Hydraulic Structures

Fourth Class

College of Eng.-University of Anbar

Lecture No.(1)

Prof.Ayad Sleibi Mustafa ,Ph.D

Chapter One
Introduction of hydraulic Structures

1-1 Definition:

Hydraulic structures are devices used to control, regulate and measure flow. Some are fixed geometrical forms, while others may be mechanically adjusted. Hydraulic structures are also referred to as "channel control structures".

1-2 Components of Hydraulic Structures:

Hydraulic Structures mainly consists of converging transition where subcritical flowing water is accelerated and guided into the throat (control section or waterway) without flow separation. A throat where it accelerates to supercritical flow so that the discharge is controlled and stilling basin where the surplus energy is dissipated, and a diverging transition where the flow velocity reduced to an acceptable subcritical velocity and potential energy is recovered.

1-3 Importance of Hydraulic Structures:

Water is a valuable commodity and hence we have to conserve every droplet of this water. To do so a number of structures need to be used. As water is very important for life to exist, water can be the cause of destruction for nations and community. Hence other structures need to be used for the safety of the human beings. We can summarize the importance of Hydraulic Structures in the following points: -

1-Continuity of life on the earth by providing the required amount of drinking, agriculture, industry, .. etc.
2-Safety and security of people, factories, farms, cities,etc.
3-Justice through providing equality of water distribution to homes, factories, farmers, countries.
4-To provide easy access for traffic movements.
5-Providing extra jobs by fishing, navigation,…etc.
6-Provide clean energy sources.

1-4 Classification of Hydraulic structures according to use:

1- Diversion structures:

Barrage or diversion weirs are usually constructed in rivers and large channels to facilities diversion of flow into canals for irrigation, power generation and improvement of navigation. These are achieved by raising the water levels on their upstream side.

2- Control and Regulating structures:

They are used in open channels systems to control and regulate the discharge and/or the water levels within the system. Structures which control discharges are called “Head Regulator” or offtakes or intakes or sometimes outlets while structures controlling water levels are called “Cross Regulators”

3- Conveyance Structures:

They are structures constructed in the canals used to convey water safely from one location to another across various exiting natural or manmade obstacles and topographic features along the canal route. They are different types as follows:

- Road crossing(culverts)
- Railway crossing(culverts)
- Inverted syphons
- Aqueducts
- Drops and chutes
-

4- Protective(safety) Structures:

They are used to protect the canal system and adjacent property from damages which result from uncontrolled excess of flow within the canal which leads to overtopping due to faults in operation. They are called “Escape or waterway structures”. They are of two main types:

- Side escape structures
- End or tail escape structures

Escape structures are usually equipped with fixed crest weirs allow the spilling of the extra discharge and sometimes equipped with gated part used for emptying the canal for purposes of inspection and maintenance.

5- Special structures:

They are structures consisted for many purposes. They are a combination of different structures. For examples, a conveyance structures when gated, it can be used as a control structure well it can be used to control both discharge and water levels (gated inverted syphons, gated culverts, and gated drop or chute structures).

6-Flow measurement structures:

They are structures constructed on irrigation system for the purpose of water management to ensure equitable water distribution to users and to prevent unnecessary wasteful water management practices thereby enhancing the conservation of water. Water measurement is made by using weirs, flumes and constant head orifice structures. In other words, by providing a control section which have a unique relationship between head and discharge (i.e. critical flow) within the channel.

Table (1) summarizes the classification of hydraulic structures according to Gupta (1989).

1-5 General design consideration:

Any hydraulic structures should be checked and designed for the following conditions:

1- Surface flow;
2- Subsurface flow (under- seepage)

The surface flow design consists of the following aspects:

- Waterway (control section or throut)
- Energy dissipation (stilling basin)
- Protection works (anti-scour arrangements)

The subsurface flow or under-seepage design consists of the following aspects:

1-Estimation of the rate of the seepage.
2-Line of the rate of the seepage.
3-Uplift pressures under the structure.
4-Seepage exit gradient.
5-Downstream heave.

Table (1): **Classification of hydraulic structures** (Source: Gupta 1989)

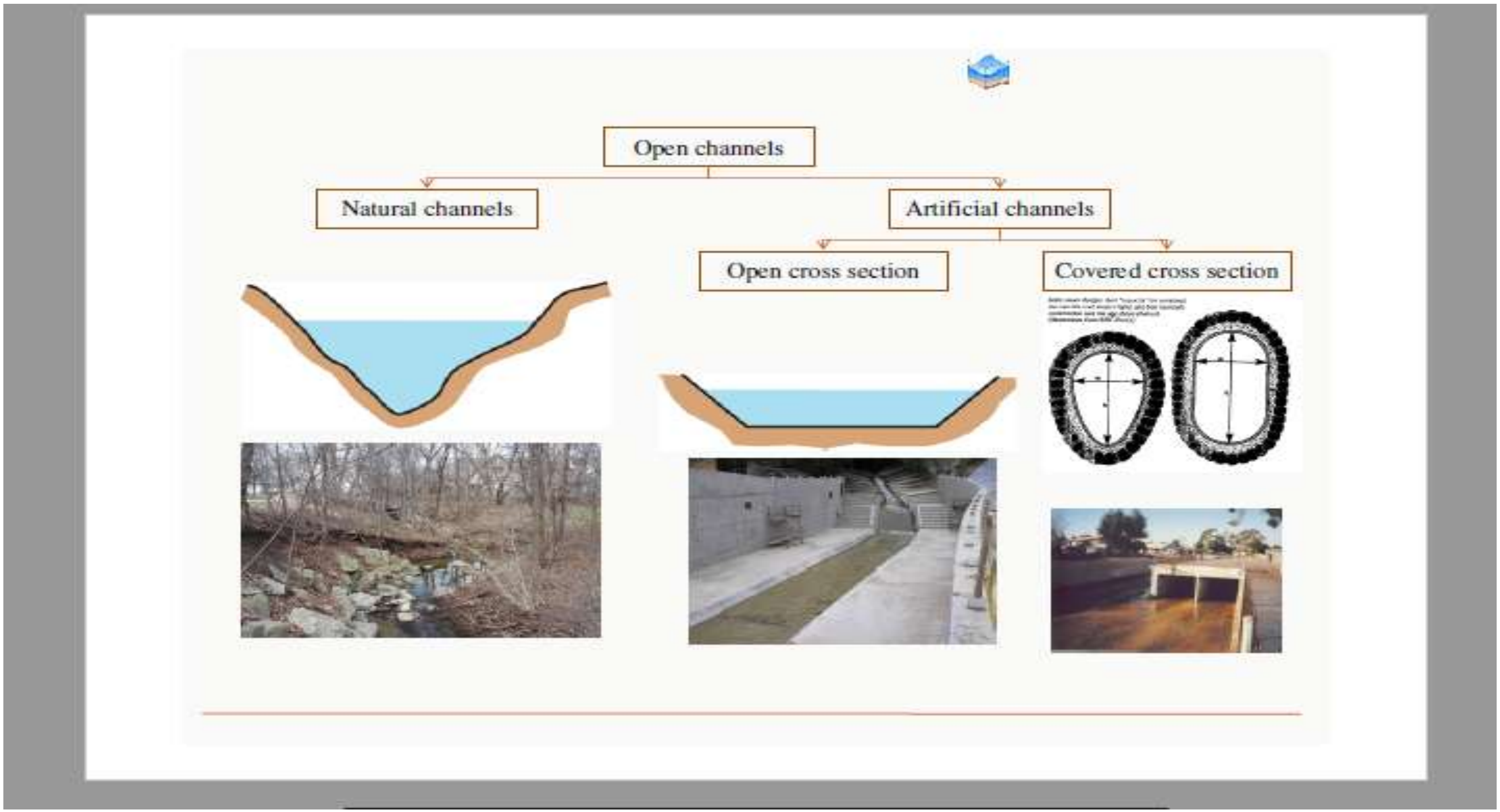
Type	Purpose	Structures
Storage structures	To store water	Dams, tanks
Flow control structures	To regulate the quantity and pass excess flow	Spillways, outlets, gates, valves
Flow measurement structures	To determine discharge	Weirs, orifices, flumes
Diversion structures	To divert the main course of water	Coffer dam, weirs, canal headwork, intake works
Conveyance structures	To guide flow from one location to another	Open channels, pressure conduits, pipes, canals, sewers
Collection structures	To collect water for disposal	Drain inlets, infiltration galleries, wells
Energy dissipation structures	To prevent erosion and structural damage	Stilling basins, surge tanks, check dams
Shore protection structures	To protect the banks	Dikes, groins, jetties, revetments, breakwaters, seawalls
River training and waterway stabilization structures	To maintain a river channel and water transportation	Levees, cutoffs, locks, piers culverts
Sediment and quality control structures	To control or remove sediments and other pollutions	Racks, screens, traps, sedimentation tanks, filters, sluiceways
Hydraulic machines	To convert energy from one form to other	Pumps, turbines, rams

Open Channel Flow
Lecture (2)

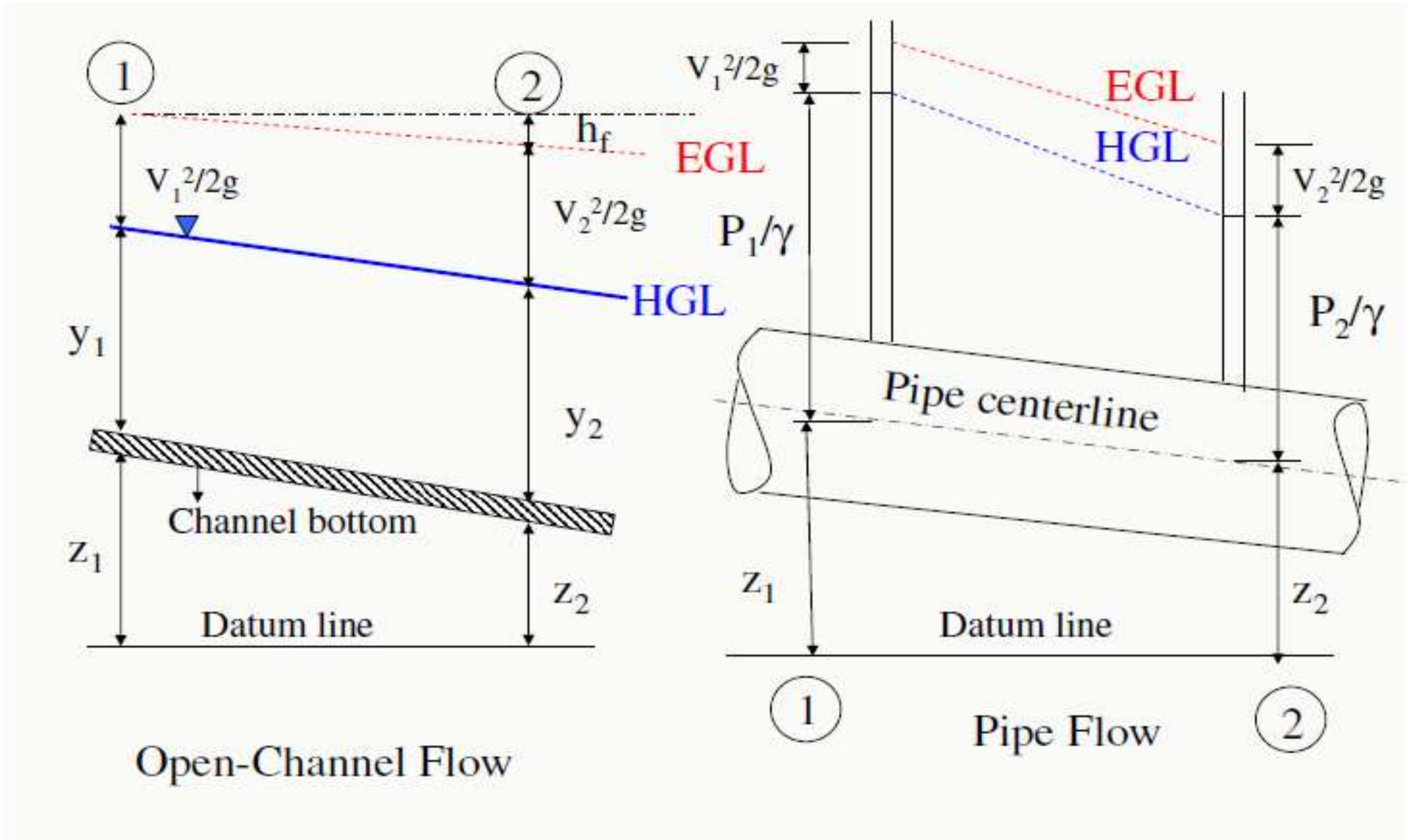
- Liquid (water) flow with a free surface (interface between water and air)

- relevant for
  - natural channels: rivers, streams
  - engineered channels: canals, sewer lines or culverts (partially full), storm drains
- of interest to hydraulic engineers
  - location of free surface
  - velocity distribution
  - discharge - stage (depth) relationships
  - optimal channel design

Classification of Open Channel Flows



Comparison of Open Channel Flow and Pipe Flow



Comparisons of Open Channel Flow and Pipe Flow

1) OCF must have a free surface	1) No free surface in pipe flow
2) A free surface is subject to atmospheric pressure	2) No direct atmospheric pressure, hydraulic pressure only.
3) The driving force is mainly the component of gravity along the flow direction.	3) The driving force is mainly the pressure force along the flow direction.
4) HGL is coincident with the free surface.	4) HGL is (usually) above the conduit
5) Flow area is determined by the geometry of the channel plus the level of free surface, which is likely to change along the flow direction and with as well as time.	5) Flow area is fixed by the pipe dimensions The cross section of a pipe is usually circular..

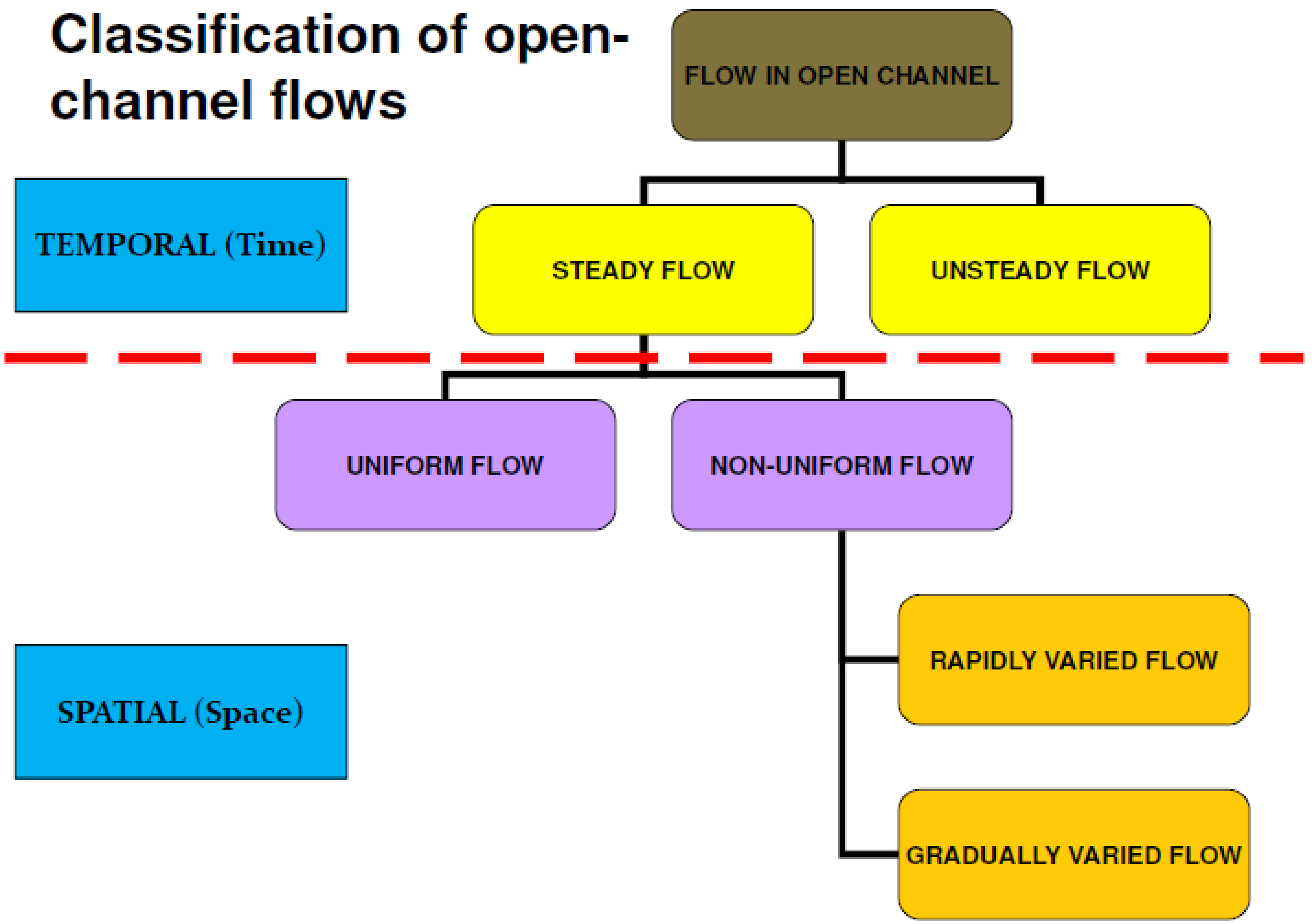
Topics in Open Channel Flow

- Uniform Flow Normal depth
  - Discharge-Depth relationships
- Channel transitions
  - Control structures (sluice gates, weirs…)
  - Rapid changes in bottom elevation or cross section
- Critical, Subcritical and Supercritical Flow
- Hydraulic Jump
- Gradually Varied Flow
  - Classification of flows
  - Surface profiles

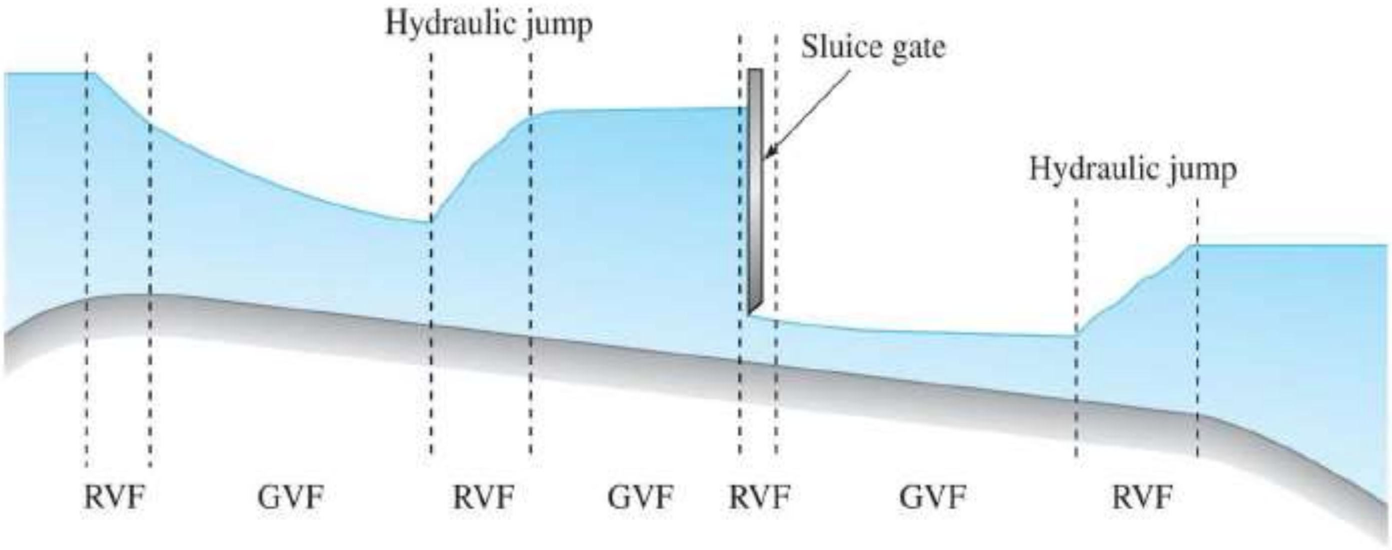
Classification of Flows

- Steady and Unsteady
  - Steady: velocity at a given point does not change with time
- Uniform, Gradually Varied, and Rapidly Varied (Spatial)
  - Uniform: velocity at a given time does not change within a given length of a channel
  - Gradually varied: gradual changes in velocity with distance
- Laminar and Turbulent
  - Laminar: flow appears to be as a movement of thin layers on top of each other
  - Turbulent: packets of liquid move in irregular paths

Classification of Flows



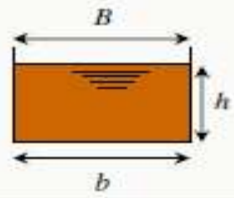
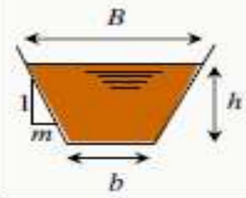
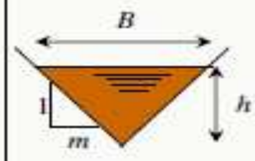
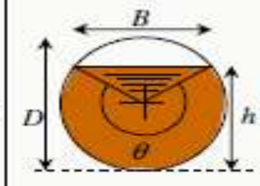
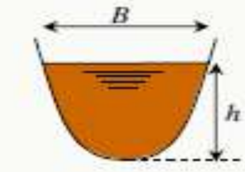
Classification of Flows



Momentum and Energy Equations

- Conservation of Energy
  - "losses" due to conversion of turbulence to heat
  - useful when energy losses are known or small
    - Contractions
  - Must account for losses if applied over long distances
    - We need an equation for losses
- Conservation of Momentum
  - "losses" due to shear at the boundaries
  - useful when energy losses are unknown
    - Expansion

Geometric elements for different channel cross sections

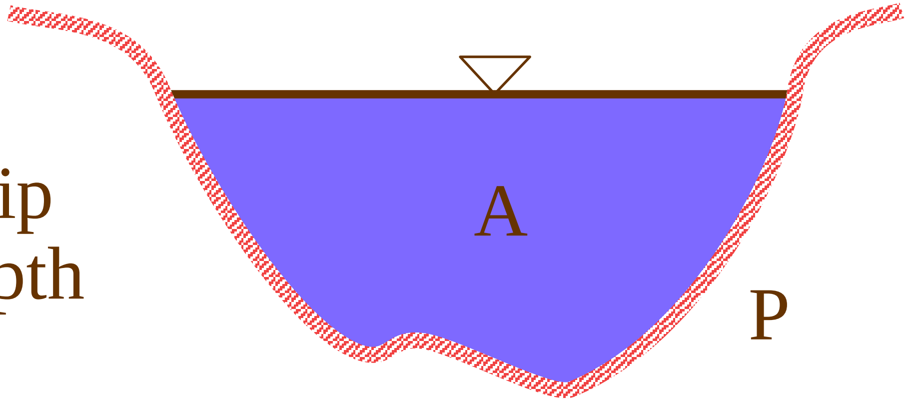
	rectangular	trapezoidal	triangular	circular	parabolic
					
flow area A	bh	$(b+mh)h$	mh^2	$\frac{1}{8}(\theta-\sin\theta)D^2$	$\frac{2}{3}Bh$
wetted perimeter P	$b+2h$	$b+2h\sqrt{1+m^2}$	$2h\sqrt{1+m^2}$	$\frac{1}{2}\theta D$	$B+\frac{8}{3}\frac{h^2}{B}$ *
hydraulic radius R_h	$\frac{bh}{b+2h}$	$\frac{(b+mh)h}{b+2h\sqrt{1+m^2}}$	$\frac{mh}{2\sqrt{1+m^2}}$	$\frac{1}{4}\left[1-\frac{\sin\theta}{\theta}\right]D$	$\frac{2B^2h}{3B^2+8h^2}$ *
top width B	b	$b+2mh$	$2mh$	$(\sin\theta/2)D$ *or* $2\sqrt{h(D-h)}$	$\frac{3}{2}Ah$
hydraulic depth D_h	h	$\frac{(b+mh)h}{b+2mh}$	$\frac{1}{2}h$	$\left[\frac{\theta-\sin\theta}{\sin\theta/2}\right]\frac{D}{8}$	$\frac{2}{3}h$

* *Valid for* $0<\xi\leq 1$ *where* $\xi=4h/B$

If $\xi>1$ *then* $P=(B/2)\left[\sqrt{1+\xi^2}+(1/\xi)\ln\left(\xi+\sqrt{1+\xi^2}\right)\right]$

Open Channel Flow: Discharge/Depth Relationship

- Given a long channel of constant slope and cross section find the relationship between discharge and depth



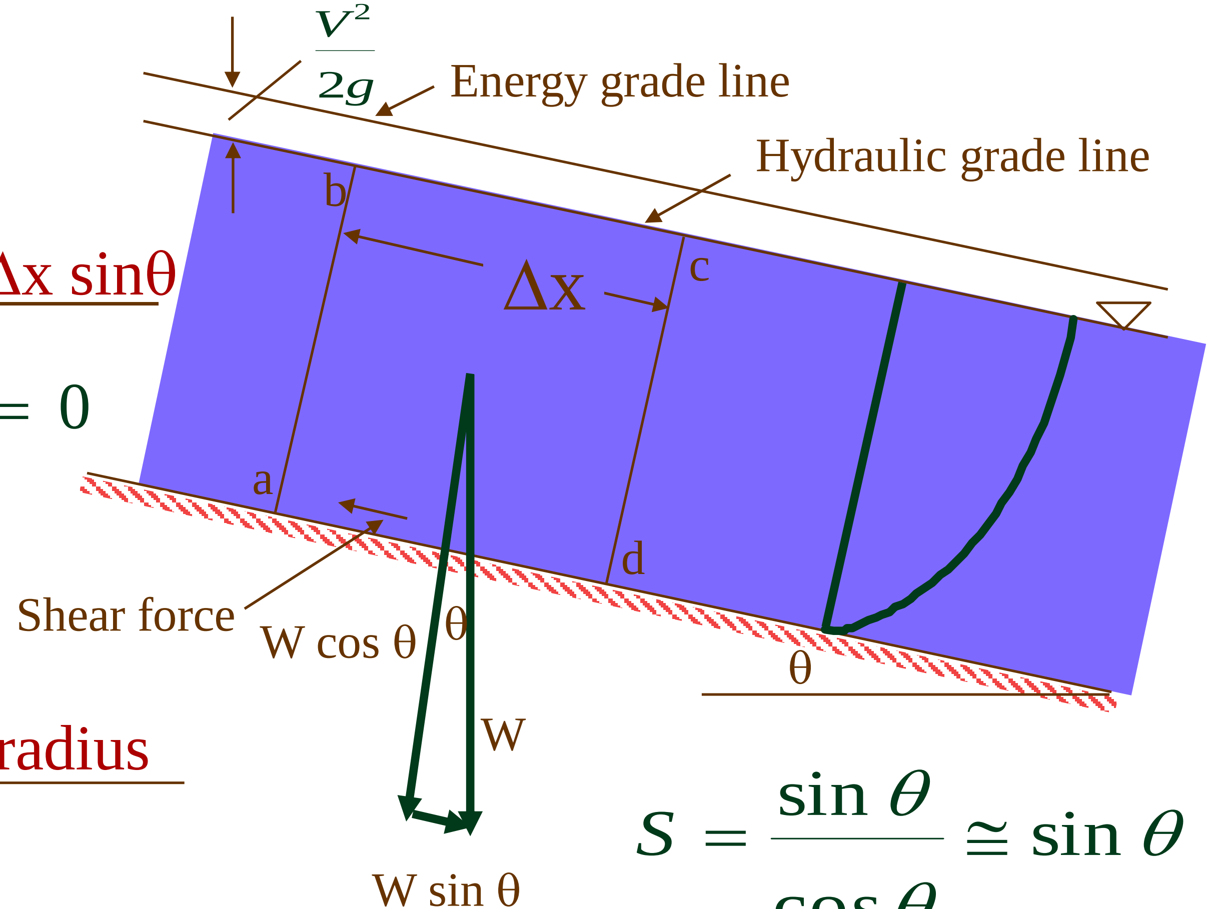
- Assume
  - Steady Uniform Flow - no acceleration
  - prismatic channel (no change in geometry with distance)
- Use Energy, Momentum, Empirical or Dimensional Analysis?

Steady-Uniform Flow: Force Balance

Shear force = $\tau_o P \Delta x$

Wetted perimeter = P

Gravitational force = $\gamma A \Delta x \sin\theta$



$$\gamma A \Delta x \sin\theta - \tau_o P \Delta x = 0$$

$$\tau_o = \gamma \frac{A}{P} \sin\theta$$

$$\frac{A}{P} = R_h$$ Hydraulic radius

$$t_o = g R_h S$$

$$S = \frac{\sin\theta}{\cos\theta} \cong \sin\theta$$

Relationship between shear and velocity? Turbulence

Chezy Equation (1768)

- Introduced by the French engineer Antoine Chezy in 1768 while designing a canal for the water-supply system of Paris

$$V = C\sqrt{R_h S_f} \qquad \text{compare} \qquad V = \sqrt{\frac{2g}{f}}\sqrt{S_f R_h}$$

where C = Chezy coefficient

$$60\frac{\sqrt{m}}{s} < C < 150\frac{\sqrt{m}}{s}$$

$$0.0054 > f > 0.00087$$

$$0.022 > f > 0.0035$$

For a pipe

$$d = 4R_h$$

where 60 is for rough and 150 is for smooth

also a function of **R** (like f in Darcy-Weisbach)

Darcy-Weisbach Equation (1840)

f = Darcy-Weisbach friction factor

$$h_l = \text{f}\,\frac{l}{d}\frac{V^2}{2g} \longrightarrow h_l = \text{f}\,\frac{l}{4R_h}\frac{V^2}{2g}$$

$$R_h = \frac{A}{\text{P}} = \frac{\left(\dfrac{\pi d^2}{4}\right)}{\pi d} = \frac{d}{4}$$

$$S_f\, l = \text{f}\,\frac{l}{4R_h}\frac{V^2}{2g} \longrightarrow S_f R_h = \text{f}\,\frac{V^2}{8g} \longrightarrow V = \sqrt{\frac{8g}{\text{f}}}\sqrt{S_f R_h}$$

Manning Equation (1891)

- Most popular in U.S. for open channels

$$V = \frac{1}{n} R_h^{2/3} S_o^{1/2}$$

Dimensions of *n*? $T/L^{1/3}$

Is *n* only a function of roughness? ______

$$V = \frac{1.49}{n} R_h^{2/3} S_o^{1/2}$$

(English system)

Bottom slope

$$Q = VA$$

$$Q = \frac{1}{n} A R_h^{2/3} S_o^{1/2}$$

very sensitive to *n*

Values of Manning (n)

Lined Canals	n
Cement plaster	0.011
Untreated gunite	0.016
Wood, planed	0.012
Wood, unplaned	0.013
Concrete, trowled	0.012
Concrete, wood forms, unfinished	0.015
Rubble in cement	0.020
Asphalt, smooth	0.013
Asphalt, rough	0.016
Natural Channels	
Gravel beds, straight	0.025
Gravel beds plus large boulders	0.040
Earth, straight, with some grass	0.026
Earth, winding, no vegetation	0.030
Earth , winding with vegetation	0.050

n = f(surface roughness, channel irregularity, stage...)

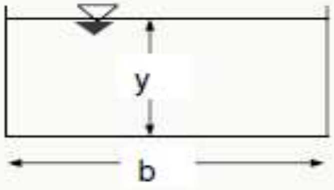
$n = 0.031 d^{1/6}$ d in ft

$n = 0.038 d^{1/6}$ d in m

d = median size of bed material

Example (1):Rectangular Channel

What are the most efficient dimensions (the best hydraulic section) for a concrete (n=0.012) rectangular channel to carry 3.5 m³/s at So=0.0006?



Given: n=0.012 Q=3.5 m³/s S_o=0.0006
Find b and y.

$$Q = \frac{A}{n} R^{2/3} S_o^{1/2} \qquad Q = \frac{A}{n}\left(\frac{A}{P}\right)^{2/3} S_o^{1/2} \qquad Q = \frac{by}{n}\left(\frac{by}{b+2y}\right)^{2/3} S_o^{1/2}$$

best section
b=2y

$$Q = \frac{2y*y}{n}\left(\frac{2y*y}{2y+2y}\right)^{2/3} S_o^{1/2} \qquad Q = \frac{2y^2}{n}\left(\frac{y}{2}\right)^{2/3} S_o^{1/2}$$

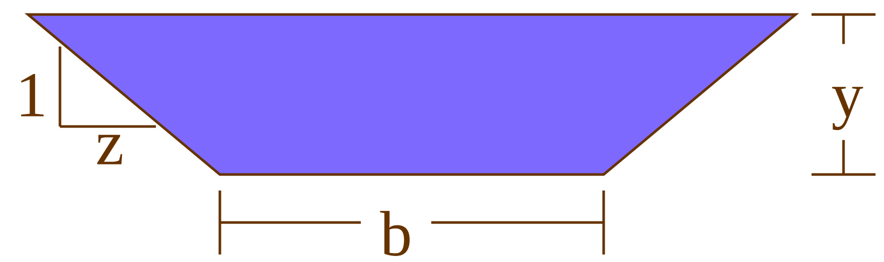
$$Q = \frac{2^{1/3}}{n}(y)^{8/3} S_o^{1/2} \qquad (y)^{8/3} = 1.36$$

y = 1.123 m and b = 2.246m

Trapezoidal Channel

- Derive P = f(y) and A = f(y) for a trapezoidal channel
- How would you obtain y = f(Q)?

$$Q = \frac{1}{n} A R_h^{2/3} S_o^{1/2}$$

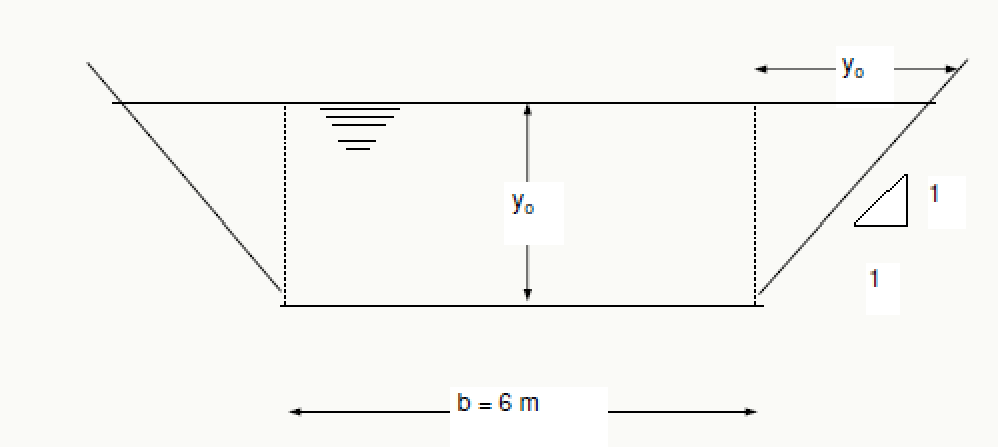


Example (2)

A trapezoidal channel has a base width b = 6 m and side slopes 1H:1V.
The channel bottom slope is So = 0.0002 and the Manning roughness coefficient is n = 0.014.
Compute

a) the depth of uniform flow if Q = 12.1 m3/s

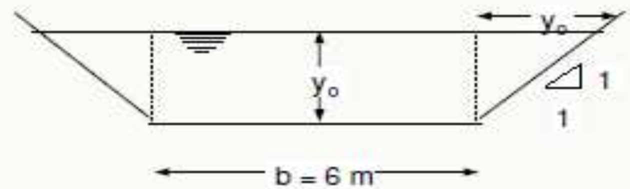
b) the state of flow



Solution of Example (2)

Solution of Example 1

a) Manning's equation is used for uniform flow;



$$Q = \frac{A}{n} R^{2/3} \sqrt{S_o}$$

$$A = b.y_o + 2.(y_o^{\,2}/2) = y_o(b + y_o\ )$$

$$P = b + 2\sqrt{2}\ y_o = 6 + 2\sqrt{2}\ y_o$$

So = 0.0002 n = 0.014 Q = 12.1 m3/s

$$AR^{2/3} = \frac{Qn}{\sqrt{S_o}} = 11.98$$

$$11.98 = y_o(6 + y_o)\left(\frac{y_o(6 + y_o)}{6 + 2\sqrt{2}y_o}\right)^{2/3}$$

Y(m)	A(m²)	P(m)	R(m)	$AR^{2/3}$
1	7	8.28	0.84	6.23
1.2	8.64	9.39	0.92	8.17
1.4	10.36	9.96	1.04	10.63
1.5	11.25	10.24	1.098	11.976

by trial & error y_o=1.5 m

Solution of Example (2)

b) The state of flow

$$Fr = \frac{V_{ave}}{\sqrt{gD}} \quad , \; D = \frac{A}{T} \quad , \quad T = b + 2y_o$$

$A = 1.5\ (6+1.5) = 11.25\ m^2$

$T = 6+2 \times 1.5 = 9\ m$

$D = 11.25 / 9 = 1.25\ m$

$$V_{ave} = \frac{Q}{A} = \frac{12.1}{11.25} = 1.076\, m/s$$

$$Fr = \frac{1.076}{\sqrt{9.81 x 1.25}} = 0.307 < 1 \ \underline{\text{ Subcritical}}$$

Flow in Round Conduits

$$\theta = \arccos\left(\frac{r-y}{r}\right)$$

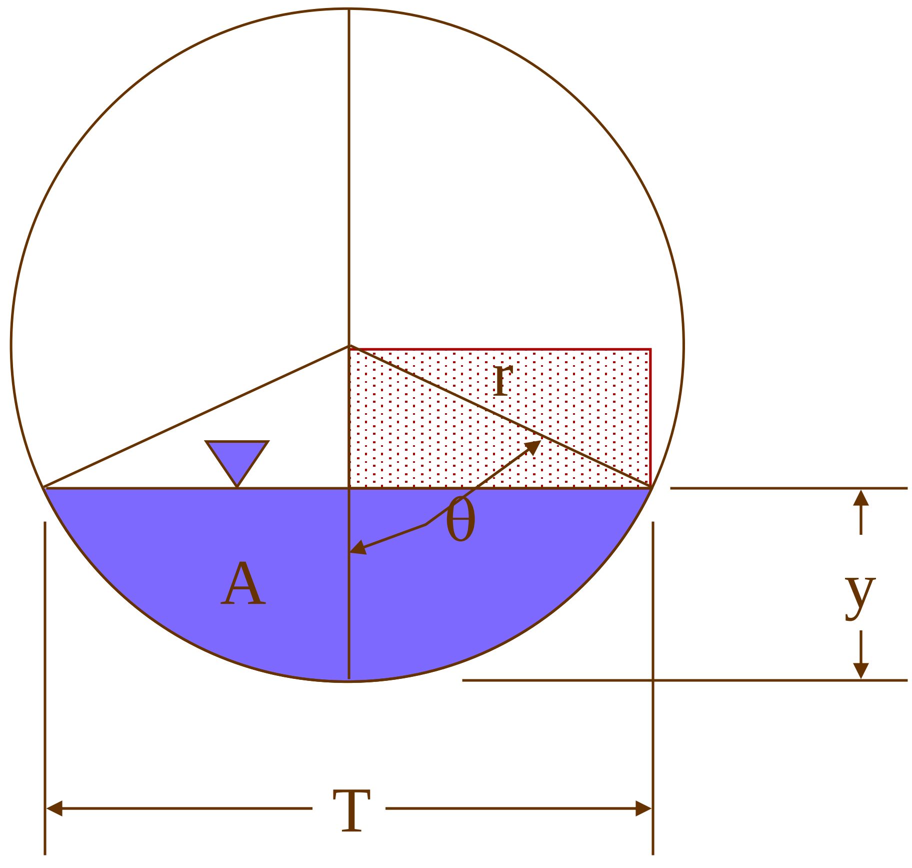
radians

$$A = r^2(\theta - \sin\theta\cos\theta)$$

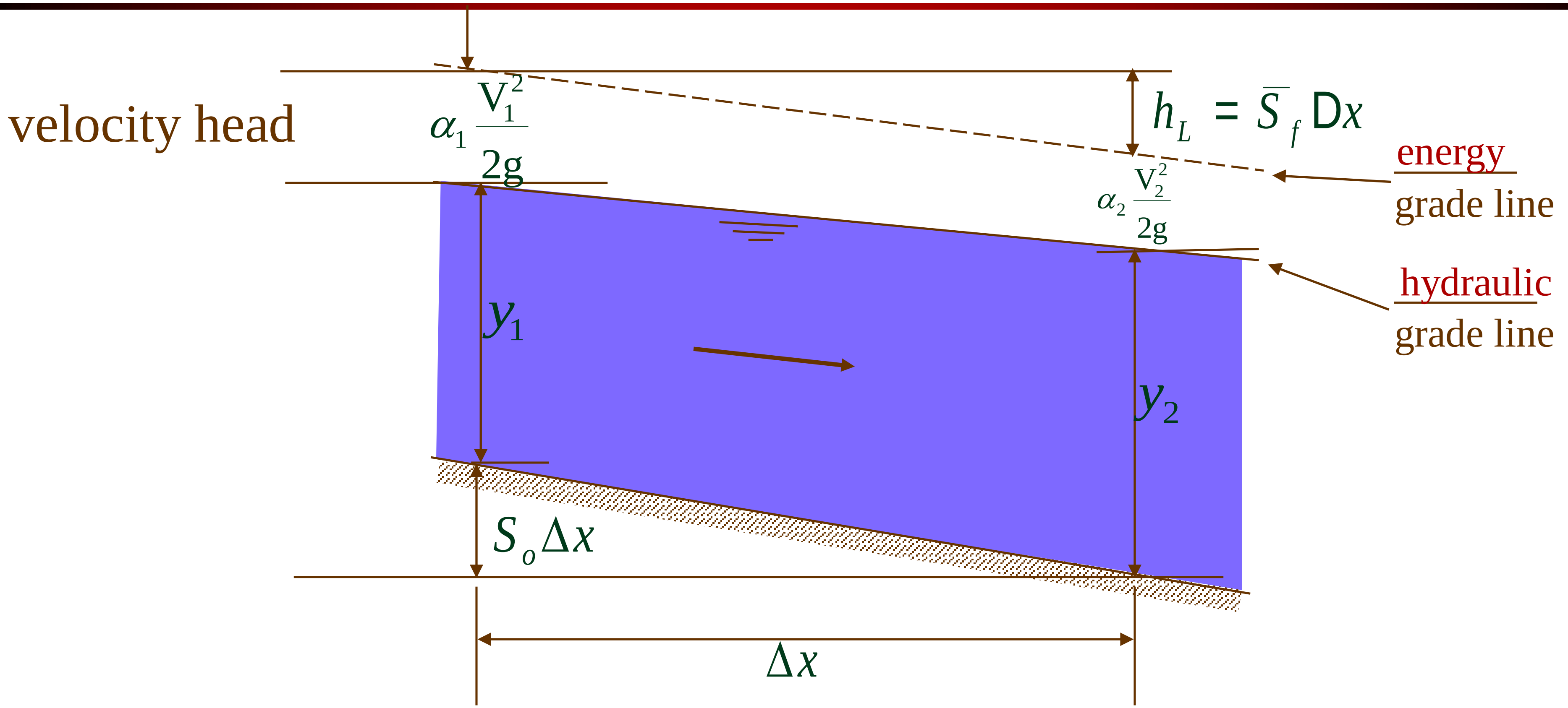
$$T = 2r\sin\theta$$

$$P = 2r\theta$$

Maximum discharge when y = 0.938d



Open Channel Flow: Energy Relations



Bottom slope (S_o) not necessarily equal to EGL slope (S_f)

Energy Relationships

$$\frac{p_1}{\gamma} + z_1 + \alpha_1 \frac{V_1^2}{2g} = \frac{p_2}{\gamma} + z_2 + \alpha_2 \frac{V_2^2}{2g} + h_L$$

Pipe flow

z - measured from horizontal datum

From diagram on previous slide...

$$y_1 + S_o \Delta x + \frac{V_1^2}{2g} = y_2 + \frac{V_2^2}{2g} + S_f \Delta x$$

Turbulent flow ($\alpha \cong 1$)

y - depth of flow

Energy Equation for Open Channel Flow

$$y_1 + \frac{V_1^2}{2g} + S_o \Delta x = y_2 + \frac{V_2^2}{2g} + S_f \Delta x$$

Specific Energy

- The sum of the depth of flow and the velocity head is the specific energy:

$$E = y + \frac{V^2}{2g}$$

kinetic

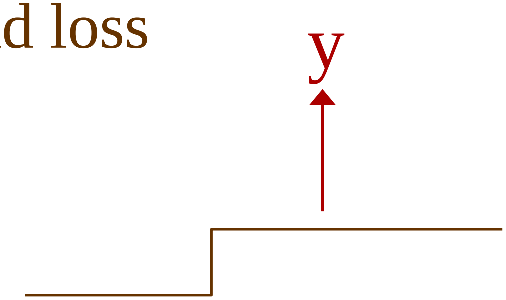
$$E_1 + S_o \Delta x = E_2 + S_f \Delta x$$

If channel bottom is horizontal and no head loss

$$E_1 = E_2$$

For a change in bottom elevation

$$E_1 - \Delta y = E_2$$



Specific Energy

In a channel with constant discharge, Q

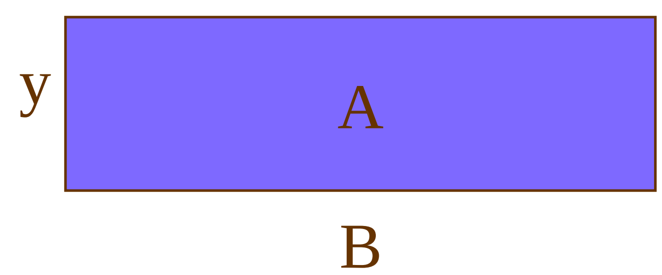
$$Q = A_1 V_1 = A_2 V_2$$

$$E = y + \frac{V^2}{2g} \longrightarrow E = y + \frac{Q^2}{2gA^2}$$ where A=f(y)

Consider rectangular channel (A = By) and Q = qB

$$E = y + \frac{q^2}{2gy^2}$$

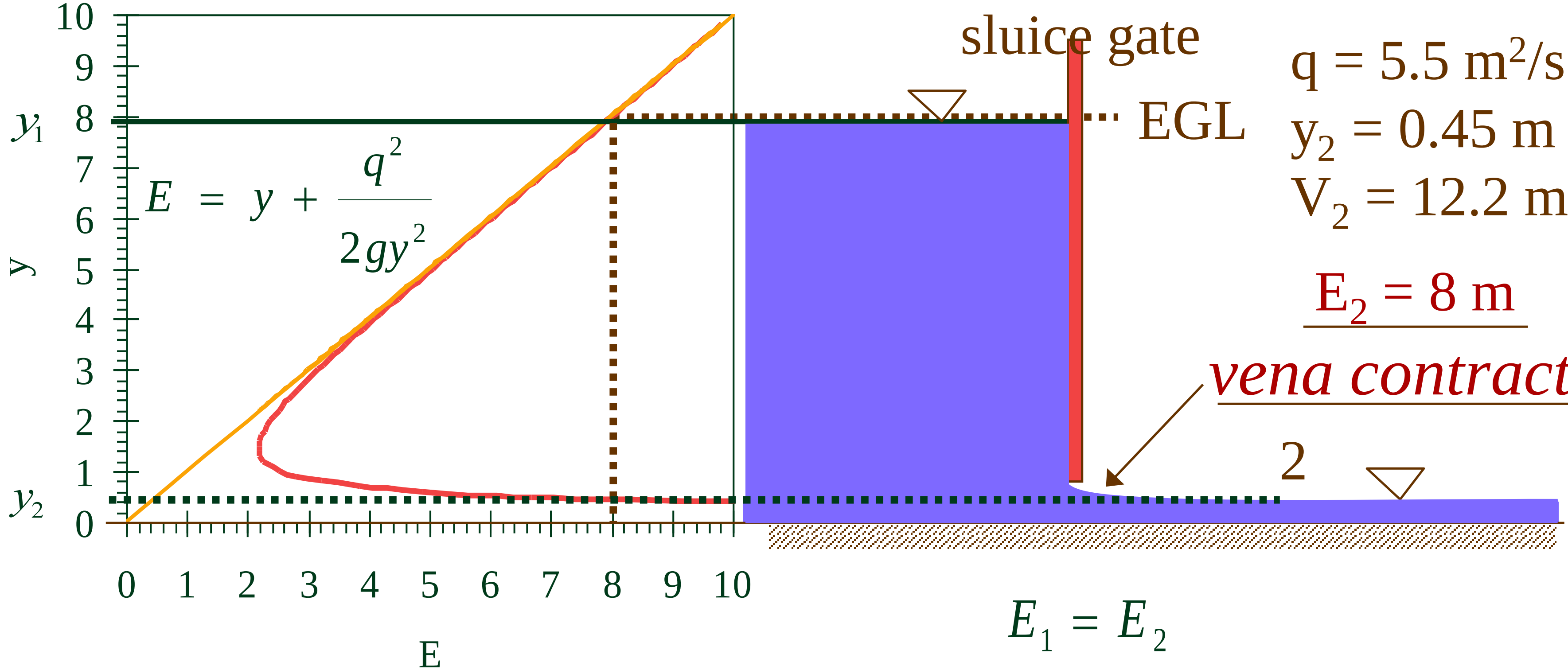
q is the discharge per unit width of channel



3 roots (one is negative)

How many possible depths given a specific energy? __2__

Specific Energy: Sluice Gate



$q = 5.5\ m^2/s$

$y_2 = 0.45\ m$

$V_2 = 12.2\ m/s$

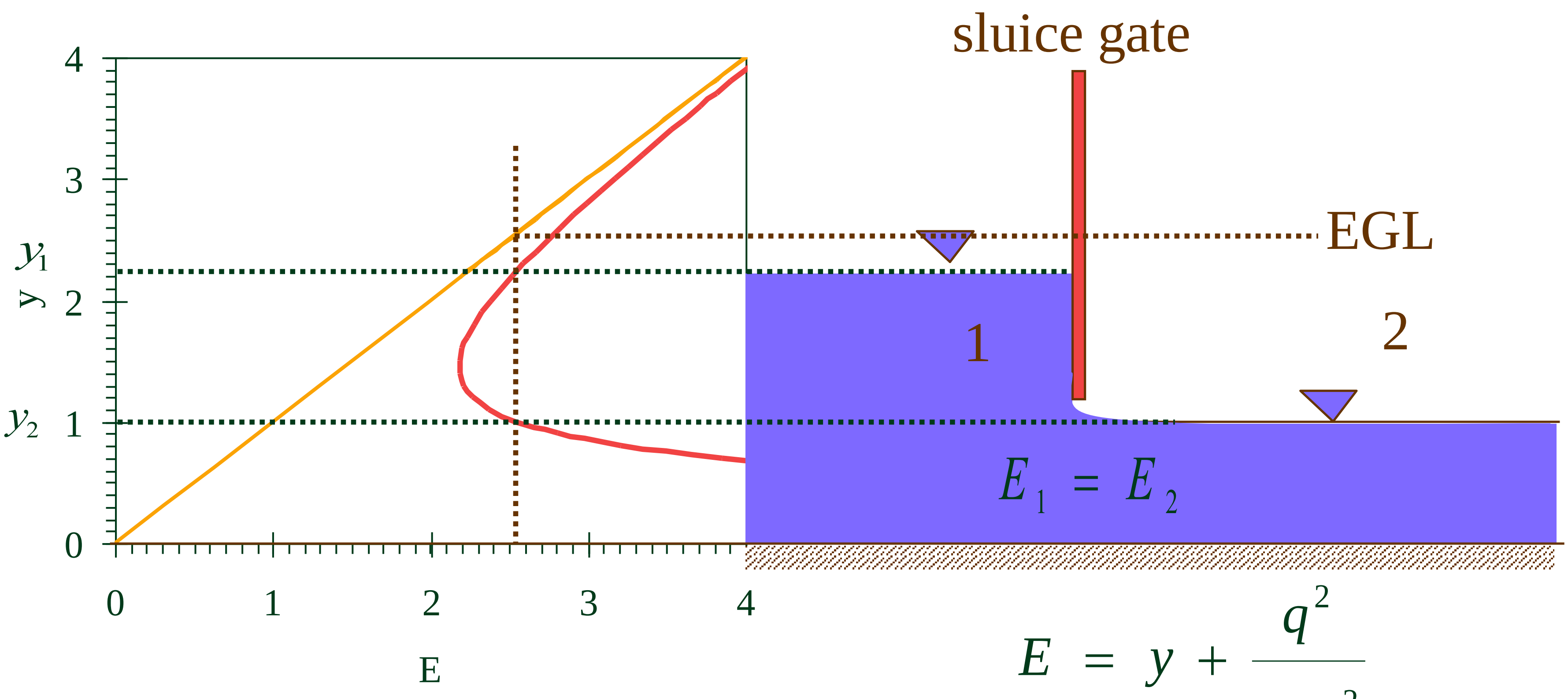
$E_2 = 8\ m$

Given downstream depth and discharge, find upstream depth.

y_1 and y_2 are _alternate__ depths (same specific energy)

Why not use momentum conservation to find y_1?

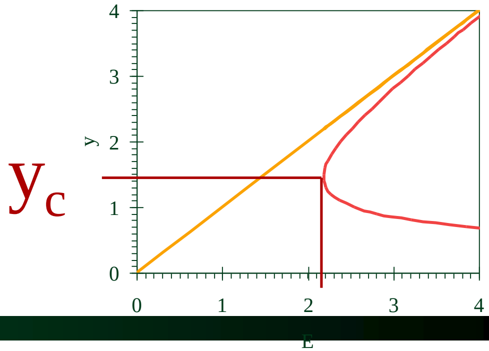
Specific Energy: Raise the Sluice Gate



$$E = y + \frac{q^2}{2gy^2}$$

as sluice gate is raised y_1 approaches y_2 and E is minimized: Maximum discharge for given energy.

Critical Flow



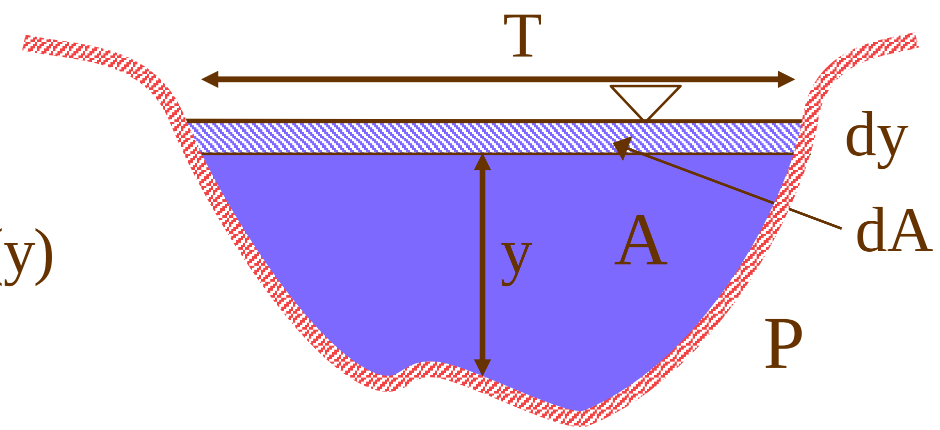
Find critical depth, y_c

Arbitrary cross-section

$$\frac{dE}{dy} = 0$$

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

A=f(y)



$$\frac{dE}{dy} = 1 - \frac{Q^2}{gA^3}\frac{dA}{dy} = 0$$

$dA = Tdy$

T=surface width

More general definition of Fr

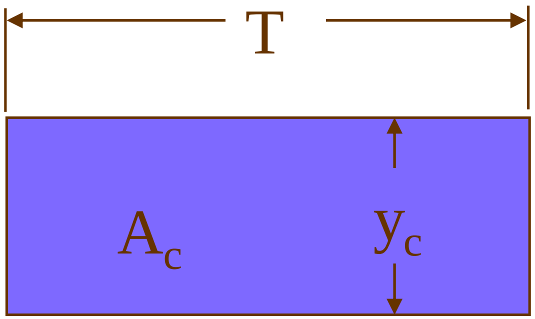
$$1 = \frac{Q^2 T_c}{gA_c^3} \qquad \frac{Q^2 T}{gA^3} = Fr^2 \qquad \frac{V^2 T}{gA} = Fr^2 \qquad \frac{A}{T} = D$$

Hydraulic Depth

Critical Flow: Rectangular channel

$$1 = \frac{Q^2 T_c}{g A_c^3} \qquad T = T_c$$

$$Q = qT \qquad A_c = y_c T$$



$$1 = \frac{q^2 T^3}{g y_c^3 T^3} = \frac{q^2}{g y_c^3}$$

$$y_c = \left(\frac{q^2}{g}\right)^{1/3}$$

Only for rectangular channels!

$$q = \sqrt{g y_c^3}$$

Given the depth we can find the flow!

Critical Flow Relationships: Rectangular Channels

$$y_c = \left(\frac{q^2}{g}\right)^{1/3} \qquad y_c^3 = \left(\frac{V_c^2 y_c^2}{g}\right) \quad \text{because} \quad q = V_c y_c$$

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

Froude number

$$\frac{\text{inertial force}}{\text{gravity force}} \qquad \sqrt{\frac{\textit{Kinetic energy}}{\textit{Potential energy}}}$$

$$y_c = \frac{V_c^2}{g} \longrightarrow \frac{y_c}{2} = \frac{V_c^2}{2g}$$

velocity head = 0.5 (depth)

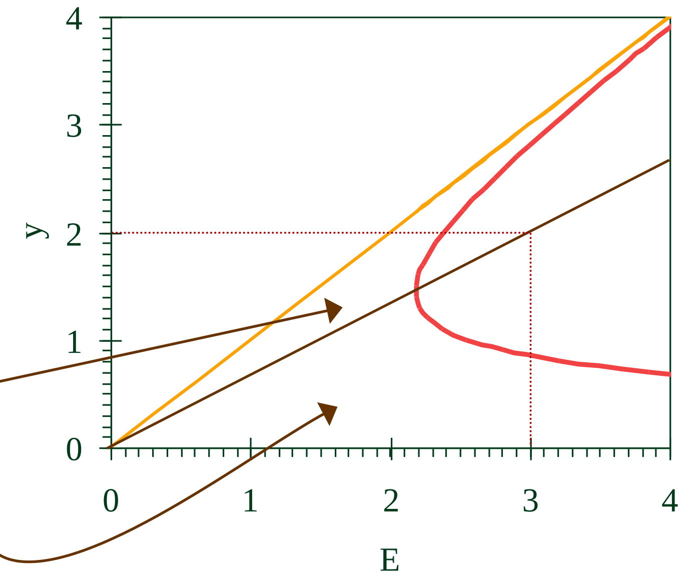
$$E = y + \frac{V^2}{2g} \longrightarrow E = y_c + \frac{y_c}{2} \longrightarrow y_c = \frac{2}{3}E$$

Critical Depth

- Minimum energy for a given q
  - Occurs when $\frac{dE}{dy}$ = 0
  - When kinetic = potential! $\frac{V_c^2}{2g} = \frac{y_c}{2}$
  - Fr=1

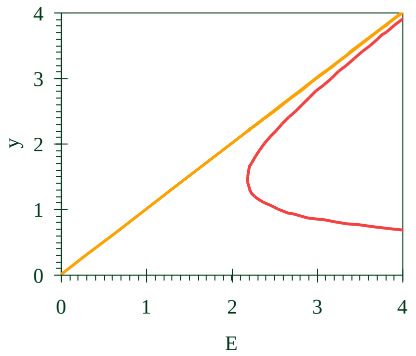
$$Fr = \frac{V_c}{\sqrt{y_c g}} = \frac{q}{\sqrt{g y_c^3}} = Q\sqrt{\frac{T}{gA^3}}$$

  - Fr>1 = supercritical
  - Fr<1 = subcritical



Critical Flow

- Characteristics $\dfrac{dE}{dy} = 0$
  - Unstable surface
  - Series of standing waves



- Occurrence
  - Broad crested weir (and other weirs)
  - Channel Controls (rapid changes in cross-section)
  - Over falls
  - Changes in channel slope from mild to steep
- Used for flow measurements
  - Unique relationship between depth and discharge

Broad-Crested Weir

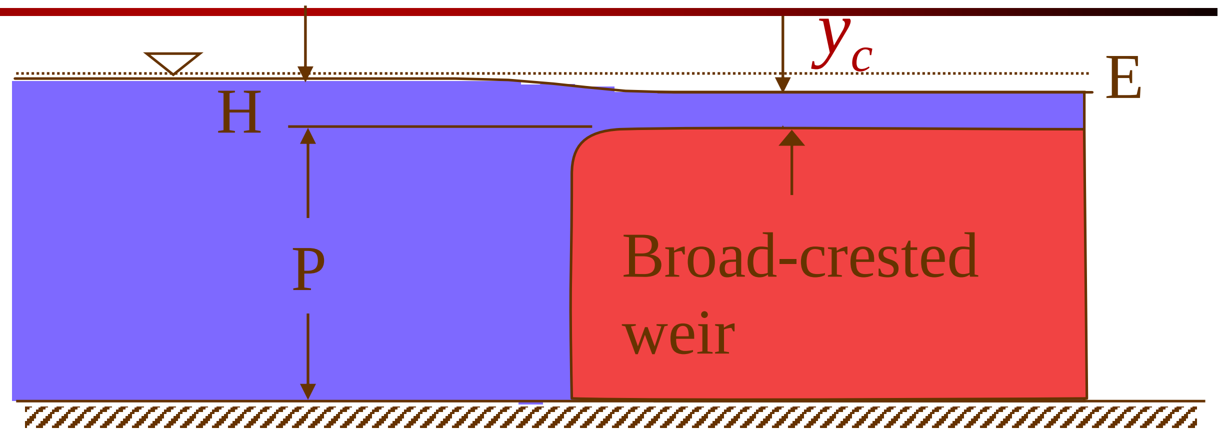
$$y_c = \left(\frac{q^2}{g}\right)^{1/3}$$

$$q = \sqrt{g y_c^3} \qquad Q = b\sqrt{g y_c^3}$$

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

$$Q = b\sqrt{g}\left(\frac{2}{3}\right)^{3/2} E^{3/2}$$

$$Q = C_d b\sqrt{g}\left(\frac{2}{3}H\right)^{3/2}$$



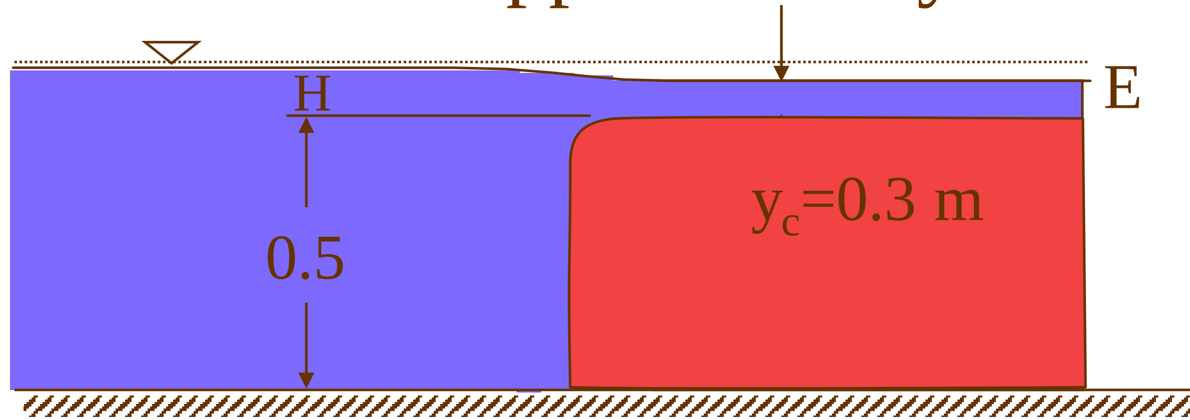
Hard to measure y_c

E measured from top of weir

C_d corrects for using H rather than E.

Broad-crested Weir: Example

- Calculate the flow and the depth upstream. The channel is 3 m wide. Is H approximately equal to E?



How do you find flow? Critical flow relation

How do you find H? Energy equation

Solution

Hydraulic Jump

- Used for energy dissipation
- Occurs when flow transitions from supercritical to subcritical
  - base of spillway
  - Steep slope to mild slope
- We would like to know depth of water downstream from jump as well as the location of the jump
- Which equation, Energy or <u>Momentum</u>?

Classification of Hydraulic Jumps

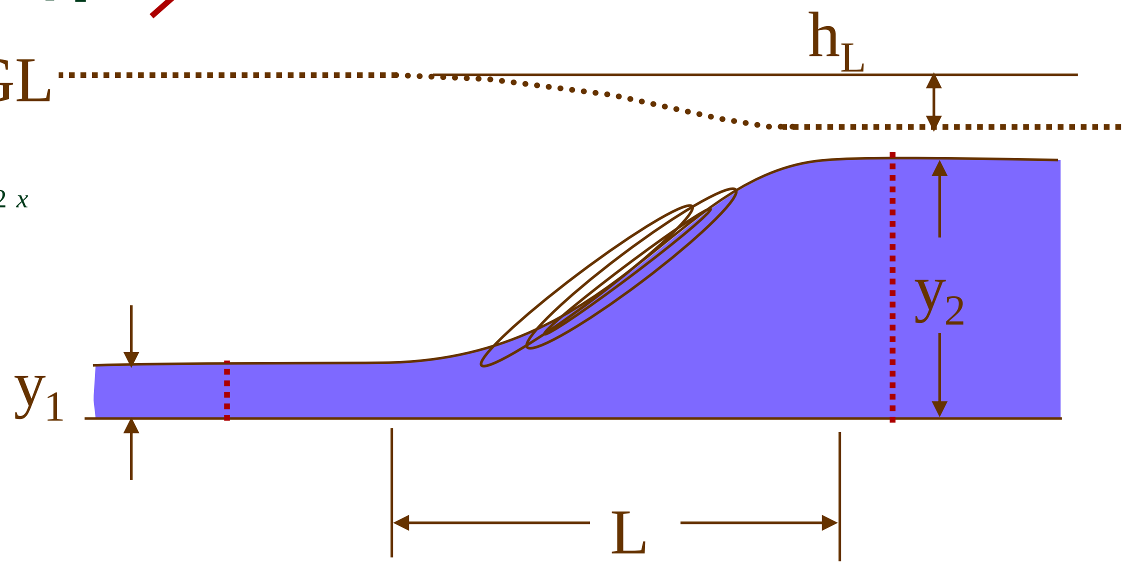
Upstream Fr	Type	Description	
1.0–1.7	Undular	Ruffled or undular water surface; surface rollers form near Fr = 1.7	
1.7–2.5	Weak	Prevailing smooth flow; low energy loss	
2.5–4.5	Oscillating	Intermittent jets from bottom to surface, causing persistent downstream waves	
4.5–9.0	Steady	Stable and well-balanced; energy dissipation contained in main body of jump	
>9.0	Strong	Effective, but with rough, wavy surface downstream	

Source: Adapted with permission from Chow, 1959. (Adapted from Chow, 1959)

Hydraulic Jump

$$\mathbf{M}_1 + \mathbf{M}_2 = \cancel{\mathbf{W}} + \mathbf{F}_{p_1} + \mathbf{F}_{p_2} + \cancel{\mathbf{F}_{ss}}$$

Conservation of Momentum



$$M_{1x} + M_{2x} = F_{p_{1x}} + F_{p_{2x}}$$

$$M_{1x} = -\rho V_1^2 A_1$$

$$M_{2x} = \rho V_2^2 A_2$$

$$-\rho Q V_1 + \rho Q V_2 = \bar{p}_1 A_1 - \bar{p}_2 A_2$$

$$-\frac{Q^2}{A_1} + \frac{Q^2}{A_2} = \frac{g y_1 A_1}{2} - \frac{g y_2 A_2}{2}$$

$$\bar{p} = \frac{r\, g y}{2}$$

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

Hydraulic Jump:Conjugate Depths

For a rectangular channel make the following substitutions

$$A = By \qquad Q = By_1V_1$$

$$Fr_1 = \frac{V_1}{\sqrt{gy_1}}$$ Froude number

Much algebra $\longrightarrow$ $$y_2 = \frac{y_1}{2}\left(-1 + \sqrt{1 + 8Fr_1^2}\right)$$

$$\frac{y_2}{y_1} = \frac{-1 + \sqrt{1 + 8Fr_1^2}}{2}$$

valid for slopes < 0.02

Hydraulic Jump: Energy Loss and Length

- Energy Loss $E_1 = E_2 + h_L$

$$E = y + \frac{q^2}{2gy^2} \xrightarrow{\text{algebra}} h_L = \frac{(y_2 - y_1)^3}{4y_1y_2}$$

significant energy loss (to turbulence) in jump

- Length of jump

No general theoretical solution

Experiments show

$$L = 6y_2 \quad \text{for } 4.5 < Fr_1 < 13$$

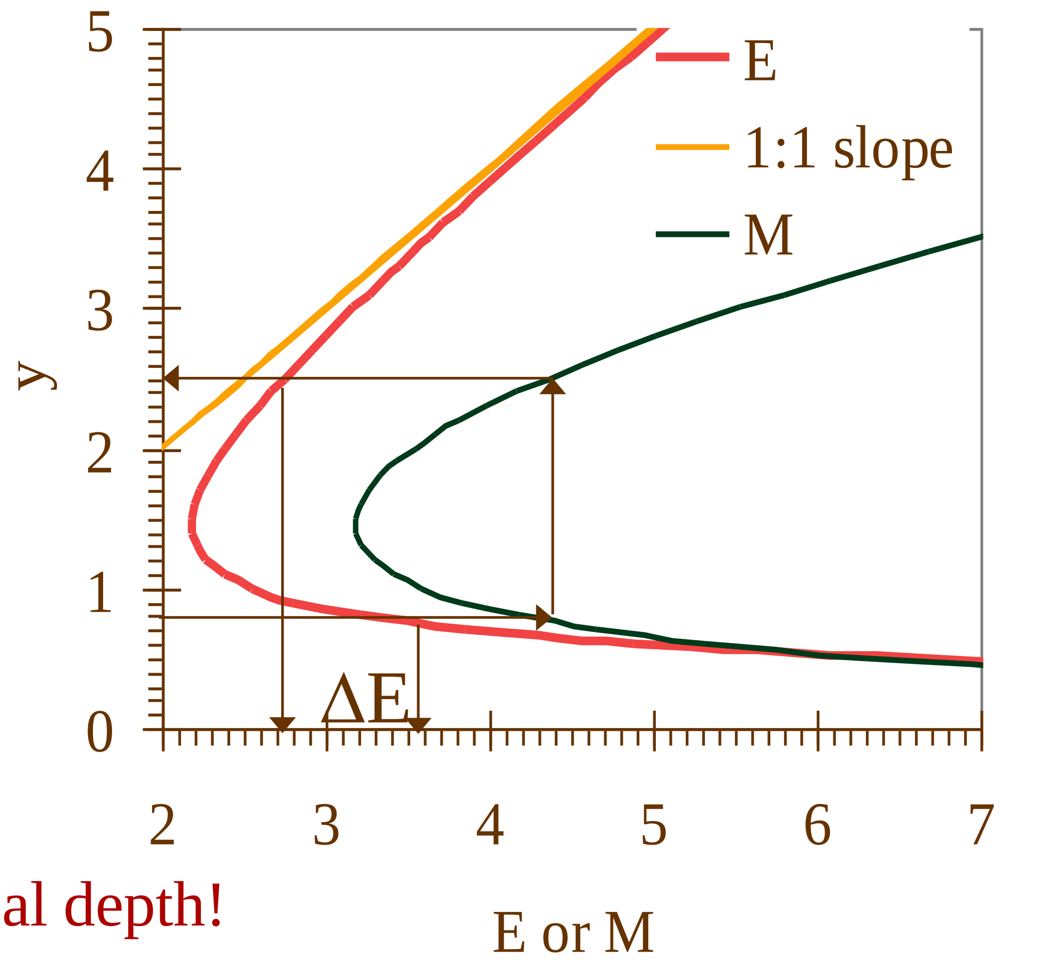
Specific Momentum

$$\frac{gy_1A_1}{2} + \frac{Q^2}{A_1} = \frac{gy_2A_2}{2} + \frac{Q^2}{A_2}$$

$$\frac{y_1A_1}{2} + \frac{Q^2}{A_1g} = \frac{y_2A_2}{2} + \frac{Q^2}{A_2g}$$

$$\frac{y_1^2}{2} + \frac{q^2}{y_1g} = \frac{y_2^2}{2} + \frac{q^2}{y_2g}$$

When is M minimum?

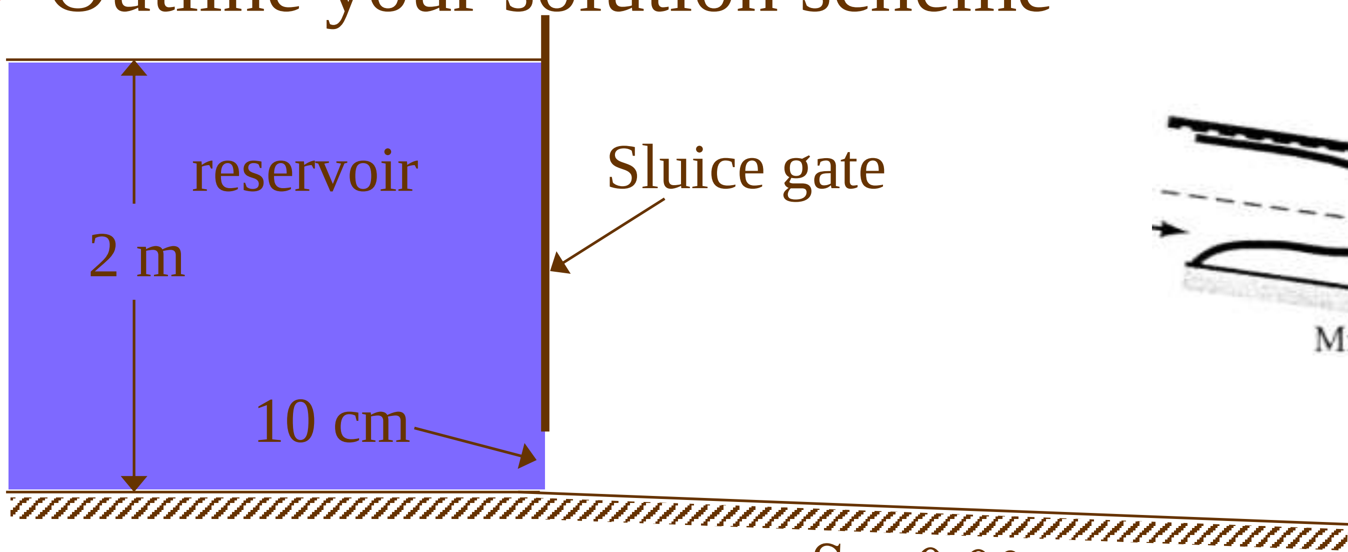


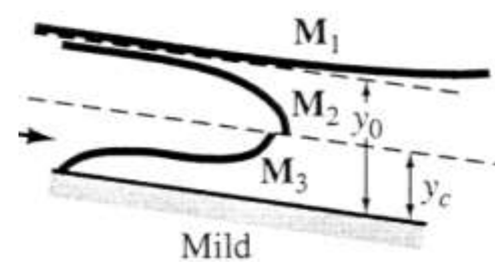
$$\frac{dM}{dy} = y + \frac{-q^2}{y^2g}$$

$$y = \left(\frac{q^2}{g}\right)^{1/3}$$ Critical depth!

Hydraulic Jump Location

- Suppose a sluice gate is located in a long channel with a mild slope. Where will the hydraulic jump be located?
- Outline your solution scheme

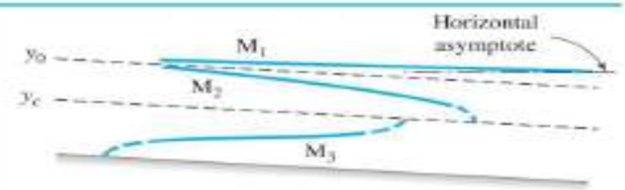
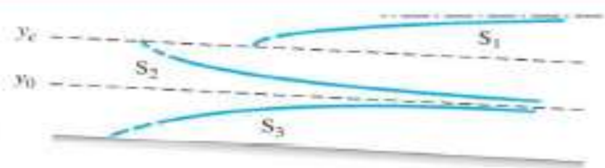
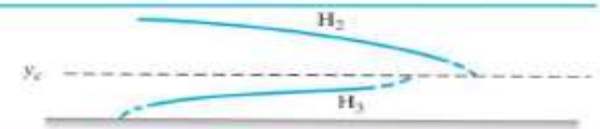




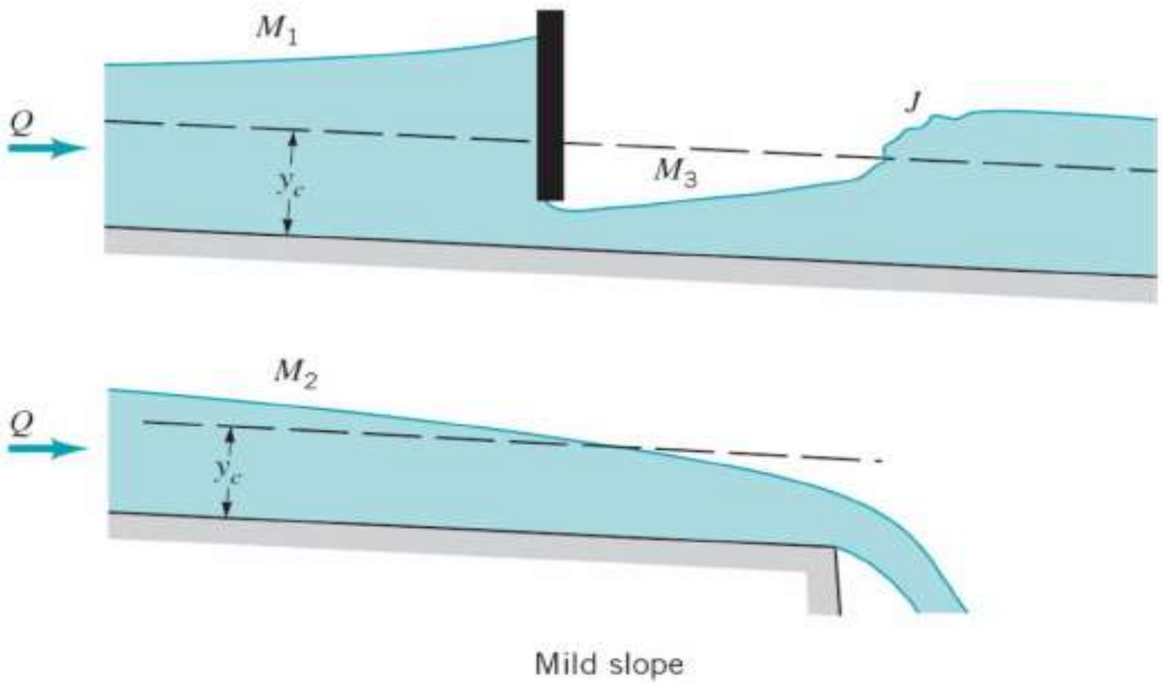
Surface Profiles

- **M**ild slope ($y_n > y_c$)
  - in a long channel subcritical flow will occur
- **S**teep slope ($y_n < y_c$)
  - in a long channel supercritical flow will occur
- **C**ritical slope ($y_n = y_c$)
  - in a long channel unstable flow will occur
- **H**orizontal slope ($S_o = 0$)
  - y_n undefined
- **A**dverse slope ($S_o < 0$)
  - y_n undefined

Classification of Surface Profiles

Channel slope	*Profile type*	*Depth range*	Fr	$\frac{dy}{dx}$	$\frac{dE}{dx}$	
Mild $S_0 < S_c$ $y_0 > y_c$	M_1	$y > y_0 > y_c$	< 1	> 0	> 0	
	M_2	$y_0 > y > y_c$	< 1	< 0	< 0	
	M_3	$y_0 > y_c > y$	> 1	> 0	< 0	
Steep $S_0 > S_c$ $y_0 < y_c$	S_1	$y > y_c > y_0$	< 1	> 0	> 0	
	S_2	$y_c > y > y_0$	> 1	< 0	> 0	
	S_3	$y_c > y_0 > y$	> 1	> 0	< 0	
Critical $S_0 = S_c$ $y_0 = y_c$	C_1	$y > y_c$ or y_0	< 1	> 0	> 0	
	C_3	y_c or $y_0 > y$	> 1	> 0	< 0	
Horizontal $S_0 = 0$ $y_0 \to \infty$	H_2	$y > y_c$	< 1	< 0	< 0	
	H_3	$y_c > y$	> 1	> 0	< 0	
Adverse $S_0 < 0$ y_0 undefined	A_2	$y > y_c$	< 1	< 0	< 0	
	A_3	$y_c > y$	> 1	> 0	< 0	

Examples of Gradually Varied Flows



Typical surface configurations for nonuniform depth flow with a **mild slope**

Discharge Measurements

- Sharp-Crested Weir $$Q = \frac{2}{3} C_d b \sqrt{2g} H^{3/2}$$
- V-Notch Weir $$Q = \frac{8}{15} C_d \sqrt{2g} \tan\left(\frac{\theta}{2}\right) H^{5/2}$$
- Broad-Crested Weir $$Q = C_d b \sqrt{g} \left(\frac{2}{3} H\right)^{3/2}$$
- Sluice Gate $$Q = C_d b y_g \sqrt{2 g y_1}$$

Explain the exponents of H! $$V = \sqrt{2gH}$$

Failure of Hydraulic Structures Founded on Permeable Foundations

Lecture (3)

Introduction:

Hydraulic structures such as dams, weirs, barrages, head regulators, cross-drainage works, etc. may either be founded on an impervious solid rock foundation or on a pervious foundation. Whenever, such a structure is founded on a pervious foundation, it is subjected to seepage of water beneath the structure, in addition to all other forces —to which it will be subjected when founded on an impervious rock foundation. Most of these hydraulic structures are required to be founded on alluvial soil foundations, which do allow seepage beneath them.

Causes of Failures of Weirs on Permeable Foundations

Causes of Failure:
1. Due to Seepage or Sub-surface Flow
2. Piping or Undermining
3. Rupture of Floor by Uplift Pressure
4. Due to Surface Flow
5. By Suction due to Hydraulic Jump
6. By Scour on the u/s and d/s of the weir

(1) Failure by Piping or Undermining. When the seepage water retains sufficient residual force at the emerging downstream end of the work, it may lift up the soil particles. This leads to increased porosity of the soil by progressive removal of soil from beneath the foundation. The structure may ultimately subside into the hollow so formed resulting in the failure of the structure.
(2) Failure by Uplift pressure. The water seeping below the structure exerts an uplift pressure on the floor of the structure. If this pressure is not counterbalanced by the weight of the concrete or masonry floor, the structure will fail by a rupture of a part of the floor.

Estimation of seepage amount

A rough estimation of the amount of seepage could be made using the Darcy's equation

$$Q = kiA$$

Where: Q = rate of seepage (m^3/s)

k = Permeability (Hydraulic conductivity) of the foundation material (m/s).

i = hydraulic gradient

$= \Delta H/\Delta L$ ΔH = upstream and downstream sections head difference (m).

ΔL = length of seepage path (m)

A = gross area of foundation through which flow takes place (m^2).

A better estimation of seepage quantity can be made by flow net analysis of the foundation. Using flow net technique,

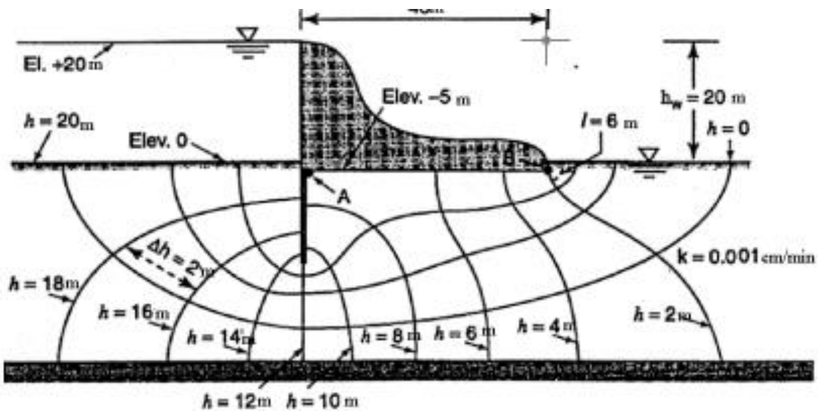
$$q = k\,\Delta H\, N_f / N_p$$

Where: q = seepage quantity per unit length ($m^3/s/m$)

N_f = number of flow channels

N_p = number of potential drops

Example: For the ogee spillway with sheetpiling cutoff shown below:



To compute the seepage

Alternative I.

$\Delta h = 2m$, $H = 20m$

$k = 0.00001 m/min$ [Hydraulic conductivity of pervious foundation]

$L = 48m$ [bottom length of the spillway]

$I = \Delta h/\Delta l = 2/6 = 0.33$

$q = KIA$ [Darcy's Equation]

$= 0.00001\ m/min * 0.33 * 24\ m^2/m$

$= 8.0 \times 10^{-5}\ m^3/min$ Per meter length of structure

Alternative II

$H = 20m$ [head difference of head and tail water level]

$N_f = 4$ [number of Flow channels]

$N_d = 10$ [number of Equipotential lines]

$$q = kH \frac{N_f}{N_d}$$

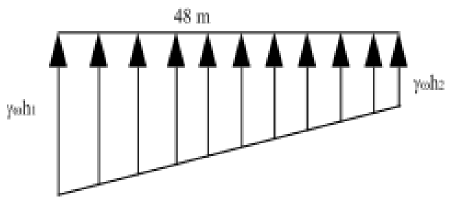
$$q = kH \frac{N_f}{N_d} = 0.00001\ m/\min*\ 20 * \frac{4}{10}$$

$$= 8.0 * 10^{-5}\ m^3/\min/\ m \text{ length}$$

Uplift force on the dam

At point A $h1 \approx 7.5m$

At point B $h2 = 2m$



Uplift Pressure

$$P_u = \gamma_w \left[\frac{h1 + h2}{2}\right] = 10\left[\frac{7.5+2}{2}\right] = 47.5KPa$$

Uplift force $F_u = P_u \times A = 47.5 * 48 = 2280.0$ kN per meter length of the structure.

a. Compute the seepage in m^3/min.; and

b. Calculate the uplift force acting on the base of the dam.

Theories of Seepage Flow

Whenever a hydraulic structure is founded on a pervious foundation, it is subjected to seepage of water beneath the structure, in addition to all other forces to which it will be subjected when founded on an impervious rock foundation. The concepts of failure of hydraulic structures due to subsurface flow were introduced by Bligh, on the basis of experiments and the research work conducted after the failure of Khanki weir, which was designed on experience and intuition without rational theory.

Cutoffs

The upstream and downstream cut-offs have a valuable purpose to serve in preventing failure of the impervious floor by slipping of the subsoil into the scour holes if any are formed. In addition, they control seepage and uplift pressures. At river bed the materials are mainly of gravel and conglomerate therefore formation of scour hole will be limited and may be not possible therefore the cutoffs will be used mainly for control of seepage and uplift under the structure. (cut-off) are generally necessary at both upstream and downstream ends of the floor.

Uplift Pressures:

The failure of a structure founded on permeable foundations can occur due to excessive uplift pressures, acting directly on the floor of the structure or due to undermining of the structure by the seeping water. Due to the removal of the fines by the excessive velocity of the under flow, resulting in what are known as pipes of flow. The failure itself is referred to as "piping".

Design of Impervious Floor:

Directly depended on the possibilities of percolation in porous subsoil
Water from upstream percolates and creeps (or travel) slowly through weir base and the subsoil below it.
The head lost by the creeping water is proportional to the distance it travels (creep length) along the base of the weir profile.
The creep length must be made as big as possible so as to prevent the piping action.
This can be achieved by providing deep vertical cut-offs or sheet piles

1. Bligh's Creep Theory (1912)
2. Lane's Weighted Creep Theory (1932)
3. Khosla's Theory (1936)

Bligh's Creep Method

The seeping water followed the outline of the contact surface of the structure and foundation soil. The length of the path traversed is the creep length [L]. The loss of head is assumed to be proportional to the creep length. Referring to Fig. (1)

The total head loss between upstream and downstream $[H_L] = h_1 - h_2$

Creep Length $[L] = 2d_1 + L_1 + 2d_2 + L_2 + 2d_3$

Head loss per unit length of hydraulic gradient $= H_L / L$

Safety against piping is ensured by providing sufficient creep length,

$L = CH_L$

Where: C = Bligh"s coefficient for the soil

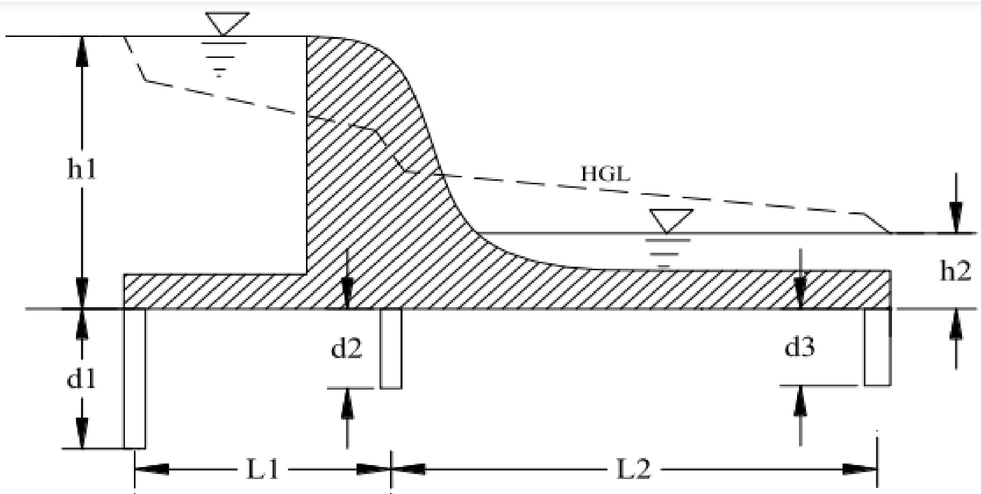


Fig. (1) Bligh's and Lane's Creep

Assumptions:

1. The percolating water follows the outline of the base of the foundation of the hydraulic structure. In other words, water creeps along the bottom contour of the structure.

2. The length of the path thus traversed by water is called the length of the creep.

3. The loss of head is proportional to the length of the creep. If H_L is the total head loss between the upstream and the downstream, and L is the length of creep, then the loss of head per unit of creep length (i.e. H_L/L) is called the hydraulic gradient.

Limitations of Bligh's theory:

- Bligh makes no distinction between horizontal and vertical creep.

- Bligh had calculated the length of the creep, by simply adding the horizontal creep length and the vertical creep length, thereby making no distinction between the two creeps.

Lane's Creep Method

The following Lane's method is adopted for calculating the uplift pressures:

Lane analyzed over 200 dams all over the world (24 of which had failed) and propounded his weighted creep theory. Lane adopted a weightage of three of vertical creep and one for horizontal creep. While this theory was an improvement over the original Bligh's creep theory, it was also empirical in nature.

$$L_W = (1/3)\, H + V = C_W\, H_L$$

L_W must not be less than $\mathbf{C_W\, H_L}$

H	Sum of Horizontal Percolation Lines	m
V	Sum of Vertical Percolation Lines	m
H_L	Maximum Head of Water across the Structure	m
C_W	Weighted Creep Ratio , from following table	
L_W	Weighted Creep Length	m

Table 2-1 Lane's recommended WCR for different materials

Material	Case a Lane 100%	Case b Lane 80 %	Case c Lane 70%
Very fine sand and silt	8.5	6.8	6
Fine sand	7.0	5.6	4.9
Medium sand	6.0	4.8	4.2
Coarse sand	5.0	4.0	3.5
Fine gravel	4.0	3.2	2.8
Medium gravel	3.5	2.8	2.5
Coarse gravel (including Cobbles)	3.0	2.4	2.1
Boulders with stone, cobbles, and gravel	2.5	2.0	1.8
Soft clay	3.0	2.4	2.1
Medium Clay	2.0	1.6	1.5
Hard clay	1.8	1.5	1.5
Very hard clay and hard pan	1.6	1.5	1.5

This method gives an upper safe limit to the average hydraulic gradient. The uplift pressure by this method can be calculated as follows:

$$P_i = 100\,(LW - L_i) / L_W$$

$$U_i = H_L\, P_i\, (+ \text{ or } -)\, EL_i$$

Where;

Pi	Percentage of Seepage Head at point (i)	%
Li	Sum of Weighted Creep Lengths to Point (i)	m
Ui	Uplift Pressure at Point (i)	m
ELi	Elevation Head above Datum at point (i)	m

Although This method is simple it gave reasonably accurate results were compared with results obtained from advanced numerical seepage models, therefore it is considered adequate for hydraulic structures design.

Limitations of Lane's theory:

Lane's theory was an improvement over Bligh's theory, but however, was purely empirical without any rational basis, and hence, is generally not adopted in any designs. Bligh's theory, though is still used (even after the invention of modern Khosla's theory), but Lane's theory is practically nowhere used, and is having only a theoretical importance.

The soil layer at foundation consists of Gravel and Conglomerates with beds of sand and clay as shown in (Fig. 2). This layer extends from the natural ground surface to the end of boring Therefore, C_W=3.5 will be adopted to be used in lane's method.
Cutoffs are planned to satisfy three requirements.

- Seepage creep line – safety against piping
- Seepage exit gradient - stability at seepage exit downstream
- Control and reduce of uplift under the structure

The cutoffs are planned to be extended to Elevation 481.00 m asl for weir part

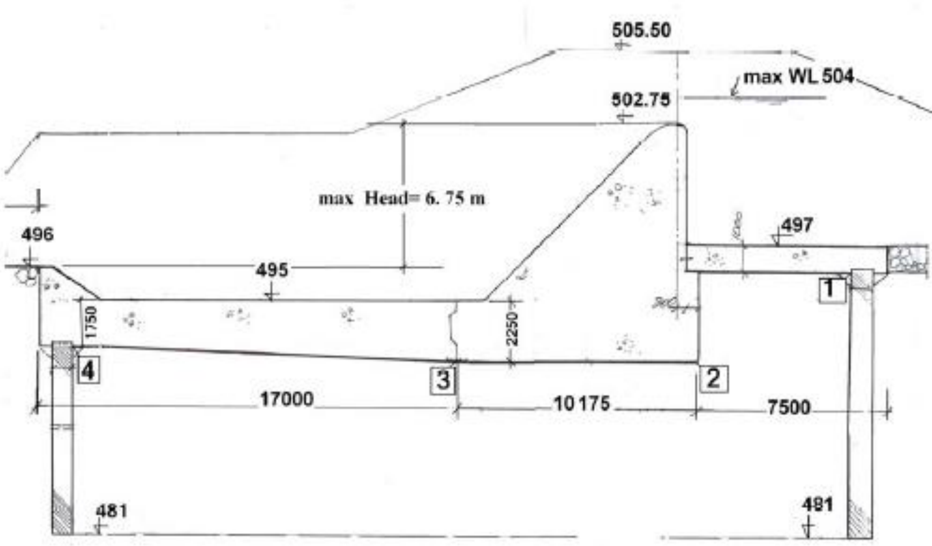


Fig.(2) Profile of Diversion Hydraulic Structure

Max Head-= Crest Level – Downstream BL = 502 .75-496= 6.75 m

According to Lane's method.
Available Weighted Creep Line (L_W) = 1/3 $\sum$(H) + $\sum$ (V)

L_w = 16+15+3.25+12.3+15+33.875/3= 72.84 m

Required min L_w=C_w*H =3.5*6.75 = 23.625 m Safety factor against piping= 3

It is O.K.

Uplift Pressure Uplift pressure under the weir structure is calculated as follows;

Point	Creep length To point (m)	% uplift at point	Uplift Pressure KN/m^2
1	16+15=31	100*(72.84-31)/ 72.84=57	38.75
2	31+3.25+7.5/3=36.75	100*(72.84-36.75)/ 72.84=48.8	33.4
3	36.75+10.175/3=40.14	100*(72.84-40.14)/ 72.84=44.9	30.4
4	40.14+17/3=45.81	100*(72.84-45.81)/ 72.84=37.1	25.05

-Uplift at points 1and 2 have no effect on the U/s Apron stability since they will be equalized by the weight of water above the apron.
Uplift at points 2 and 3 will be used in the stability analysis of the mass concrete weir
- Uplift between points 3 and 4 will be used in the stability check of the stilling basin mass concrete floor the submerged floor will resist this uplift by its own buoyant weight so,

Factor of safety= (buoyant weight)/uplift pressure Fs (3) = 2.25*(25-10)/30.375= 1.11 Fs (4) = 1.75*(25-10)/25.048= 1.05
So, a variable thickness 2.25 m -1.75 m floor can resist uplift. And hence it is stable.

Exit Seepage Gradient and safety against Failure by piping or undermining

when a hydraulic structure is founded on a previous foundation, it is subjected to seepage of water beneath the structure. The water thus seeping below the body of the hydraulic structure and may cause its failure either by piping or undermining or by direct uplift
When the seepage water retains sufficient residual force at the emerging downstream end of the floor it may lift up the soil particles. This leads to increased porosity of the soil by progressive removal of soil from beneath the foundations. The structure may ultimately subside into the hollow so formed, resulting in the failure of the structure.
According to Khosla's theory and concept of flow nets, the seepage water exerts a force at each point in the direction of flow and tangential to the stream lines. This force has an upward component from the point where the streamlines turn upward. For the soil grains to remain stable, the upward component of this force should be counter balanced by the submerged weight of the soil grain. This force has the maximum disturbing tendency at the exit end, because the direction of this force at the exit point is vertically upward and hence full force acts as its exclusive upward component. For the soil grain to remain stable, the submerged weight of soil grain should be more than this upward force. The disturbing force at any point is proportional to the gradient of pressure seepage water at the point. This gradient of pressure of water at exit is called the "exit gradient".
In order that the soil particles at exit remain stable, the upward pressure at exit should be safe. In other words, the exit gradient should be safe, the exit gradient is said to be critical when the upward disturbing force on the soil grain is just equal to the submerged weight of the soil grain at exit, of a unit volume of soil grain at exit.
(Plate 11.2) gives the equations proposed by Khosla's for the exit gradient calculation
It is seen from the above Figure that the exit gradient depends upon the following:

- head causing flow.
- length of the impervious floor.
- depth of downstream cut-off

While designing the structure, the impervious floor length and the depth of downstream sheet pile will be so adjusted that the safe exit gradient as computed for the structure shall be within the safe limits shown in the following table:

Soil type	**Safe exit gradient G_E**
Shingles & gravel	1/4 to 1/5
Coarse sand	1/5 to 1/6
Fine sand	1/6 to 1/7
Alluvial soils	1/7 to 1/10

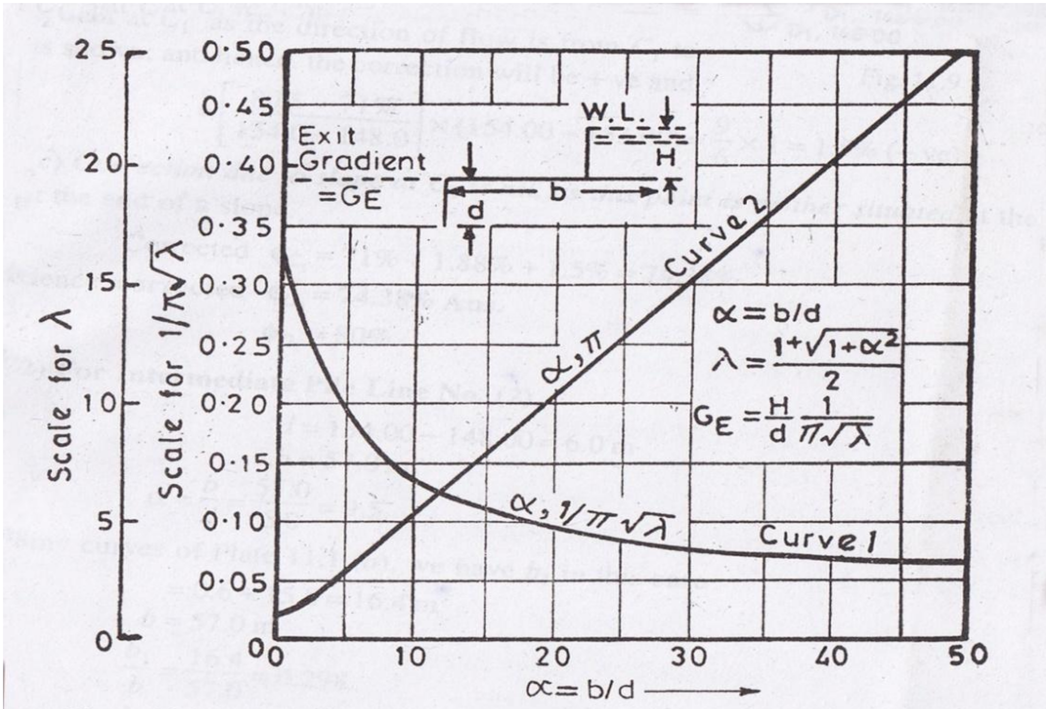


Plate 11.2

The seepage exit gradient is calculated using Khosla's method for the diversion weir as for α =b/d = 33.875/15= 2.258 From Fig 13 K=0.1

Ge = k (H/d) = 0. 1*6.75/15 = 0.045 or 1/ 22 which is safe
for the type of soils we have as shown in above table

Launching Aprons

To protect the downstream concrete floor, a loose apron called launching apron is provided for a sufficient length. This loose apron (launching apron) as the name implies is supposed to launch to a certain slope when scour takes place below it. The stone provided in the launching apron will cover the launched slope and thus prevent the scour of the launched apron further. The quantity of stone over the launched slope should be sufficiently thick to prevent further scour. The quantity of stone in the aprons should be sufficient to afford sufficient cover over a launched apron position below the level at which the apron is originally laid to the bottom of the deepest scour that is likely to occur at the particular locality using the scouring criteria for alluvial soils. The scouring depth R will be;

$$R = 1.35 \left(q^{2} / f \right)^{1/3}$$

Where; f = Silt Factor depends on bed material = 1.76 $(D_{50})^{1/2}$

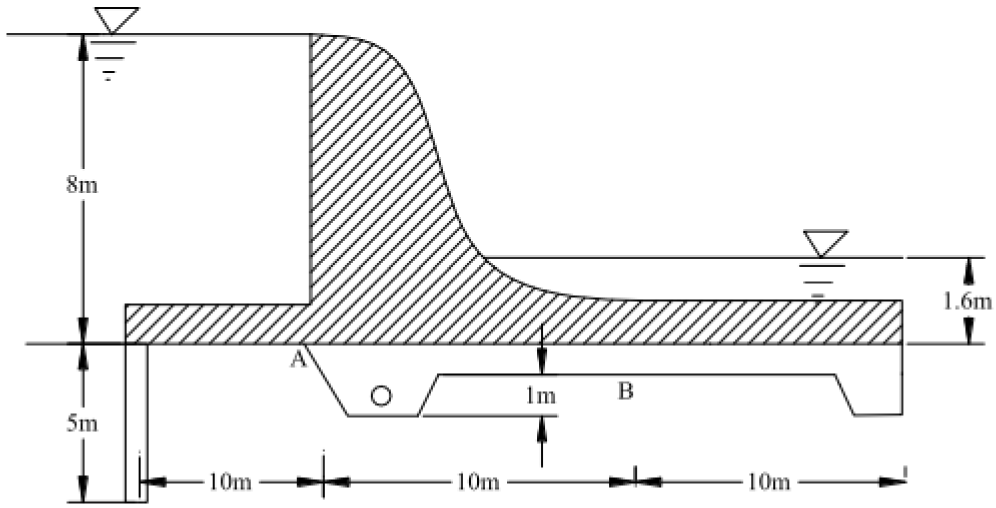
If the structure will be founded on the main soil layer which consist of dense gravel with some sands, then the D50 is taken to be 50 mm then f = 12, having q= 222.6/75 = 2.968 m^3/s/m, R is estimated to be 0.85 m , measured from water surface ,this means that no scour is anticipated either side of the structure .despite of this fact, loose riprap protection aprons are added on both upstream and downstream sides of the structure.

Example:

For the hydraulic structure section shown below determine

1- the type of the foundation on which the dam section shown below may be judged safe;

2- the magnitude of the uplift force for the section A to B



Solution

Weighted creep length = 5 + 5 + 4*1 + (10 + 10 + 10)/3 = 24 m

Net head on structure = Head water – Tail water = 8-1.6 = 6.4 m

Weighted creep ratio = 24 / 6.4 = 3.75

According to Lane's ratios, the dam(spillway) would be safe on clay or on medium gravel and coarse gravel. With properly provided drains and filters, it may be considered safe on fine gravel foundations [case b]

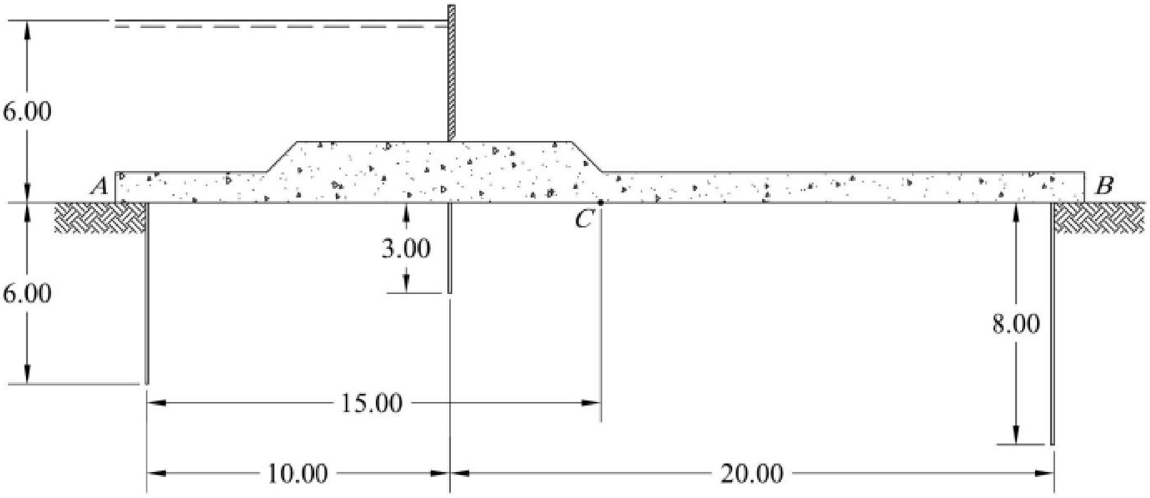
$$\text{Uplift at point A} = 6.4 - \frac{(5 + 5 + 10/3)}{24} * 6.4 + 1.6 = 4.44\ m$$

$$\text{Uplift at point B} = 6.4 - \frac{(5 + 5 + 1 + 1 + 10/3 + 10/3)}{24} * 6.4 + 1.6 = 3.02$$

$$\text{Total Uplift on section AB} = \frac{(4.44 + 3.02)}{2}\gamma_w = 36.591\ kN/m \text{ crest length}$$

Example

Find the hydraulic gradient and uplift pressure at a point 15 m from the upstream end of the floor i the figure below.



Dimensions are in meters

Solution

Water percolates at point A and emerges at point B,

Total creep length $= 2\times 6+10+2\times 3+20+2\times 8=64m$

Head of water on structure= 6 m

Hydraulic gradient $= \dfrac{6}{64}=\dfrac{1}{10.66}$

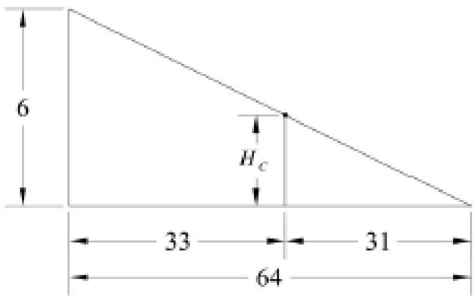
According to Bligh's theory, the structure would be safe on sand mixed with boulders

Creep length up to point C=$L_1 = 2\times 6+2\times 3+15=33m$

$$H_C = \frac{6}{64}(64-33)=2.91m$$

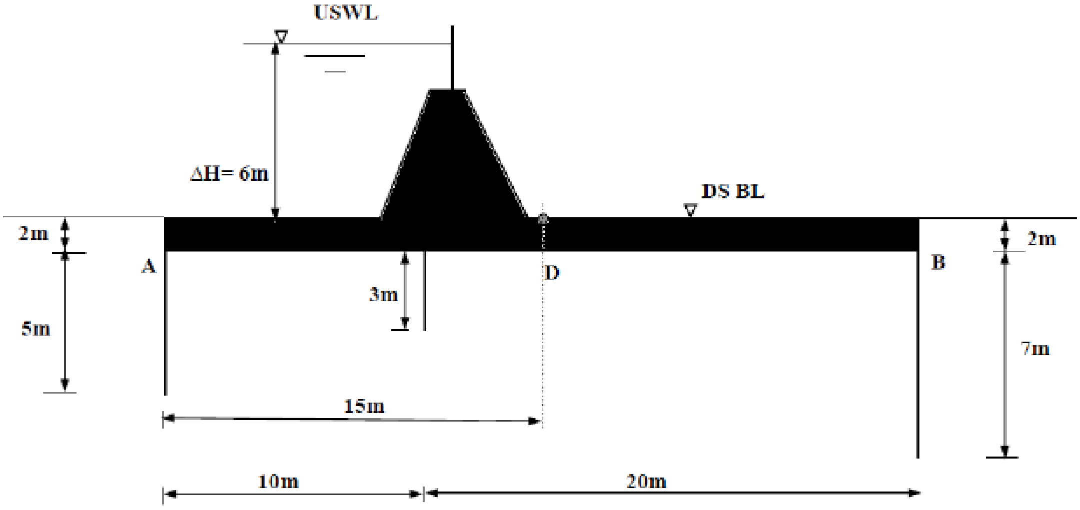
$$t_c = \frac{H_C}{G_c-1}=\frac{\gamma_w H_C}{\gamma_w G_c-\gamma_w}$$

$$=\frac{2.91}{2.4-1}=2.076m \text{ of concrete}$$



Example: Find the hydraulic gradient and the head at point D of the following structure for Static condition.

The water percolates at A and exits at B.



Example: <u>1. Using Blight' Creep theory</u>

Total creep length, Lc = 2 + 5*2 + 10 + 2*3 + 20 + 2*7 + 2 = 64m

Hydraulic gradient, i = HL/L=ΔH/Lc=6/64=1/10.66

According to the Bligh's table, the structure is safe on gravel and sand but not on coarse and fine sand.

Remember HL/L ≤1/C !!!

Creep length up to point D (LcD) =2 + 5*2 + 15 + 2*3 = 33m

The residual uplift pressure head at D = UD = ΔH(1-LcD/Lc)

=6(1-33/64)=2.9m

Example: <u>1. Using Blight' Creep theory</u>

The thickness of floor at any point should be sufficient to resist the residual uplift pressure. If hD is the unbalanced head at point D, then

hD= UD- (elevation of point D – elevation of DS floor) = 2.9 – (0) = 2.9 m (Note: Point D is at the same level of DS floor level).

The thickness of floor, T_D, at point D should be h_D/(G-1) where G, is the specific gravity of the concrete floor, let Gs= 2.24, then

$$T_D=h_D/(G-1)=2.9/(2.24-1)=2.34m$$

Design of Hydraulic Structures: Barrages and Regulators

Lecture No.(4)

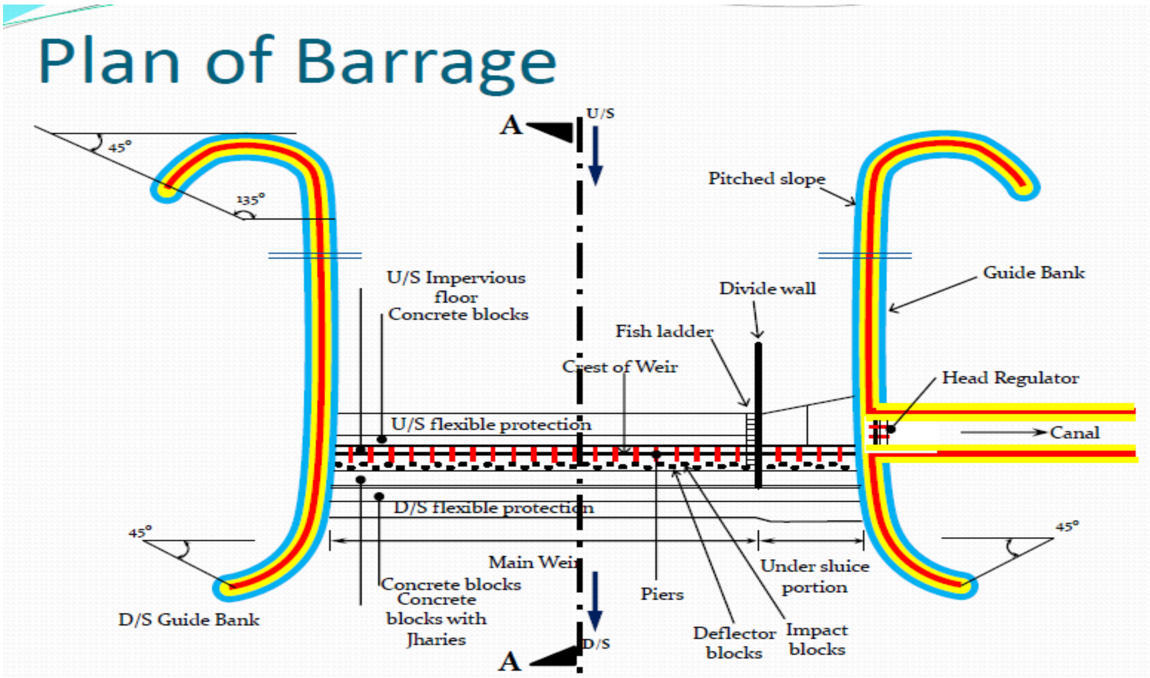
Canal Head Works:

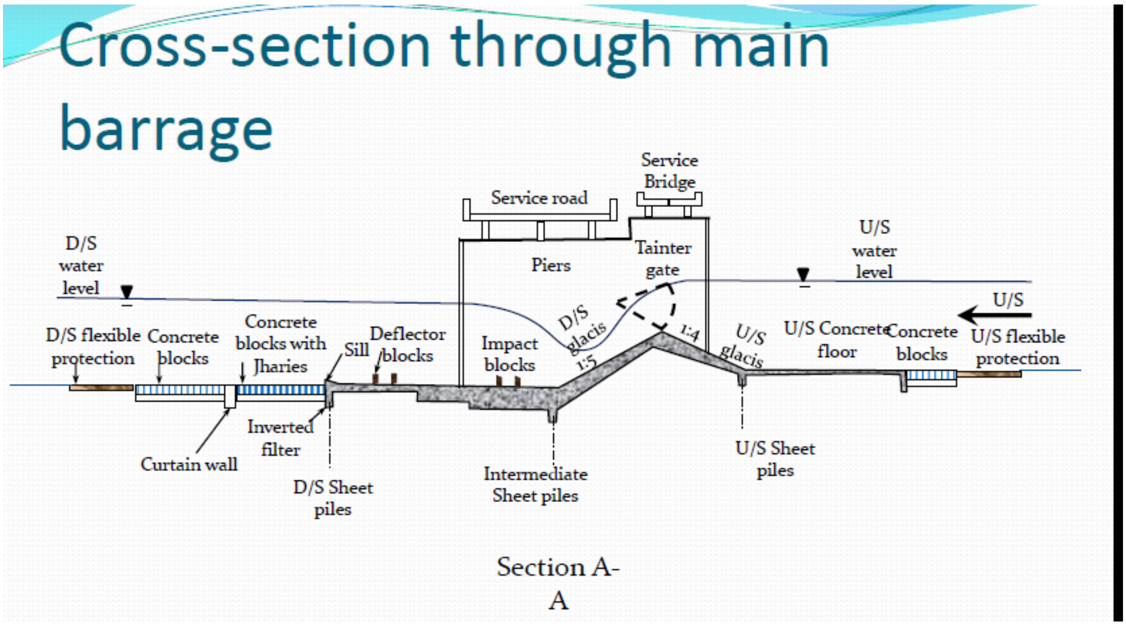
Any hydraulic structure which delivers water to off-taking canal is called a ***headwork.*** Canal Headworks are classified to:

Diversion Headworks: A barrage or regulator is constructed across a river to raise water level and to divert water to a canal is known as a diversion headwork.

Storage Headworks: A dam is constructed across a river valley to form a storage reservoir is known as a storage headwork.

A dam is constructed across a river valley to form a storage reservoir upstream the dam which diverts specified discharge flow downstream the dam. The main object of the canal headworks is to divert the water from the river into the canal. It is sometimes termed diversion headworks also. Canal Headwork is a civil engineering term for any hydraulic structure constructed across a river and supplies water to the off taking canal.





Diversion Headwork:

These include the ***cross regulator*** and the ***distributary head regulator*** structures for controlling the flow through a parent canal and its off-taking distributary as shown in Figure 1. They also help to maintain the water level in the canal on the upstream of the regulator. Canal regulators, which are gated structures, may be combined with bridges and falls for economic and other considerations, like topography, etc. A typical view of a distributary head regulator and a cross regulator shown in Figure 2.

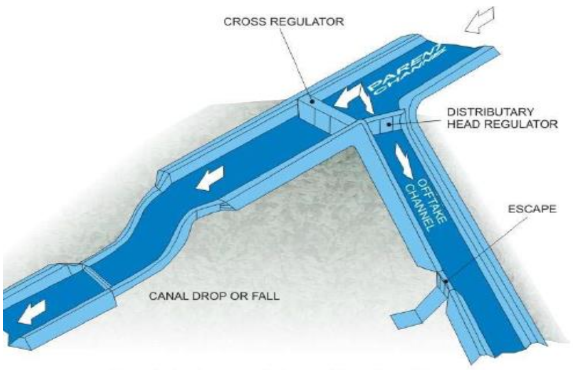


Fig.1 Canal Structures for Flow Regulation and control

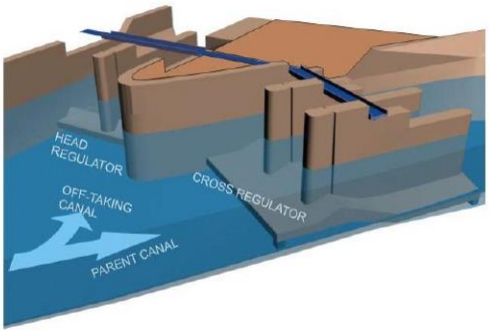


Fig.2 Gross and Head Regulators

Head and Cross Regulators

Head and Cross Regulators

The supplies passing down the parent canal and off take channel are controlled by cross regulator and head regulator respectively, Fig.(3).

Functions of Cross Regulators

1. Regulation of the canal system.

2. Raising the water level in the main canal in order to feed the off- take channels.

3. To facilitate communication by building a road over the cross regulator with little extra cost.

4. To absorb the fluctuations in the canal system.

Functions of Head Regulators:

1- to regulate and control supplies entering the off take channel (distributary) from the main (parent) canal.

2- To control silt entering into the distributary channel.

3. To serve for measurement of discharge.

-

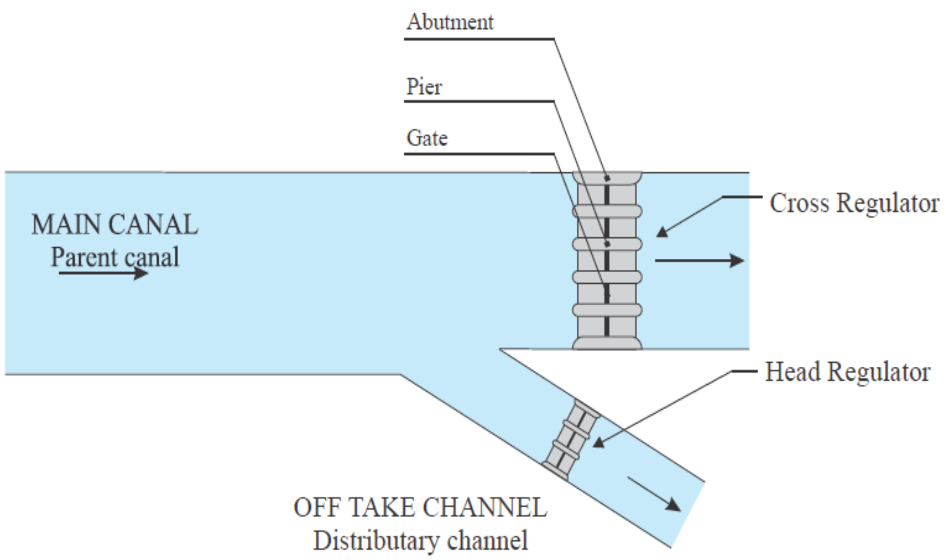


Fig.(3).Off-take Distributary Channel

Alignment

The best alignment of the off-take channel is when it makes angle zero with the parent canal initially and then separates out in a transition. See Fig. (4). In this case there is a transition curve for both off take and parent channel to avoid silt accumulation. Another alternative by making both channels an angle with respect to parent channel upstream, Fig.(5). In case of obligatory straight alignment of the parent channel, the usual angle of the off-take channel is 60º to 80º (in most important works needs a model study). For excessive silt entry into the off-take channel. Fig.(6)

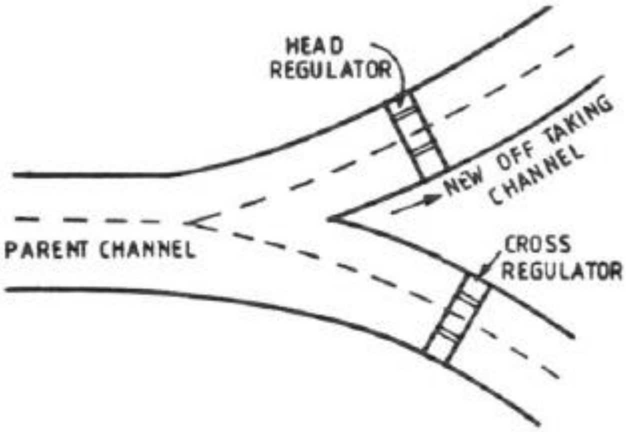


Fig. (4)

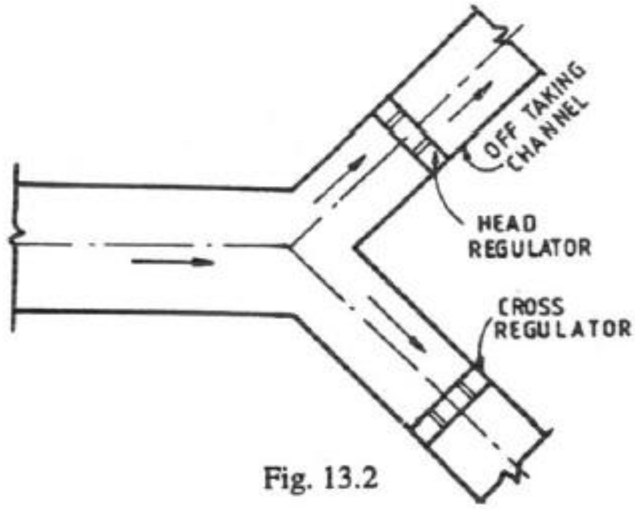


Fig. (5)

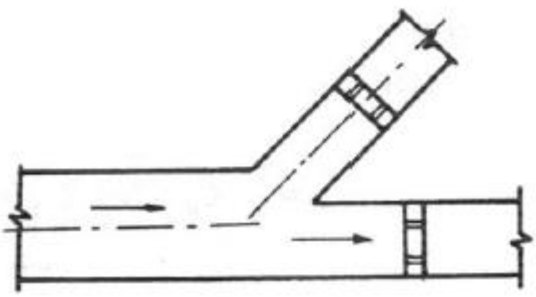


Fig. (6)

Components of Head Works

Head works consist of the following components: Fig.(7)

- Under-sluices.
- Canal head regulator.
- Divide wall.
- Fish ladder.
- Piers and abutments.
- Protection works.
- River training Wall

.

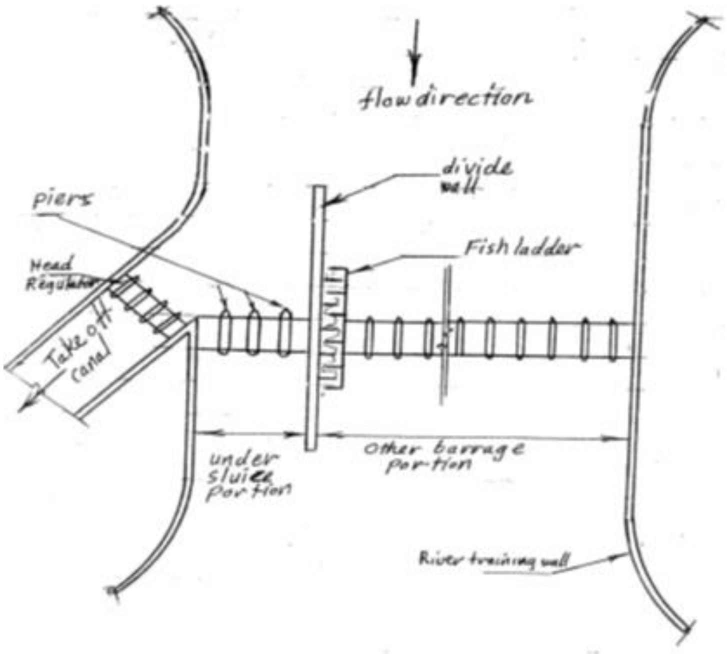


Fig.(7) Headwater Components

Under-sluices

These are gates-controlled openings in the weir with crest at low level. They are located on the same side as off-take canal. If two canal take off on either side of the river, it would be necessary to provide under-sluices on either side.

Functions of under-sluices

-To preserve a clear and defined river channel approaching the canal regulator.

-To scour silt deposited in front of canal regulator and control silt entry in the canal.

-To facilitate working of weir crest shutters or gates. The flood can easily pass.

-To lower the highest flood level.

Discharge Capacity of under-sluices is provided of the following:

a. $Q_u = 2\ (Q_{max.})_{offtake}$

b. $Q_u = 20\%\ (Q_{max.})_{flood}$

Divide Walls

It is a wall located between weir and under-sluices extending a little U.S of canal regulator, and D.S up to end of loose protection of the under-sluices. It is a concrete or masonry structure, with top width (t)= (l.5-3) m, and aligned at right angle to the weir axis.

The functions of divide walls are

-To separate the floor of scouring sluices which is at lower level than the weir proper.

-To isolated the pockets upstream of the canal head regulator to facilitate scouring operation.

-To prevent formations of cross currents to a void their damaging effects. Additional divide walls are sometimes provided for this purpose.

Fish ladder:

Fish ladder or fish passes are generally provided to enable the fish to ascend the head waters of the river and thus reach their spawning grounds for propagation or to follow their migratory habits in search of food.

The general requirements of a fish ladder are:

-The slope of the fish ladder should not be steeper than 1:10 (i.e velocity not exceeding 2 m/s in any portion of the fish-way).

-The compartments of bays of the pass must be such dimensions that the fish do not risk collision with the sides and upper end of each bay when ascending.

-The water supply should be ample at all times.

The top and sides of a fish-way should be above ordinary high-water level,Fig.(8,9)

Fig.(8-9) Fish-Ladder

Piers and abutments

In barrages or regulators, piers are provided at an interval of 10 to 20 *m*. The piers support bridge decking, and working platform for the operation of gates. Cutwaters are usually simple in shape and the side face of piers is often vertical. Tapering if done, does not exceed 1/50 to 1/40. Piers should be provided with separate foundation, Fig.(10)

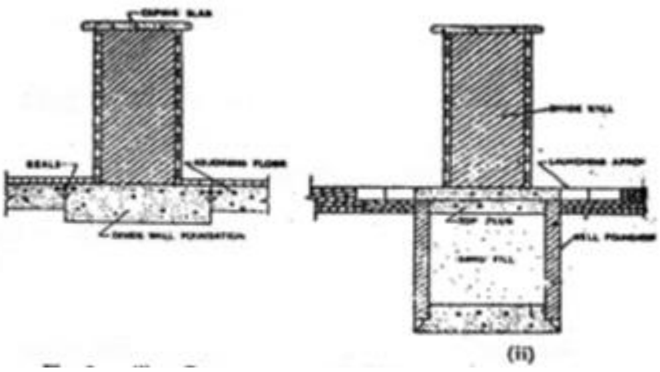


Fig.(10) Cross section of divide wall on pucca floor