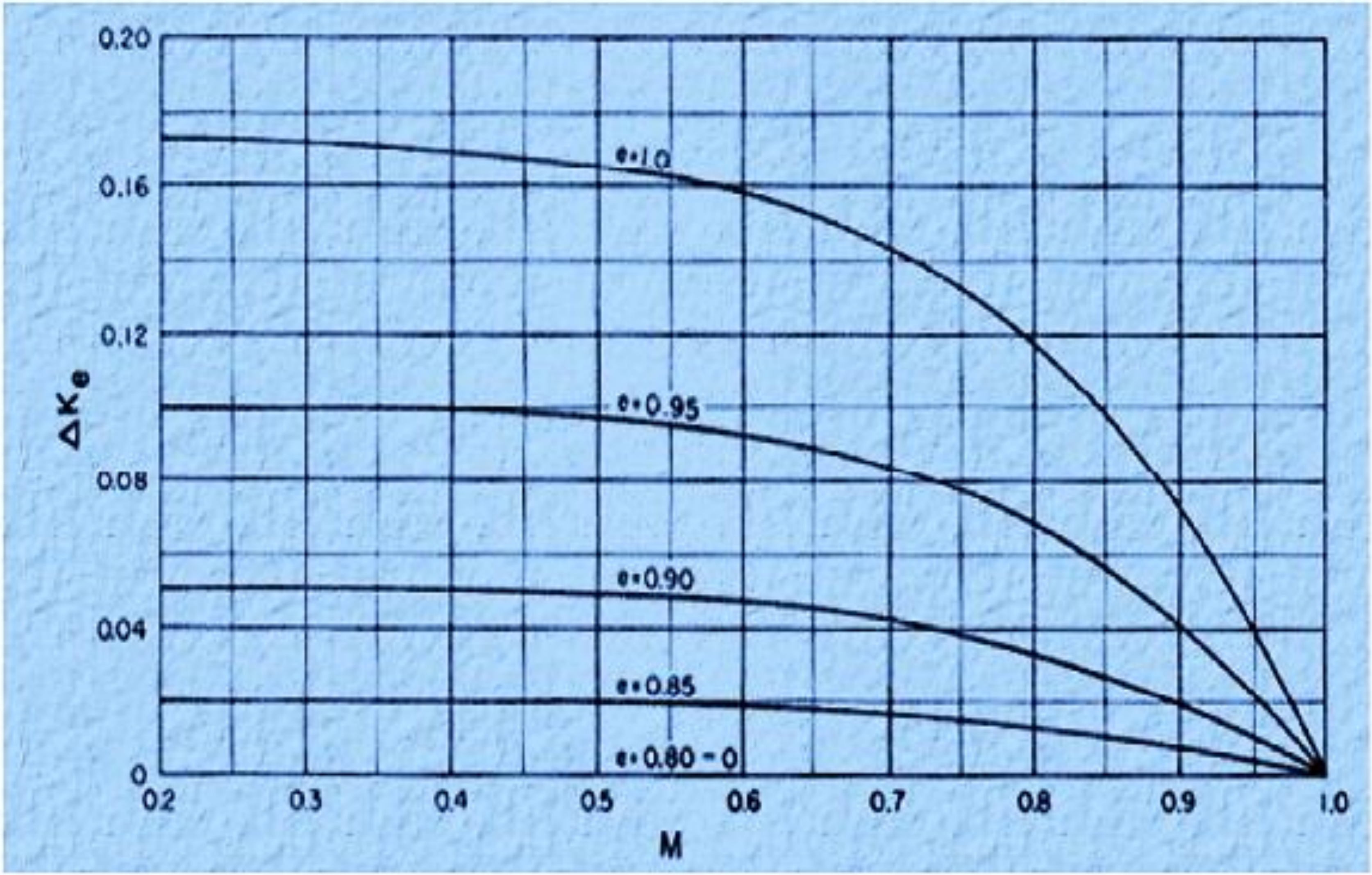


Figure . Incremental backwater coefficient for eccentricity.

Effects of skew (ΔK_s)

- ✓ The opening width is projected upstream on section-1, which is perpendicular to the general direction of flow.
- ✓ Note that the incremental backwater coefficient, ΔK_s, can be negative or positive.
- ✓ The negative values result from the method of computation and do not necessarily indicate that the backwater will be reduced (increasing hydraulic efficiency) by employing a skewed crossing.
- ✓ These incremental values are to be added to K_b.

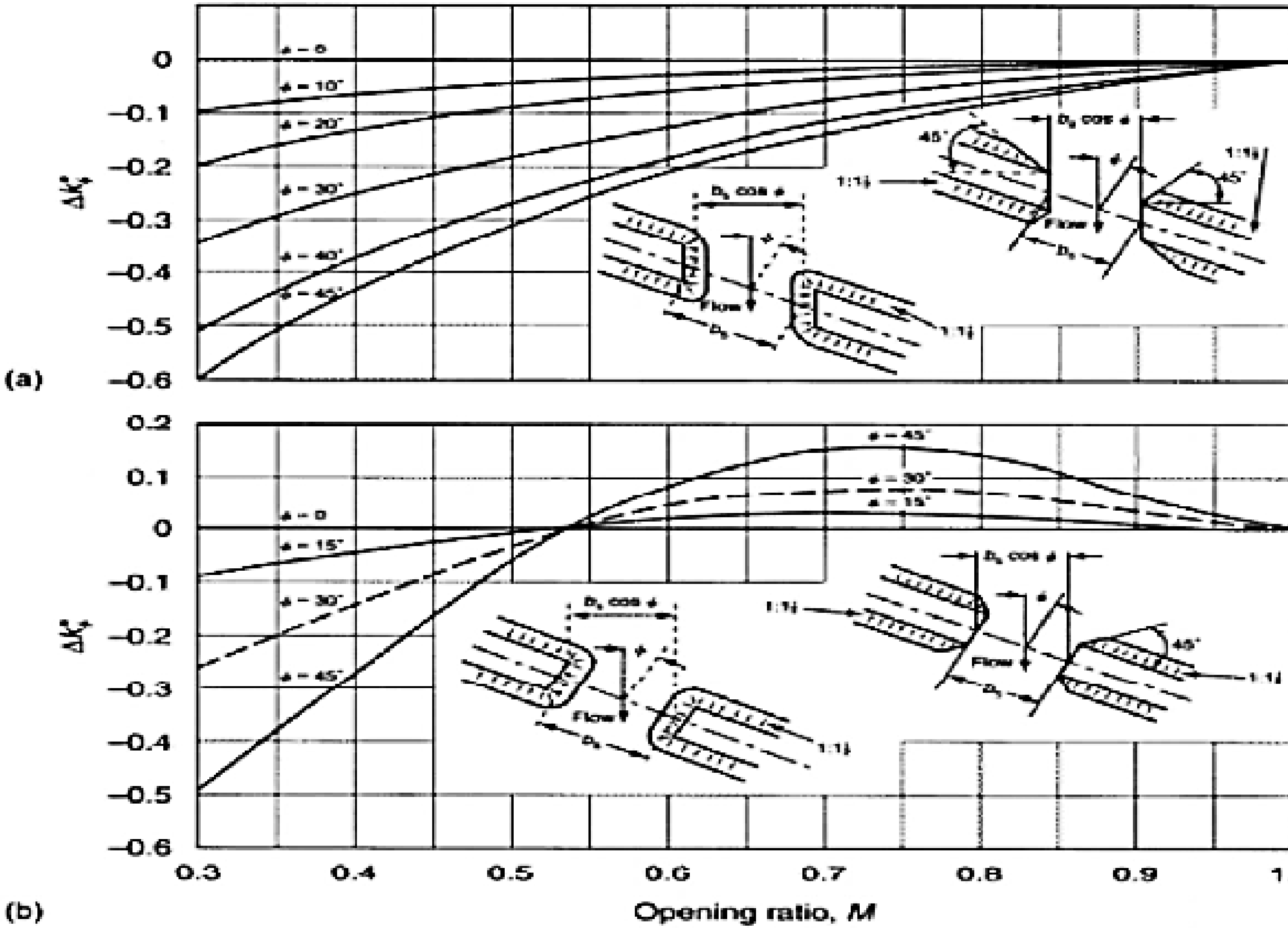


Figure Incremental backwater coefficient for skew

- ✓ Total backwater coefficient (K*) = $K_b + \Delta K_p + \Delta K_e + \Delta K_s$
- ✓ It is possible to determine from M and the angle of skew (ø), the width of skewed opening (b_s) needed to give the same backwater as a normal opening of width b.

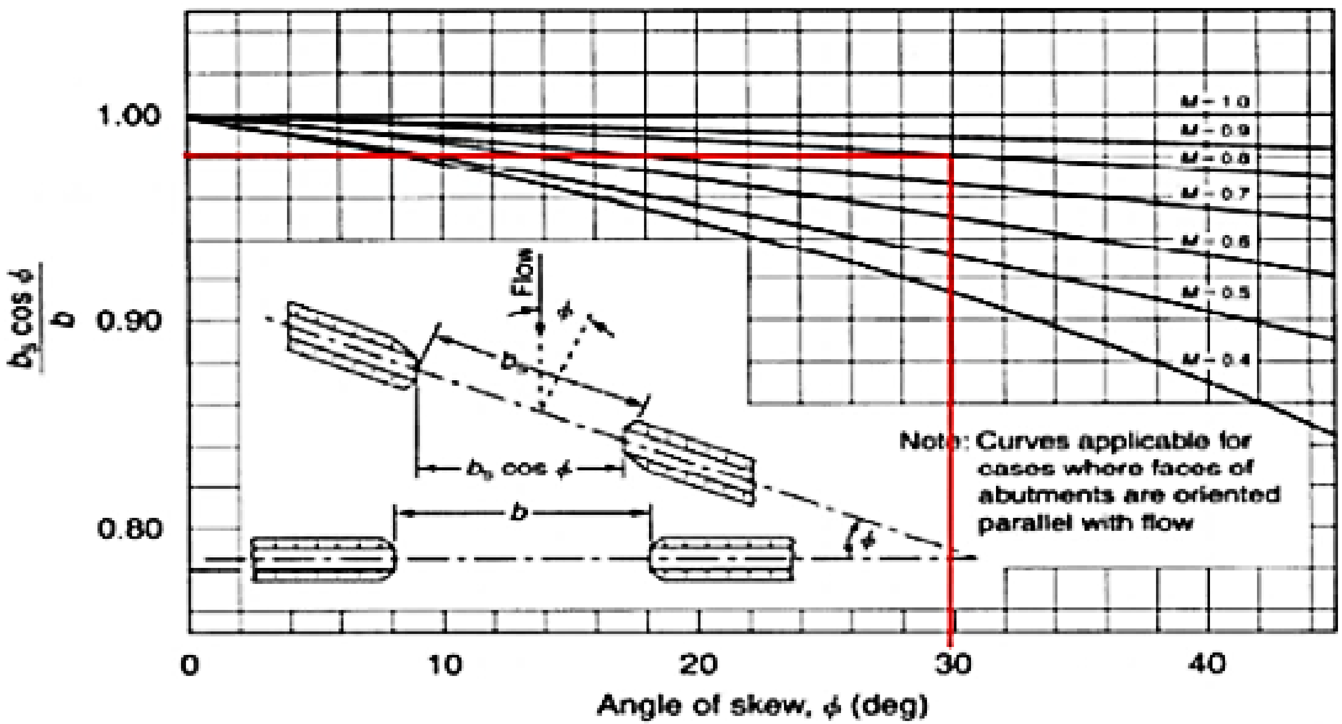


Figure: skewed span (b_s) required to give the same backwater as an opening of width b perpendicular to the direction of the approaching flow.

Bridge backwater computation procedure

The following steps are used to determine the backwater produced by a bridge constriction:

1. Determine the magnitude and frequency of the discharge for which the bridge is to be designed.
2. Determine the stage of the stream at the bridge site for the design discharge.
3. Plot a representative cross section of stream for design discharge at section-1.
4. Subdivide the cross section plotted in step-3 according to marked changes in depth of flow and changes in roughness. Assign values of Manning roughness coefficient, n, to each sub-section (Table).
5. Compute conveyance and then discharge in each subsection
6. Using cumulative conveyance and discharge at section-1, compute slope of stream, S_o.
7. Determine value of kinetic energy coefficient, α_1
8. Plot natural cross section under proposed bridge based on normal water surface for design discharge and compute gross water area (including area occupied by piers).

9. Compute bridge opening ratio, M
10. Obtain value of K_b^* from base curve for symmetrical normal crossings.
11. If piers are involved, compute value of J and obtain incremental coefficient, ΔK_p, from curve.
12. If eccentricity is severe, compute value of ***e*** and obtain incremental coefficient, ΔK_e, from curve.
13. If a skewed crossing is involved, observe proper procedure in previous steps, then obtain incremental coefficient, ΔK_s, from curve.
14. Determine total backwater coefficient, K*, by adding incremental coefficients to base curve coefficient, K_b.
15. Estimate α_2 from curve, then make allowable for any unusual topographic, vegetative or approach condition which may lead to further asymmetrical velocity distribution in the bridge constriction.
16. Compute backwater h*
17. Convert backwater to water surface elevation at section-1 if computations are based on normal stage at bridge.

<u>Example 3.2</u>

A bridge is being designed to cross the channel described in example 3.1. The crossing will have a skew (ø) of 30^o with vertical wing wall abutments parallel to the flow, as shown in figure below. As in the previous example, assume that the design flood is 98.030m^3/s, Y_N = 4m and the height of the opening is 5m above bed level. Two alternate designs are being considered:

- **(a)** a single span with a skewed width of 11.547m, which leaves the main channel free of obstruction, and
- **(b)** a two -span structure with a skewed width of 11.547m between the abutments including a round-nosed pier with a skewed width of 1.000m in the Centre of the main channel. Calculate the afflux h_1*.

- ****(b)***individual assignment-II*

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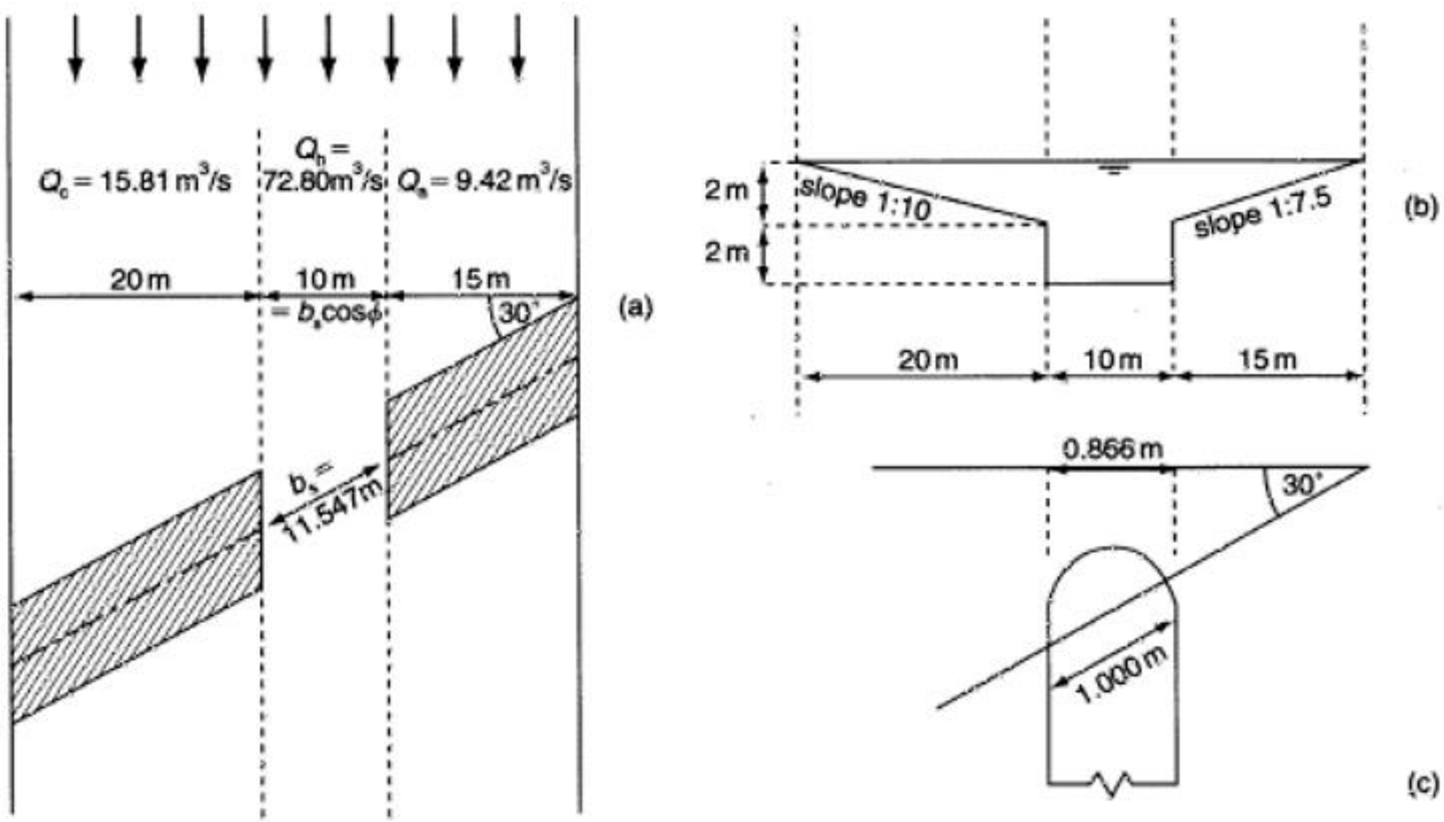
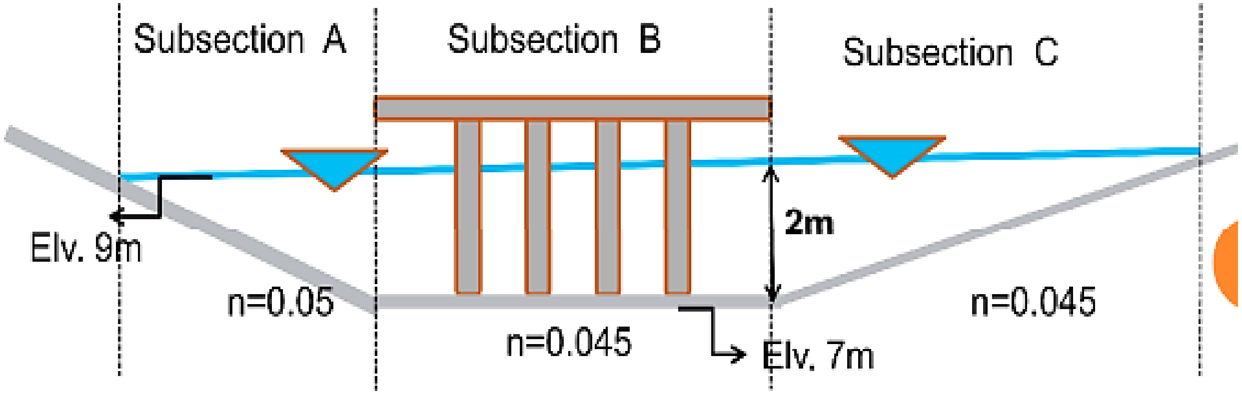
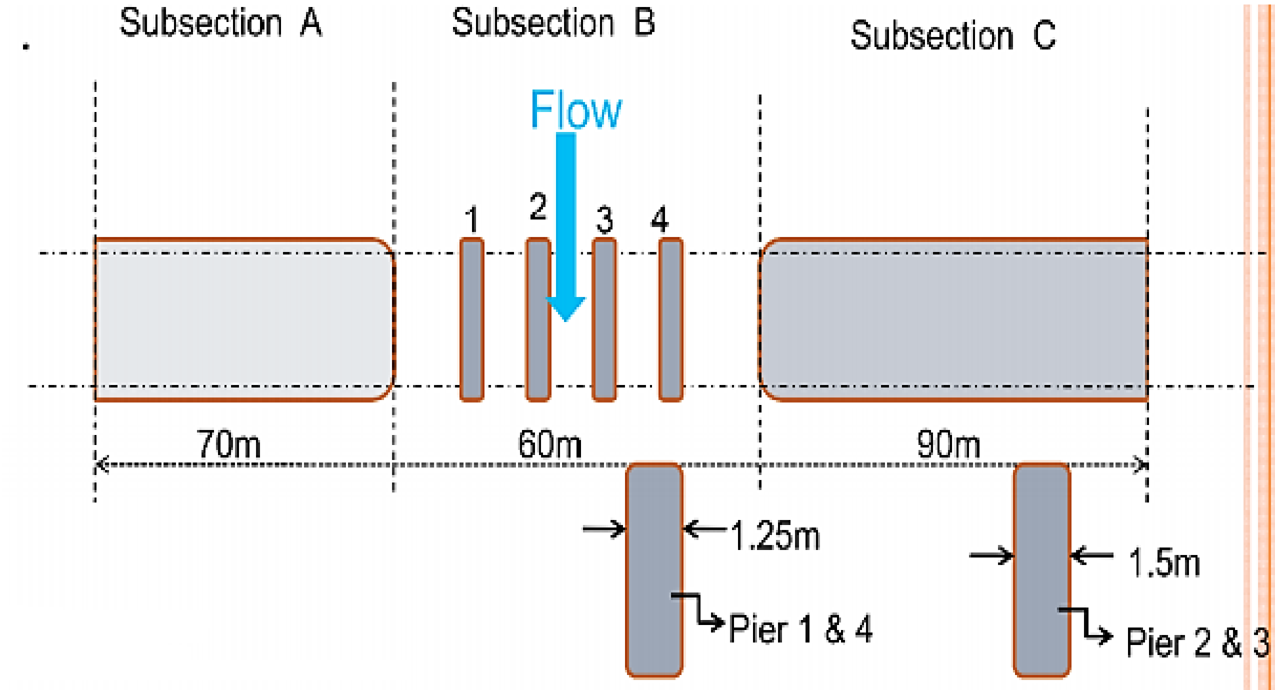


$Q_c = 15.81\ m^3/s$
$Q_h = 72.80 m^3/s$
$Q_s = 9.42\ m^3/s$
20 m
10 m
$= b_s \cos\phi$
15 m
30°
$b_s = 11.547 m$
(a)
2 m
2 m
slope 1:10
slope 1:7.5
(b)
20 m
10 m
15 m
0.866 m
30°
1.000 m
(c)

Group assignment-(III)

The channel crossing shown in figure below is with the following information: Cross section of river at bridge site showing areas, wetted perimeters and values of Manning n; normal water surface for design = Elv. 9m at bridge; average slope of river in vicinity of bridge S_o =0.0005 m/m; cross section under bridge showing area below normal water surface and width of roadway = 12m. The stream is essentially straight, the cross section is relatively constant in the vicinity of the bridge and the crossing is normal to the general direction of flow.



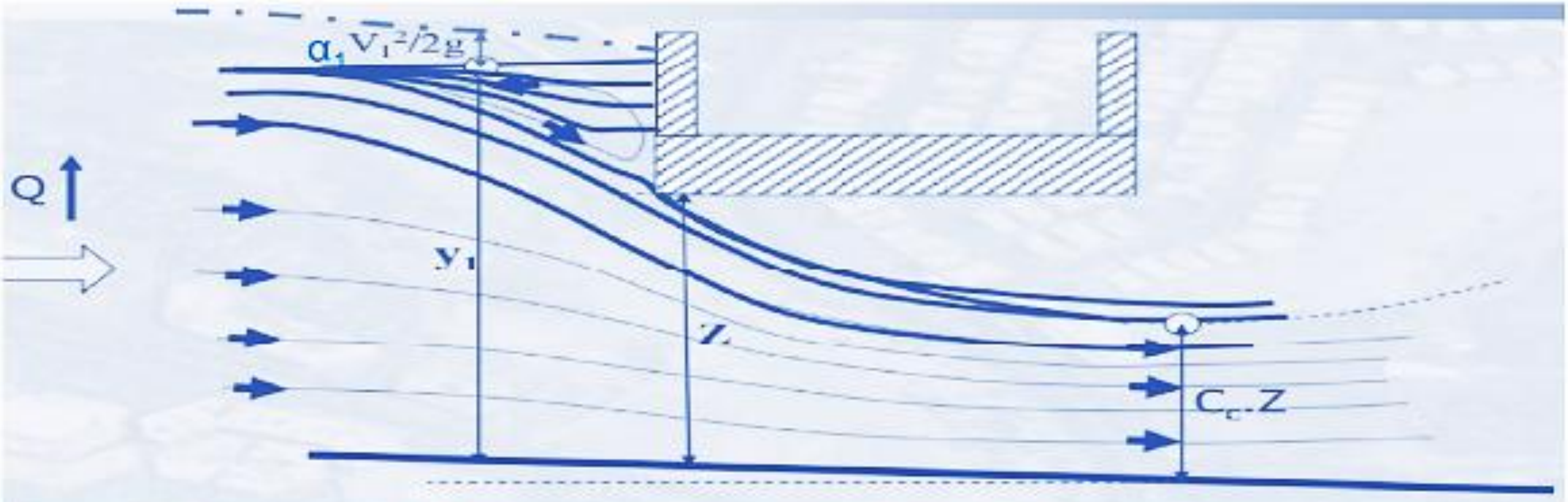


Find:

a) Conveyance at upstream cross section (K)
b) Discharge of stream at Elv. 9m
c) Velocity head correction coefficient, α_1
d) Bridge opening ratio, M
e) Backwater produced by the bridge

B) High flow case (waterway opening is submerged)

Orifice flow (sluice gate pressure flow)



$$y_1 + \frac{\alpha_1 V_1^2}{2g} = y_{vena} + \frac{V_{vena}^2}{2g} + H_L$$

Cc = 0.5

$$V_{vena} = C_v\sqrt{2g\left(y_1 - y_{vena} + \frac{\alpha_1 V_1^2}{2g}\right)}$$

$$V_{vena} = C_v\sqrt{2g\left(y_1 - C_c.Z + \frac{\alpha_1 V_1^2}{2g}\right)}$$

$$V_{vena} = C_v\sqrt{2g\left(y_1 - 0.5Z + \frac{\alpha_1 V_1^2}{2g}\right)}$$

$$q = y_{vena}.C_v\sqrt{2g\left(y_1 - 0.5Z + \frac{\alpha_1 V_1^2}{2g}\right)}$$

$$q = C_c Z.C_v\sqrt{2g\left(y_1 - 0.5Z + \frac{\alpha_1 V_1^2}{2g}\right)}$$

$$q = C_d Z\sqrt{2g\left(y_1 - 0.5Z + \frac{\alpha_1 V_1^2}{2g}\right)}$$

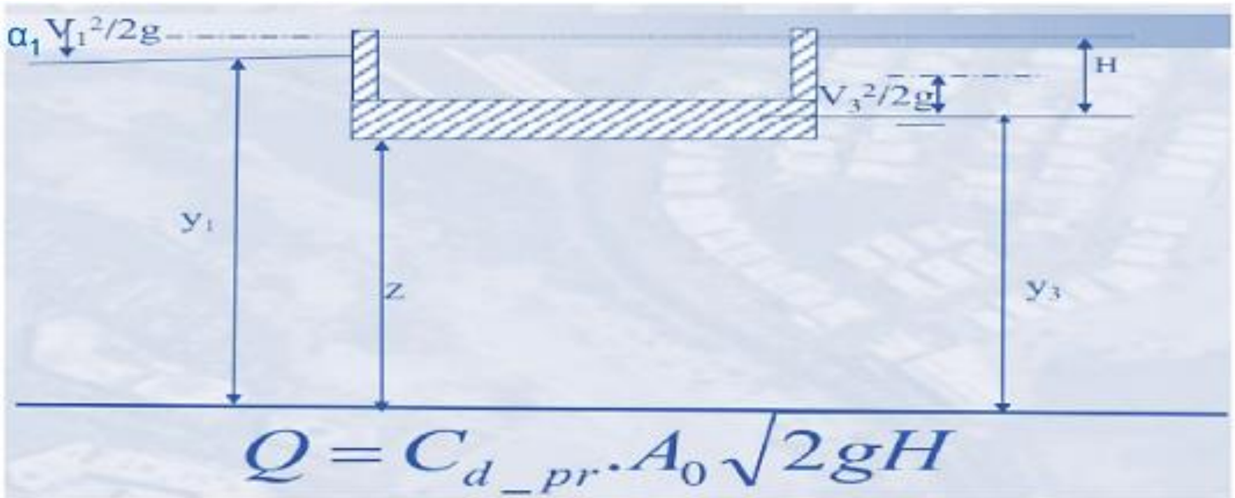
$$Q = C_{d\_orf} . A_o \sqrt{2g\left(y_1 + \frac{\alpha_1 V_1^2}{2g} - \frac{z}{2}\right)}$$

$$Q = C_{d\_orf} . A_o \sqrt{2g(H)}$$

Where,

- $C_{d\text{-orifice}}$ = is the coefficient of discharge, it ranges from 0.35 – 0.5
- A_o = is the net area of the bridge opening, $A_o = Z * b_{net}$
- Z = is the bridge low chord height
- b_{net} = clear span width of bridge opening.

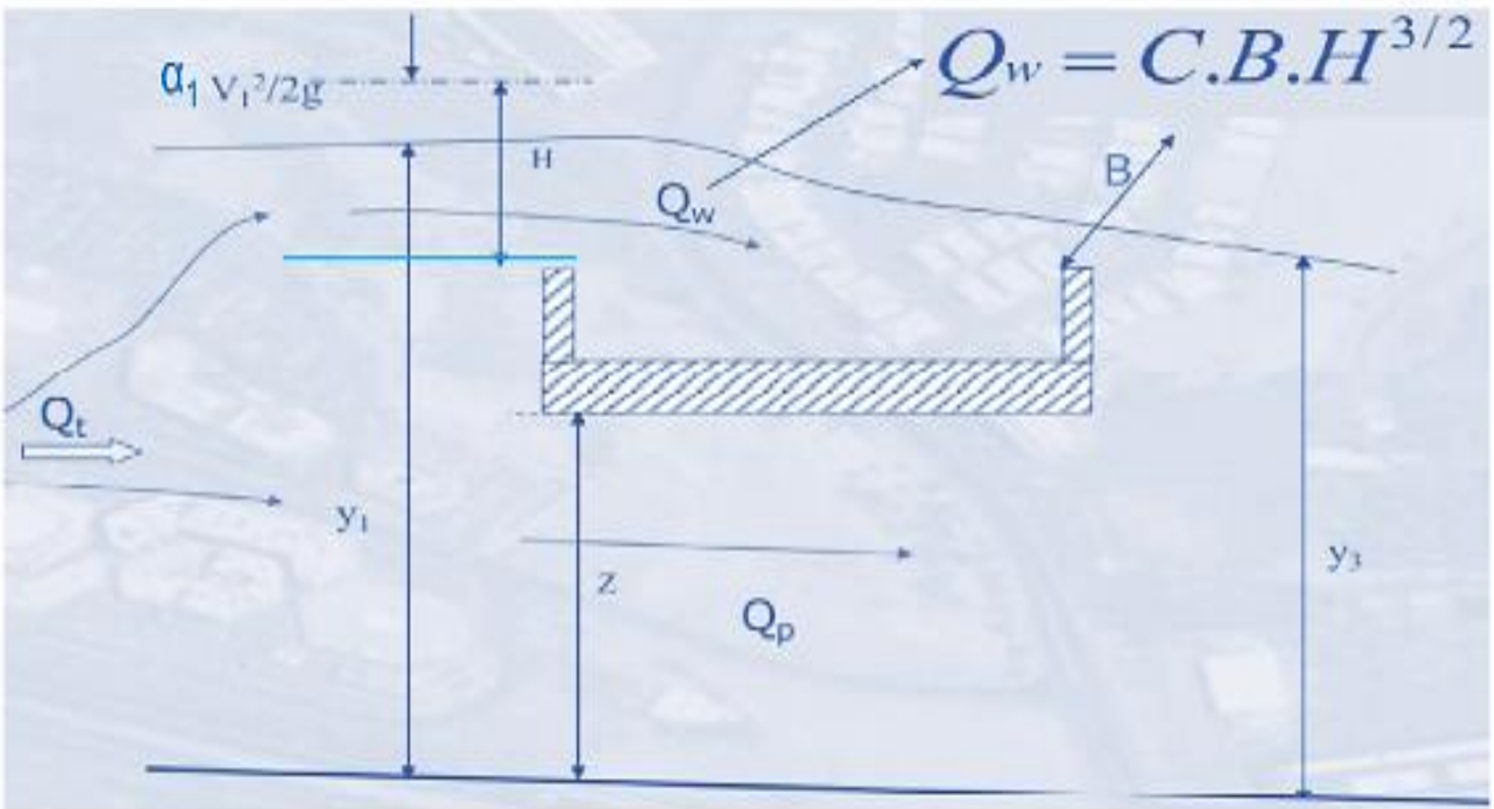
Pressurized Flow



$$H = Y_1 + \alpha_1 \frac{V_1^2}{2g} - Y_3$$

- ✓ Typical values for the discharge coefficient C_d range from 0.7 to 0.9.
- ✓ A value of 0.8 is used for most bridges.

Weir flow

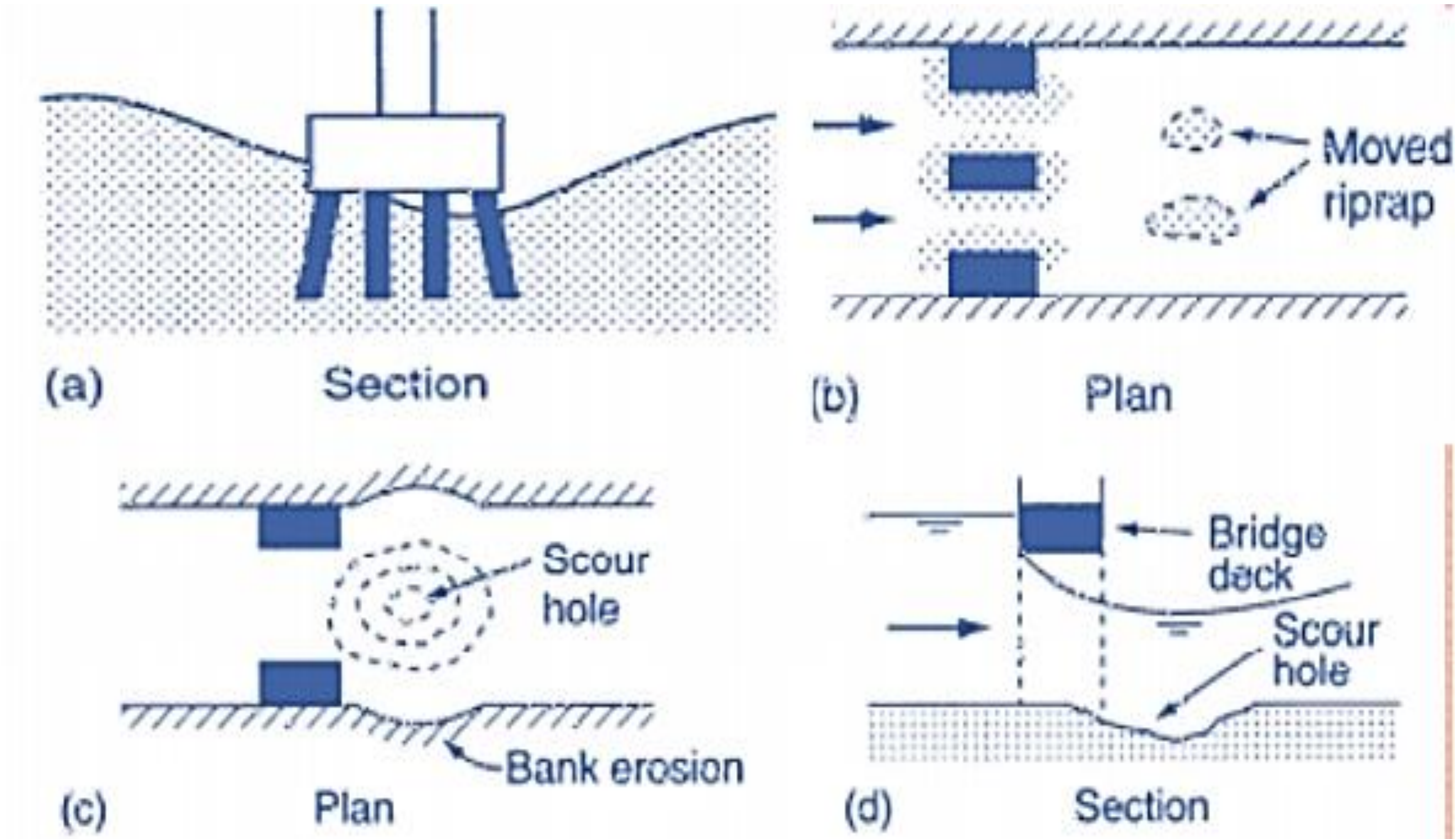


$$Q_{pr} = C_{d\_pr}.A_2\sqrt{2g(y_1 + \frac{\alpha_1 V_1^2}{2g} - y_3)}$$

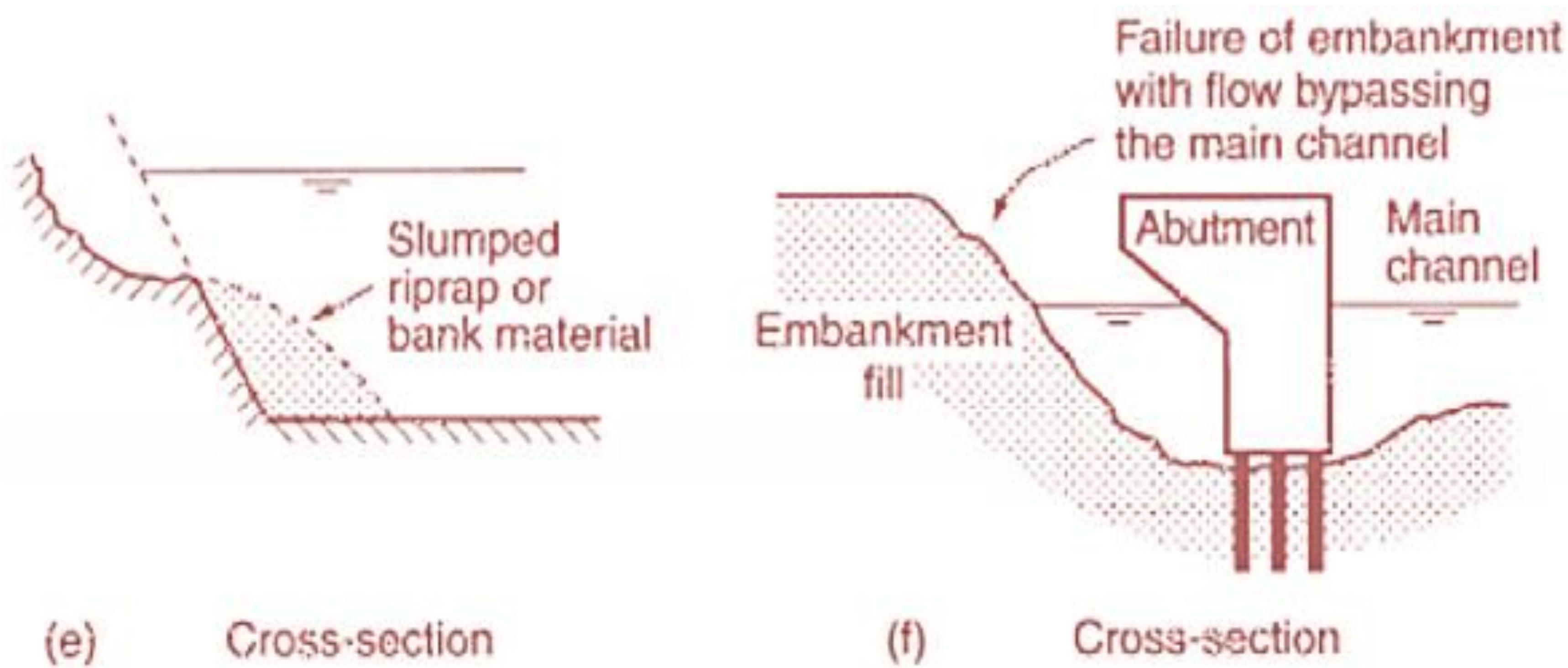
Bridge scour

- ✓ Scour is the removal of material from the bed and banks of streams as a result of the erosive power of flowing water.
- ✓ The most common cause of bridge failures is from floods eroding bed material around bridge foundations.
- ✓ Such failure is primarily due to:
  - Inadequate knowledge about scour phenomenon when the bridge was constructed
  - Inadequate data and knowledge about design flood
- ✓ Scour rate depends mainly on:
  - Flow power, material resistance, sediment in/out balance (equilibrium scour).

(a) Pier piles and pile cap exposed ***(b) Pier and abutment riprap moved d/s***



(c) D/s scour hole and bank erosion ***(d) D/s scour hole arising from submergence (pressure flow)***



e. Slumped material at the toe arising from failure of the riprap or bank.

f. Erosion and failure of the highway embankment with flow on both sides of the abutment.

Fig. Zharawa Bridge was greatly damaged by flood. The bridge was subjected to foundation settlement, movement and displacement which resulted in cracks and concrete failure to several beams.

Fig. Abutment Fill Loss

Types of Scour

- ✓ Based on bridge crossings scour as *clear water scour* and *live bed scour*.
- ✓ If mean velocity (V) of the flow at upstream is less than the scour critical velocity (V_s) then the bed material upstream of the bridge is at rest:
  - *this is referred to as* the *clear water condition* because the approach flow is clear and *does not contain sediment*.
- ✓ In this case *any bed material that is removed from a local scour hole is not replaced by sediment* being transported by the approach flow.
- ✓ *Live-bed scour* occurs where $V > V_s$ and the bed material upstream of the crossing is moving.
- ✓ This means that the approach flow continuously transports sediment into a local scour hole.

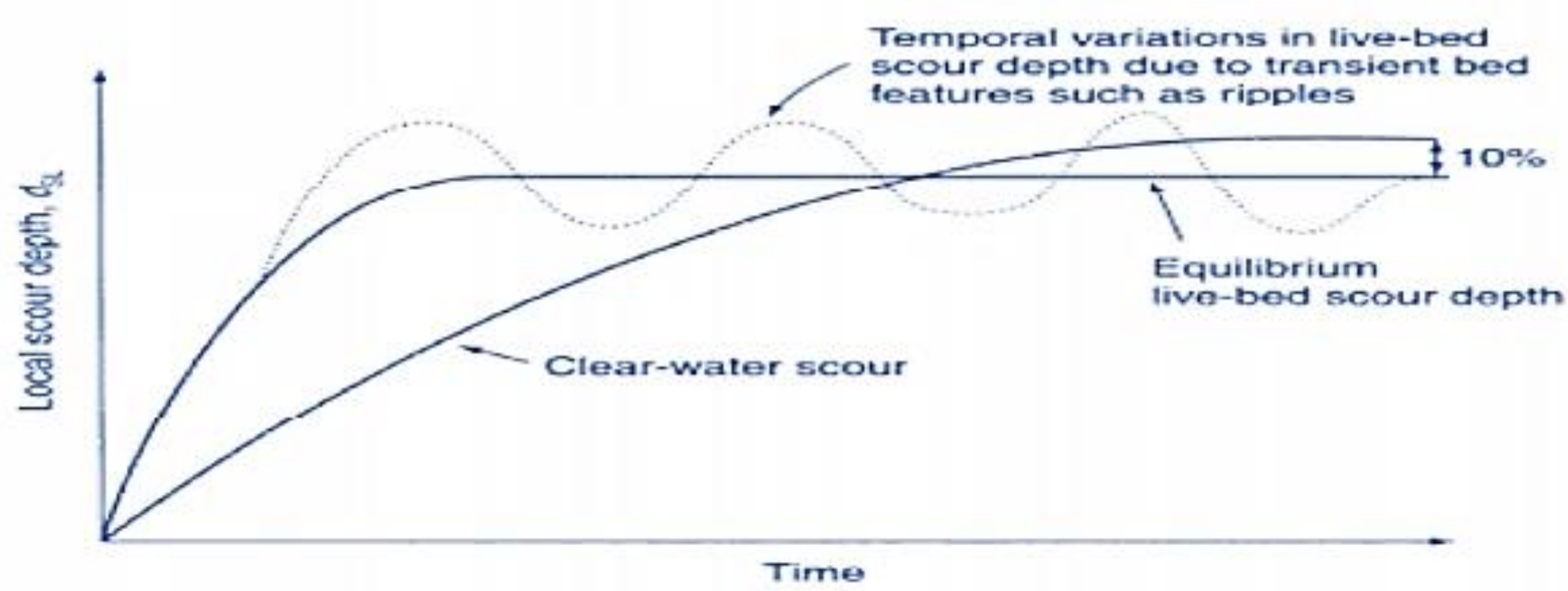
Cont’d…

- ✓ The equilibrium scour depth is achieved when material is transported into the scour hole & at the same rate at which it is transported out.
- ✓ To determine whether the flow condition is *clear-water or live-bed*, Neill (1968) equation given below, applied to the unobstructed flow. If the average velocity (V = Q/A m/s) in the scour will be live bed.

$$V_s = 1.58\,[(s_s - 1)\,g\,D_{50}]^{1/2}\left(\frac{Y}{D_{50}}\right)^{1/6}$$

Where, S_s=2.65 (specific gravity of the sediment), g=9.81 m/s^2; D_{50} is the median diameter (m) at which 50% of the bed material by weight is smaller than the size denoted and Y is the average depth (m) in the upstream channel. So that the equation becomes

$$V_s = 6.36\,Y^{1/6}D_{50}^{\;1/3}$$



- ✓ Figure diagrammatic illustration of the increase in local scour depth (d_{sL}) with time for clear water and live bed conditions.
- ✓ The oscillations for the live bed condition (dotted) are due to transient bed features such as ripples and dune.
- ✓ The final clear water scour depth exceeds the equivalent equilibrium live bed depth by about 10%. At any particular location both clear water and live bed scour may be experienced.
- ✓ During a single flood the mean velocity will increase and decrease as the discharge rises and falls, so it is possible to have clear water conditions initially, then a live bed, then finally clear water again (Fig. below). The maximum scour depth may occur under clear water conditions, not at the flood peak when live bed scour is experienced.

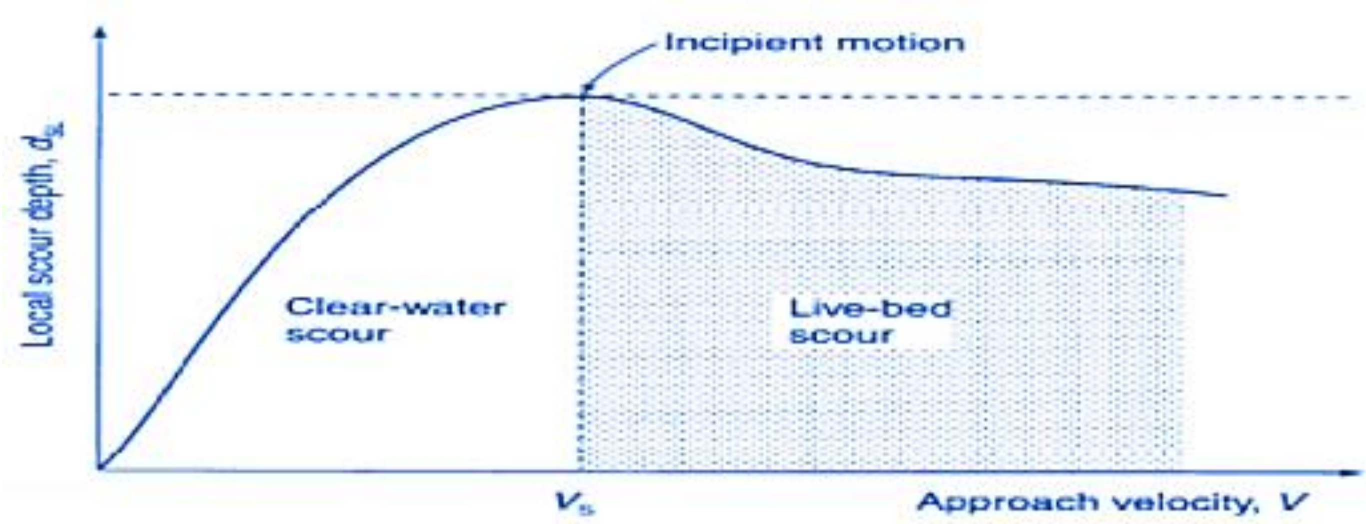
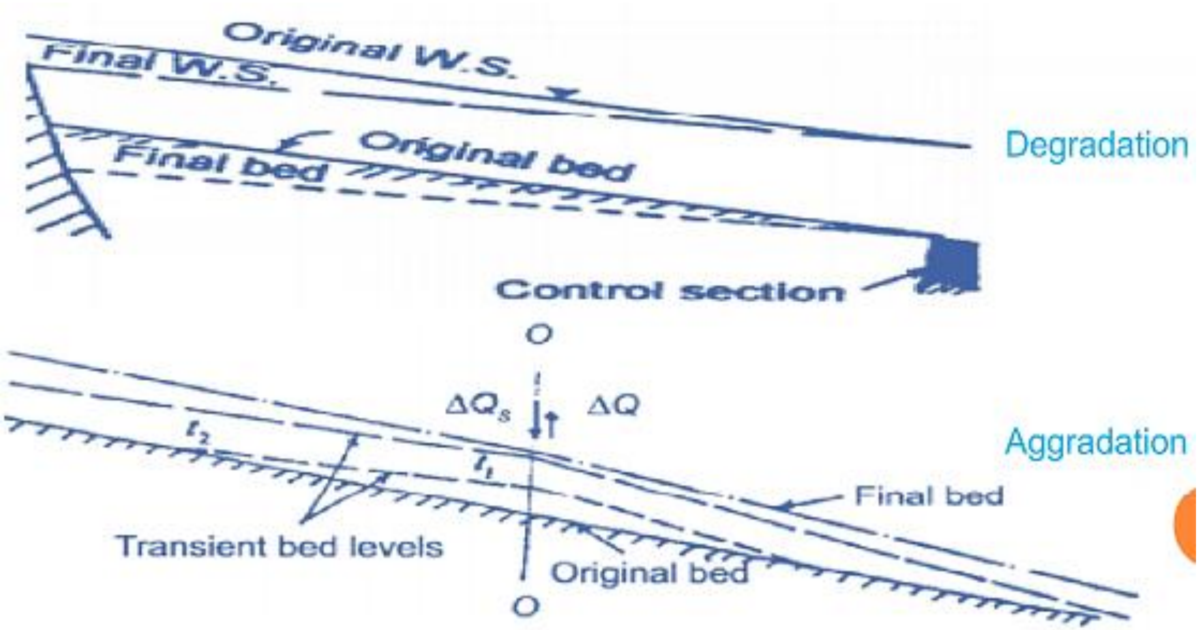


Figure above shows the variation of local scour depth (d_{SL}) with approach velocity

- ✓ The main components of scour include:- long term or natural scour (degradation or aggradations), contraction scour (bridge opening) and local scour (piers and abutments)
- ✓ Degradation or Aggradations:- can only occur with a live bed ($V > V_s$), and it is not the result of bridge or embankment construction.
- ✓ Degradation is the scouring of bed material due to increased stream sediment transport capacity that results from an increase in the energy gradient or a decrease in the sediment load.
- ✓ Aggradation is the deposition of bed load due to a decrease in stream sediment transport capacity that results from a reduction in the energy gradient or an increase in the sediment load.



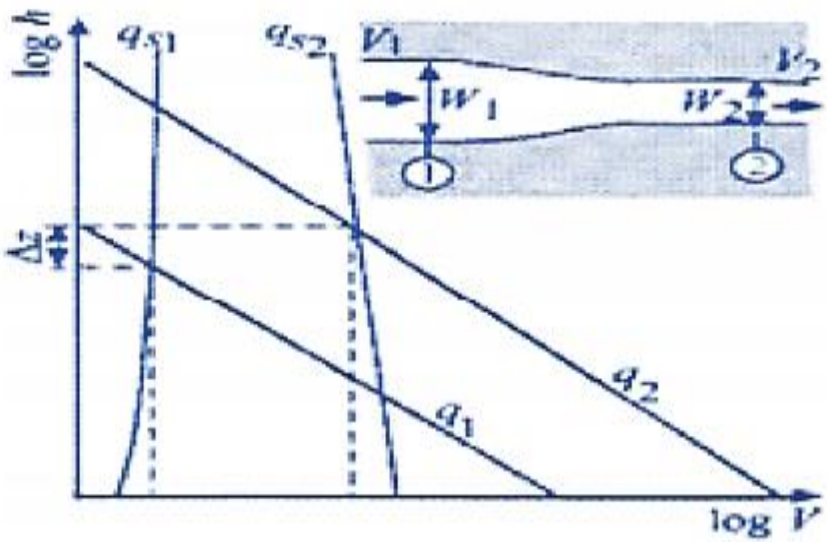
- ✓ Degradation can be expressed qualitatively by Lane's balance analogy (Lane, 1955; Bryan et al. 1995) equation:

$$QS_O \propto Q_S D_{50}$$

Where, Q is the water discharge, S_o is the channel bed slope, Q_s is the bed material discharge, and D_{50} is the median grain size of the bed material.

- The significance of degradation scour to bridge design is that the engineer has to decide whether the existing channel elevation is likely to be constant over the 100 year life of the bridge or whether it will change.
- If change is probable then it must be allowed for when designing the waterway and foundations.

✓ <u>Contraction Scour</u>:- occurs over a whole cross section as a result of the increased velocities arising from a narrowing of the channel by a constriction.



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- The approach flow depth h_1 and average approach flow velocity V_1 result in the sediment transport rate q_{s1}.
- The total transport rate to the contraction is $W_1 q_{s1}$, in which W_1 is the width of the approach.
- If the water flow rate $Q_1 = W_1 q_1$ in the upstream channel is equal to the flow rate at the contracted section then by continuity:

$$q_2 = \frac{W_1}{W_2} q_1$$

Where, $q_1 = h_1 V_1$, $q_2 = h_2 V_2$

- The sediment transport rate at the contracted section after equilibrium is esta [illegible]e:

$$q_{s_2} = \frac{W_1}{W_2} q_{s_1}$$

- The depth of scour Δz that is due to the contraction is then, $\Delta z = h_2 - h_1$. The contracted scour depth represents an average the channel width, and symmetry is assumed in the calculation.
- *Local scour (Piers, abutments)*:- arises from the increased velocities and associated vortices as water accelerates around the corners of abutments, piers and spur dykes.
- *Local pier scour*:- The velocity upstream of the pier accelerate around the pier and flow is directed downward along the front face of pier.
- A “horseshoe” vortex forms where the downward flow reaches the bed and the size of the vortex increases as the scour hole enlarges.
- Sediment deposition occurs in the wake area downstream of the pier.

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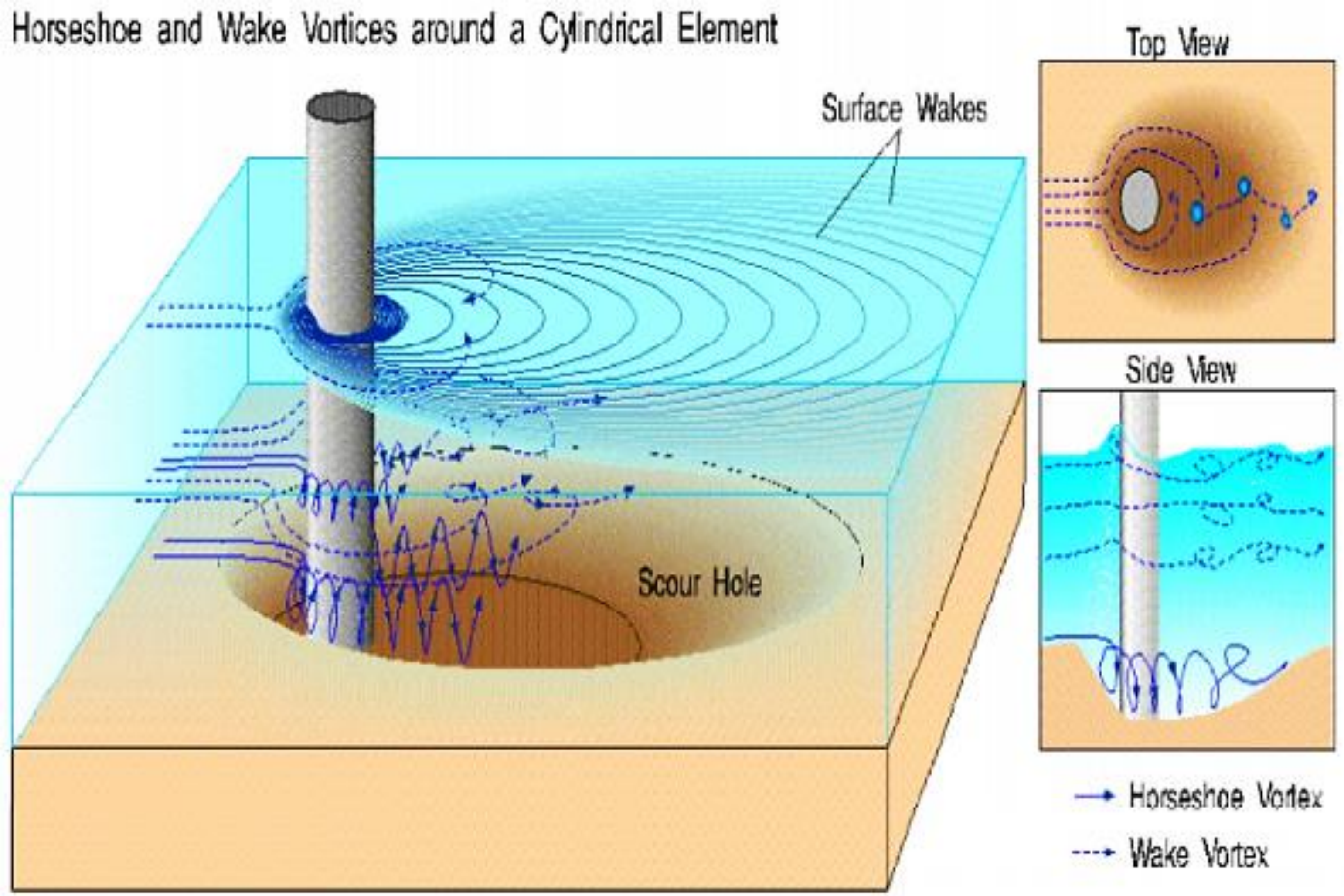

Pier scour

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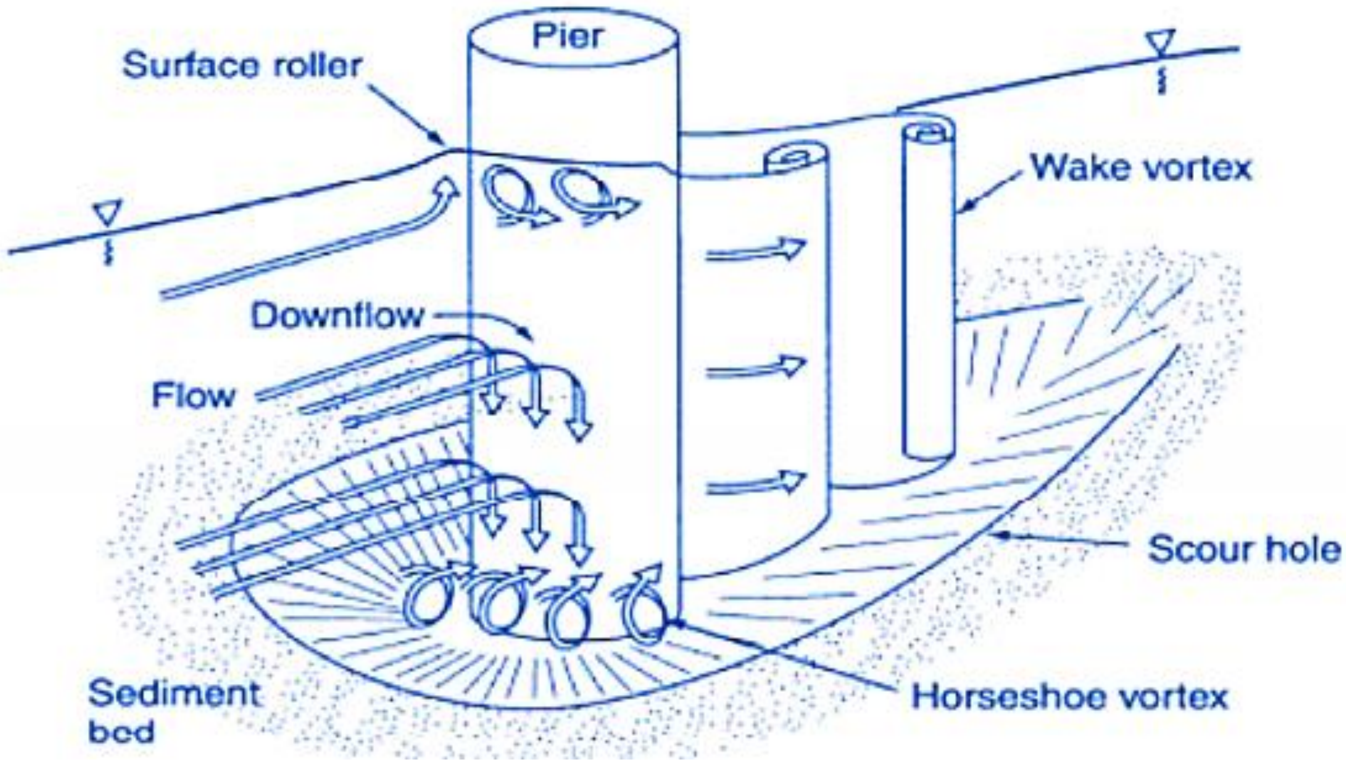

Figure above shows the flow pattern and scour hole at a cylindrical pier. The down flow, horseshoe vortex and wake vortex are the principal cause of local bed erosion

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There are many factors that influence the magnitude of pier scour:

- ✓ *Hydraulic factors*: (velocity (V), depth (y) and angle of attack (θ) of the flow approaching the pier, the water's density and viscosity, Froude number)
- ✓ *Pier factors*: (width, length, shape); Scour depth increases with increasing pier width (b_p).
- ✓ Square shape piers increasing scour depths by 10 – 30% than round nosed piers.
- ✓ *Sediment factors:* (grain size distribution, size (D_{50}), density, cohesive properties).
- ✓ *Bed configuration*: those with D < 0.7mm can have various bed configurations: ripples, dunes, plane bed or antidunes…etc.
- ✓ For both clear water and live bed conditions Richardson et al. (1993) recommended the Colorado State University (CSU) equation for the estimation of equilibrium pier scour depth (d_{sp}m)

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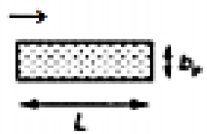
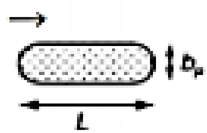
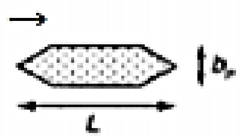
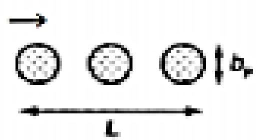
$$d_{SP} = 2.0\, Y_2\, K_{1P}\, K_{2P}\, K_{3P} \left(\frac{b_P}{Y_2}\right)^{0.65} F_2^{0.43}$$

Where, Y_2 is the flow depth (m) at the bridge section directly upstream of the pier (m), K_{1P} is an adjustment factor for pier nose shape obtained from Table below for ø < 5^o, K_{2P} is an adjustment factor for the angle of attack (ø > 5^o) obtained from curve, K_{3P} is an adjustment factor for bed configuration obtained from Table, b_p is the pier width (m) V_2 is the mean velocity of flow (m/s) at the bridge directly upstream of the pier and F_2 is the Froude number = $V_2/(gy_2)^{1/2}$.

- Note that if ø > 5^o the factor K_{2P} dominates, so K_{1P} can be taken as 1.0.

Table K_{1P}—adjustment factor for pier nose shape applicable when $\varphi<5°$

Shape of pier		K_{1P}
Square nose		1.1
Round nose		1.0
Circular cylinder		1.0
Sharp nose		0.9
Group of cylinders		1.0

Note that these values are the ones most compatible with the CSU pier scour equation recommended by Richardson et al., 1993. They are applicable up to an angle of attack of 5° after which K_{2P} dominates and regardless of shape K_{1P}=1.0. Pier length is not considered important.

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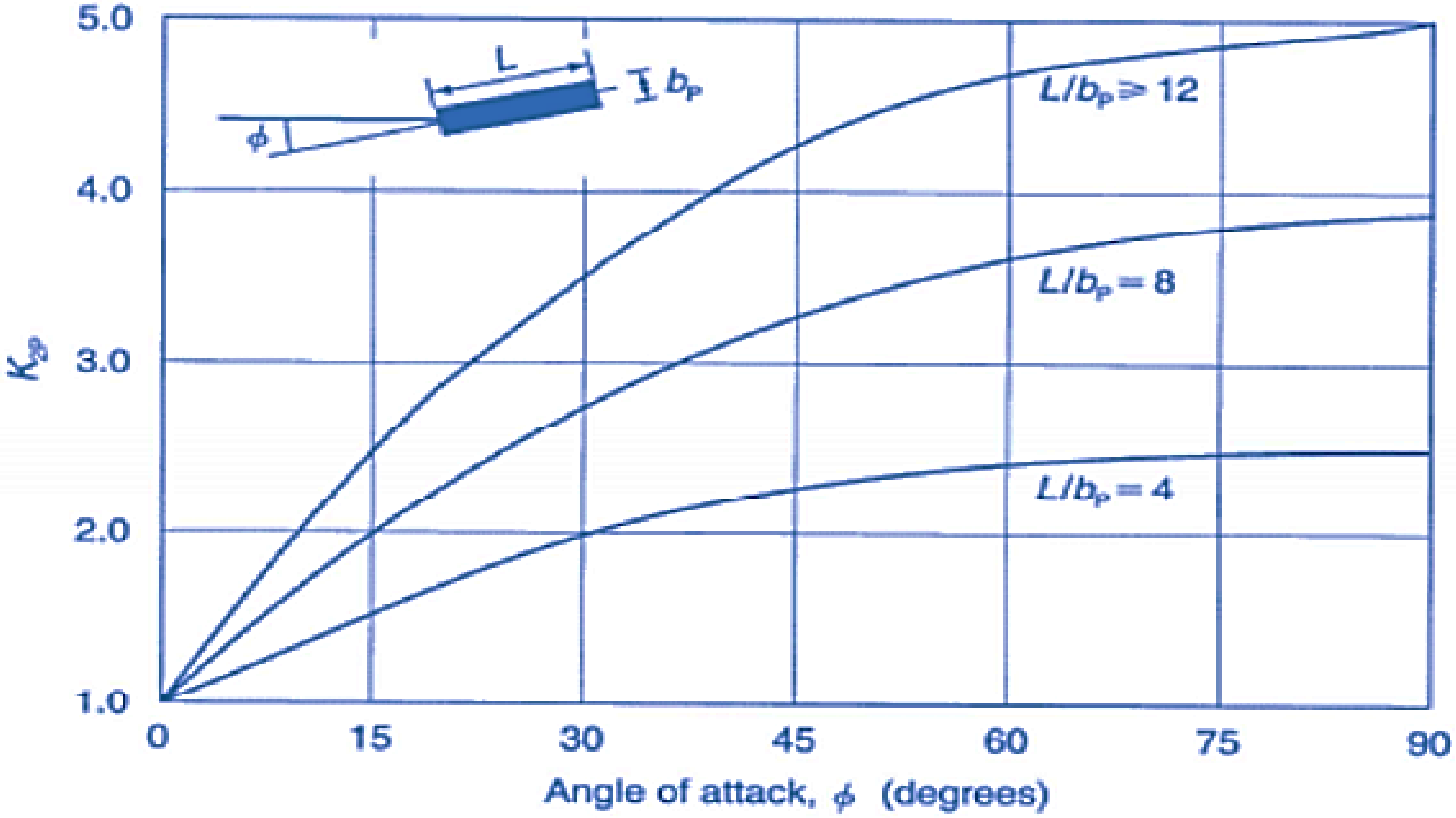

Fig. Pier skew correction factor (K_{2P}) for use with the CSU equation . If ø<5° the correction for skew can be ignored. If ø>5° the skew dominates, so use the value of K_{2P} obtained from the diagram with K_{1P} =1.0 regardless of shape. (After Richardson et al., 1993)

Table K_{3P}: increase in equilibrium pier scour depths for various bed conditions

Bed condition	*Dune height, H (m)*	K_{3P}
Clear-water scour	Not applicable	1.1
Plane bed and antidunes	Not applicable	1.1
Small dunes	0.6–3.0 m	1.1
Medium dunes	3.0–9 m	1.1–1.2
Large dunes	>9m	1.3

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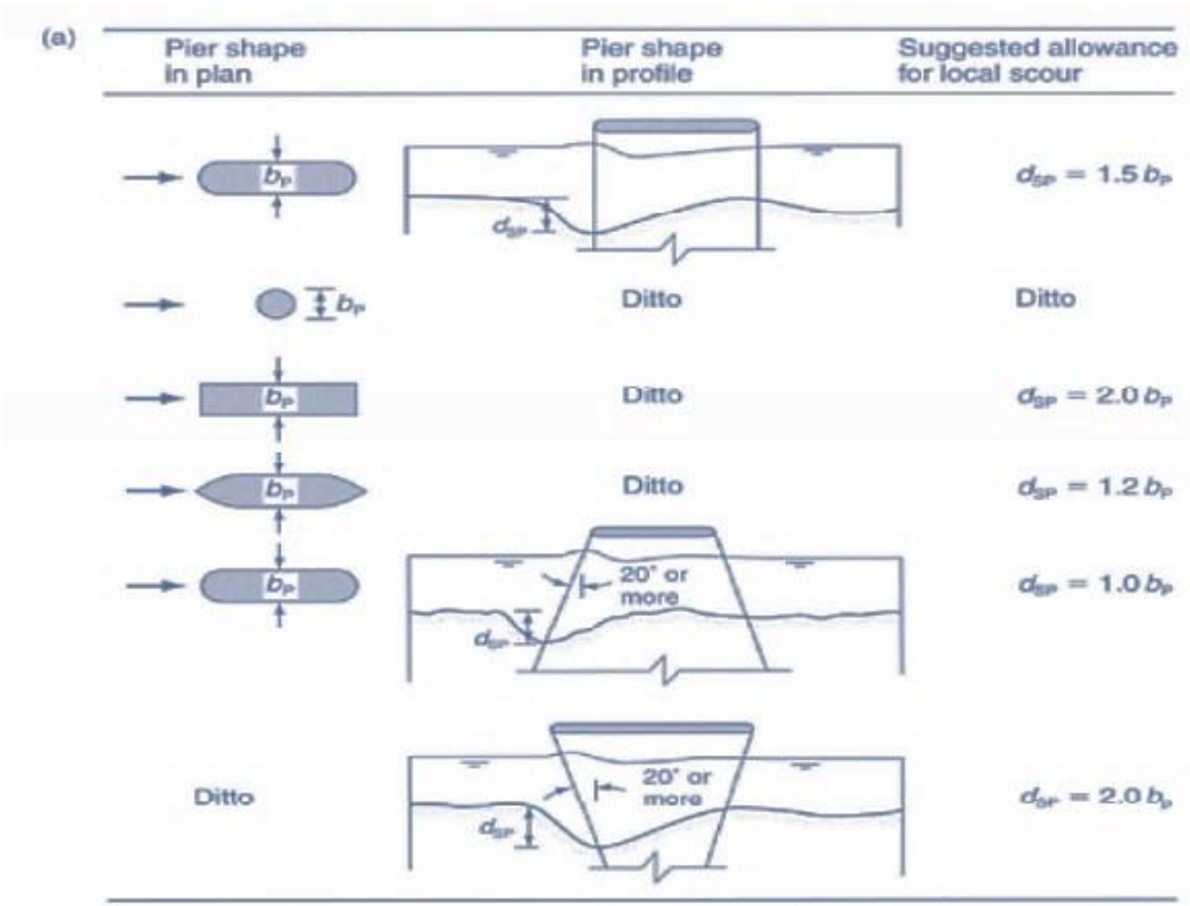

a) Approximate pier scour depth, d_{SP}, for various pier shapes aligned to the approach flow, where b_p is the pier width perpendicular to the flow. If the depth of flow exceeds 5_{bp} then d_{SP} should be increased by 50%.

b) If the approach flow is at an angle to the pier then multiply d_{SP} from part (a) by the factor shown in the table. After (Neill, 1973).

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b Multiplying factors for local scour at skewed piers* (to be applied to local scour allowances of part a).

Angle of attack	Length-to-width ratio of pier in plan		
	4	8	≥12
0°	1.0	1.0	1.0
15°	1.5	2.0	2.5
30°	2.0	2.5	3.5
45°	2.5	3.5	4.5

Local scour depth at abutments (d_{SA}):- Abutments, as well as spur dykes, can have different shapes and they can be set at various angles to the flow. The scour mechanism at a bridge abutment is similar to that at a pier.

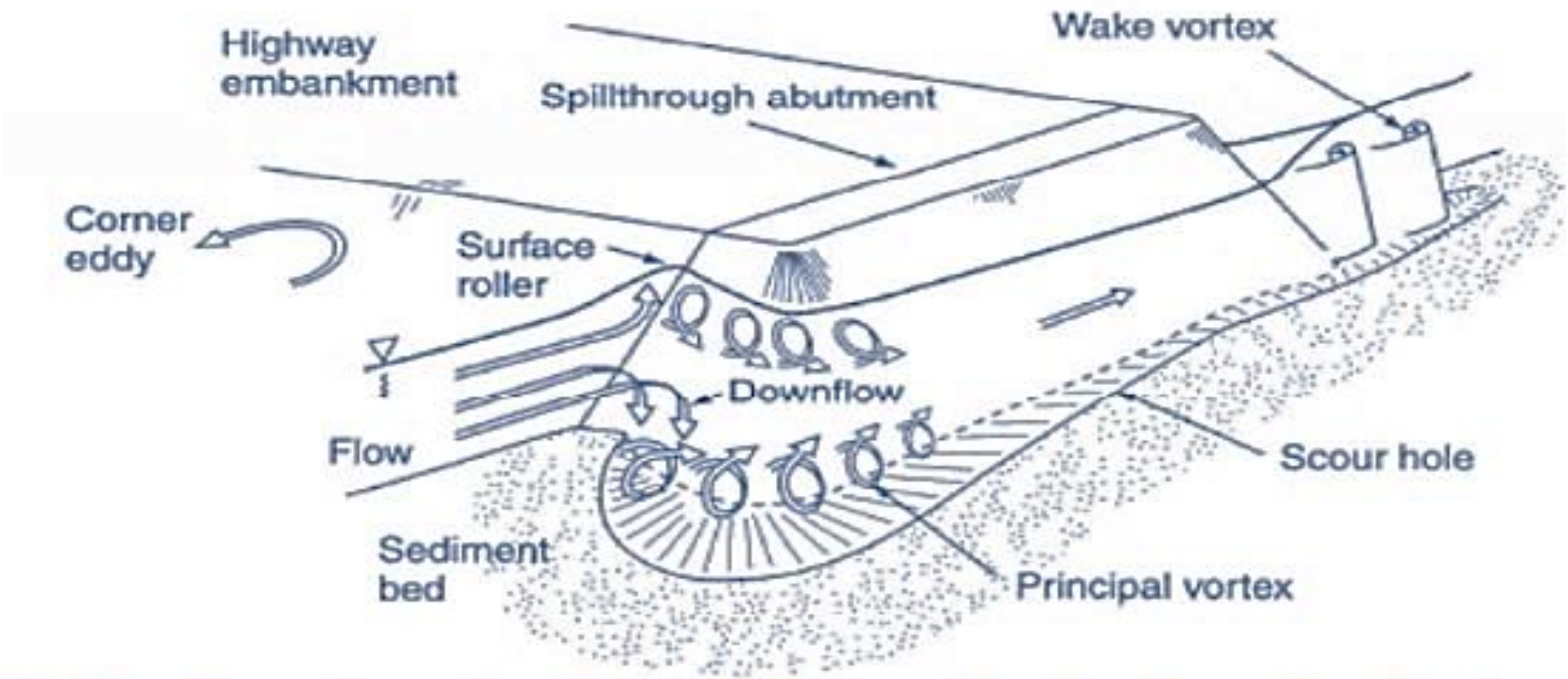
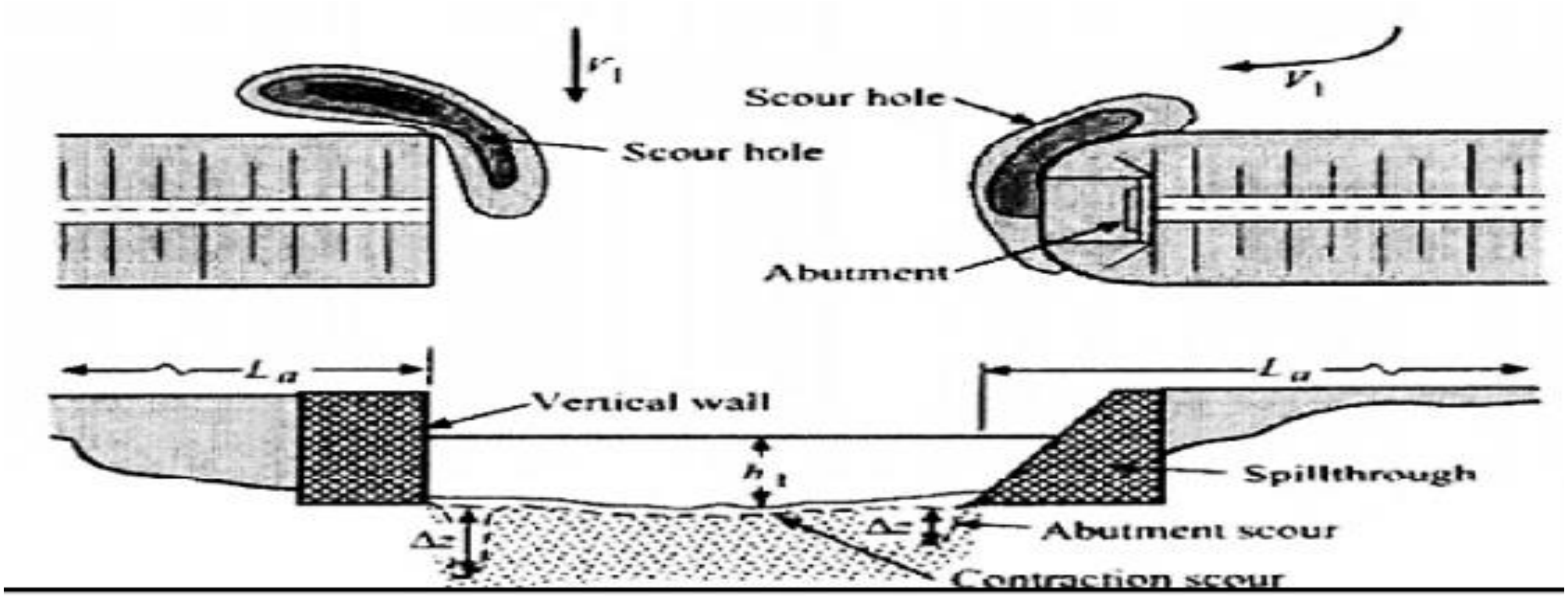


Fig. The flow pattern at a spillthrough abutment. The downflow and principal vortex are the main causes of local bed erosion.

- ✓ Some of the factors that influence abutment scour include the topography of the site (amount of flow intercepted by the bridge abutments), the abutment shape, and the hydraulic characteristics of sediments.



- ✓ For both clear water and live bed conditions Richardson et al. (1993) recommended for the calculation of abutment scour depths, d_{SA}.

$$d_{SA} = Y_{M1} + 2.27 Y_{M1} K_{1A} K_{2A} \left(L_A / Y_{M1} \right)^{0.43} F_{M1}^{\;0.61}$$

Cont'd…

Where: Y_{M1} is the mean depth of flow (m) on the upstream floodplain, K_{1A} is the coefficient for abutment shape from Table, $K_{2A} = (\phi/90)^{0.13}$ is the coefficient for the angle of the embankment abutment relative to the approach flow, L_A is the length (m) of the embankment/abutment projected normal to the flow and F_{M1} is the Froude number of the approach flow upstream of the abutment.

- ✓ In this case $F_{M1} = V_{M1}/(gY_{M1})^{1/2}$

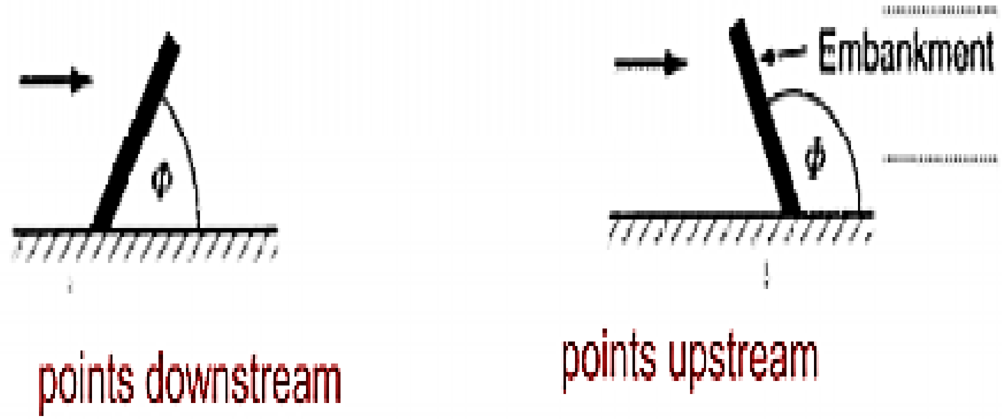
Where, V_{M1} is the mean velocity (m/s) on the floodplain, which is calculated as $V_{M1} = Q_A/A_A$

Where, Q_A is the approach flow (m^3/s) obstructed by the embankment abutment and ($A_A = L_A Y_{M1}$) is the flow area (m^2) of the approach cross section obstructed by the embankment abutment.

Table for K1A: coefficients for abutment type

Abutment type	K_{1A}
Vertical-wall abutment	1.00
Vertical-wall abutment with wingwalls	0.82
Spillthrough abutment	0.55

✓ Note that with respect to the angle of the approach flow, $\phi < 90^o$ if the embankment abutment points downstream and $\phi > 90^o$ if the embankment abutment points upstream.



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Total scour depth (d_s)

- ✓ Degradation, contraction and local scour are additive, but only where the scour holes overlap.
- ✓ For instance, contraction scour may have to be added to pier or abutment scour to get the total scour depth. However, pier scour and abutment scour would not be added unless the two scour holes overlap.

$$\text{total scour } (d_S) = \text{degradation } (\Delta d) + \text{contraction scour } (d_{SC}) + \text{local scour } (d_{SL})$$

Example 3.3

Just upstream of a 0.9m wide round nosed pier the depth of flow is 1.3m with a velocity of 1.6m/s. The pier is skewed to the approach flow with ø = 15^{o}. The length of the waterway in the direction of flow (L) is 14.4m. Assume that the channel bed is plane. Calculate the local pier scour depth.

Example 3.4

The longitudinal Centre line of an embankment leading to a bridge abutment is skewed at an angle of 30^{o} compared with a perpendicular crossing. The length of the embankment/abutment is 33m measured along the Centre line. The abutments are of the vertical wall type. It is estimated that the mean depth on the upstream floodplain is 1.2m with a mean velocity of 0.7m/s. Calculate the maximum abutment scour depth.

References

1. L Hamill-Bridge hydraulics-E. & F.N. Spon.1999.
2. Larry W Mays Hydraulic design handbook.1999.
3. Hydraulic Structures, P. Novak. 2004.

THE END

MANY THANKS!!!