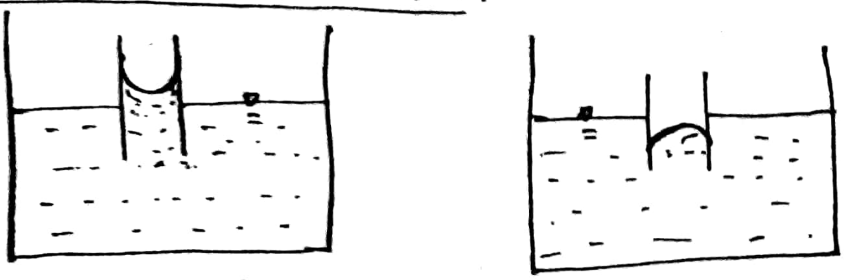
Fluid Mechanics

Introduction

Application of Fluid Mechanics:

- Navigation and water power engineering
- Drainage network
- Irrigation
- Flow in pipes & channels
- Design of hydraulic structures.
- Water distribution / sewerage network.

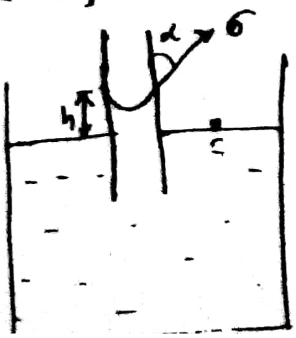
Effects of surface tension:



a) in water b) in mercury

- Water rises up in the tube with an upward concave surface due to the reason that the adhesion between the tube and water molecules is more than the cohesion between the water molecules.
- Mercury depresses down in the tube with an upward convex surface due to the reason that the adhesion between the tube and mercury molecules is less than the cohesion between the mercury molecules.

Cappillarity:
The phenomenon of rise or fall of liquid in the tube of smaller diameter is called cappilarity of liquid.



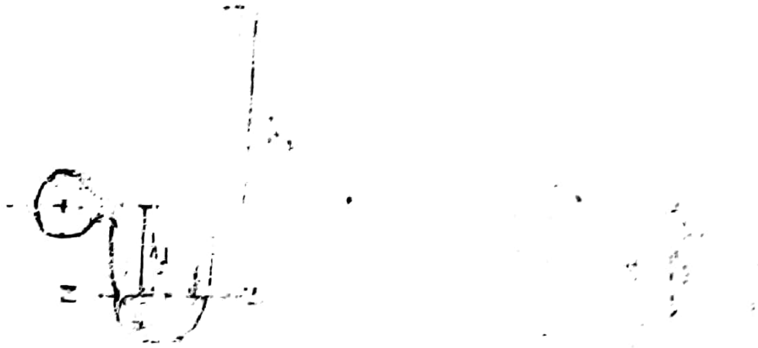
$$h = \frac{4\sigma \cos\alpha}{wd}$$

Fluid pressure Measurement

Simple Manometer:

$s_1 = 1$ for water

$s_2 = 13.6$ for mercury



(a) positive pressure (b) Negative pressure

For positive pressure:

pressure in left limb above Z-Z = pressure in right limb above Z-Z

$\Rightarrow h + s_1 h_1 = s_2 h_2 \Rightarrow \boxed{h = s_2 h_2 - s_1 h_1}$

where, h = pressure in pipe in terms of he[illegible]ter, m

h_1 = height of light liquid above Z-Z

h_2 = " " heavy " " Z-Z

s_1 = sp. gr. of light liquid

s_2 = sp. gr. of heavy liquid

For negative pressure:

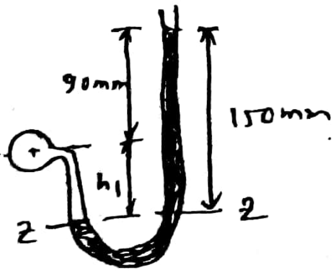
pressure in left limb above Z-Z = pressure in right limb

$\Rightarrow h + s_1 h_1 + s_2 h_2 = 0$

* A simple manometer is used to measure the pressure of oil (sp. gr. = 0.8) flowing in a pipe line. Its right limb is open to the atmosphere and the left limb is connected to the pipe. The centre of pipe is 90 mm below the level of mercury (sp. gr. = 13.6) in the right limb. If the difference of mercury levels in the two limbs is 150 mm, find the pressure of oil in pipe.

Soln:

$\therefore h_1 = 150 - 90 = 60\,mm$



$h = S_2 h_2 - S_1 h_1$

$= 13.6 \times 150 - 0.8 \times 60$

$= 1992\,mm$

$= 1.992\,m$ of water

$\therefore P = wh = 9.81 \times 1.992 = 19.54\ KN/m^2$

Ans.

* A simple manometer containing mercury was used to find the negative pressure in the pipe containing water 20 mm height an 50 mm mercury in left limb. The right limb of the manometer was open to the atmosphere. Find the negative pressure below the atmosphere in the pipe.

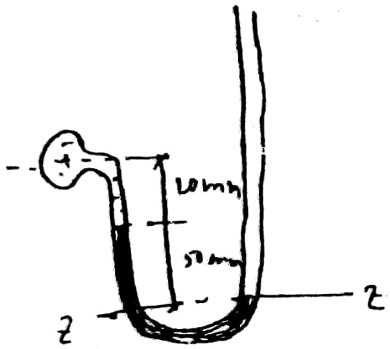
Soln:

$h + S_1 h_1 + S_2 h_2 = 0$

$\Rightarrow h = -1 \times 20 - 13.6 \times 50$

$= -700\,mm$

$= -0.7\,m$ of water



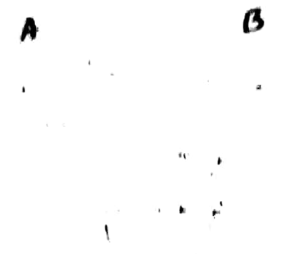
$P = wG$

$= 9.81 \times -7$

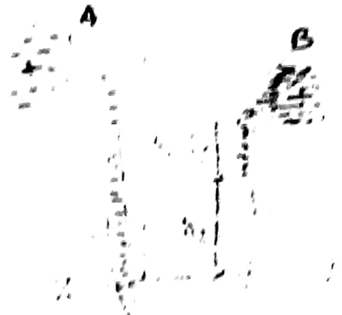
$= -6.867\ KN/m^2$

Ans.

Differential manometer:



a) A and B at the same level containing same liquid



b) A and B at differents level and containg different Liquid

For A and B at the same level containing same Liquid:

$$h_A + s_1(H+h) = h_B + s_1 H + s_2 h$$

For A and B at different level containg different liquid:

$$h_A + s_1 h_1 = h_B + s_2 h_2 + s_3 h_3$$

* A differential manometer connected at the two points A and B at the same level in a pipe containing an c (Specific gravity 0.8, shows a difference in mercury levels as 100 mm. Determine the difference in pressures at the two point.

Soln: $s_1 = 0.8$, $s_2 = 13.6$, $h = 100\,mm$.

We know,

$$h_A + s_1(H+h) = h_B + s_1 H + s_2 h$$

$$\Rightarrow h_A + \cancel{s_1 H} + s_1 h = h_B + \cancel{s_1 H} + s_2 h$$

$$\Rightarrow h_A - h_B = 13.6 \times 100 - 0.8 \times 100$$

$$= 1280\,mm$$

$$= 1.28\,m \text{ of water}$$

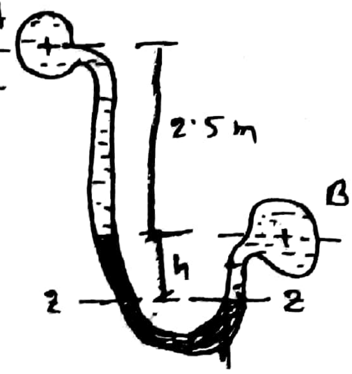
Ans.

\# A U-tube differential manometer connects two pressure pipes A and B. The pipe A contains carbon tetrachloride having a sp. gr. 1.6 under a pressure of 120 kPa. The pipe B contains oil of sp. gr. 0.8 under a pressure of 200 kPa. The pipe A lies 2.5 m above pipe B. Find the difference of pressures measured by mercury as fluid filling U-tube.

Solⁿ:

$$h_A = \frac{P}{W_W} = \frac{120}{9.81} = 12.2 \text{ m of water}$$

$$h_B = \frac{200}{9.81} = 20.4 \text{ m of water}$$



$$h_A + S_1 \times 2.5 + 13.6h = h_B + S_2 h$$

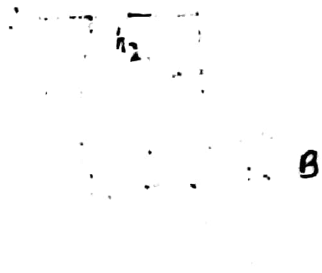
$$\Rightarrow 12.2 + 1.6 \times 2.5 + 13.6h = 20.4 + 0.8 \times h$$

$$\Rightarrow h = 0.328 \text{ m}$$

Ans.

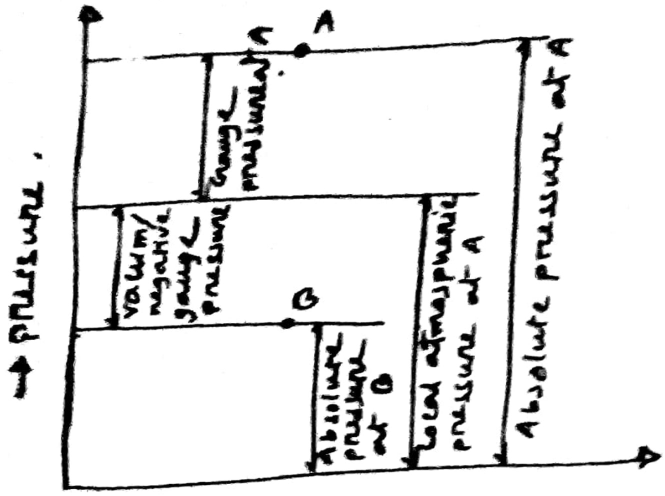
Inverted Differential Manometer:

$$h_A - S_1 h_1 = h_B - S_2 h_2 - S_3 h_3$$



\# **Water hammer:** The sudden rise of pressure has the
14 effect of hammering action on the walls of the pipe which is known as hammer blow or water hammer.

* Relation between absolute, gauge & vacuum pressure:



* Total pressure:

1) Total pressure on a horizontally immersed surface

$$P = wA\bar{x}$$

Where, w = sp. wt of the liquid

A = Area of immersed surface in m^2

$\bar{x}$ = Depth of horizontal surface from liquid level.

2) Total pressure on a vertically / Inclined immersed surface

$$P = wA\bar{x}$$

$\bar{x}$ = Depth of centre of gravity of the immersed surface from the liquid surface.

* Centre of pressure:

1) for a vertically immersed surface

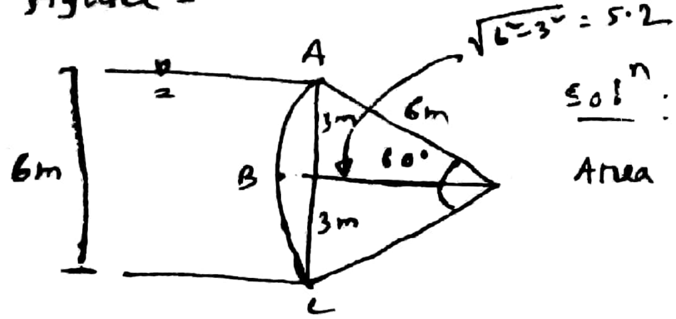
$$\bar{h} = \frac{I_G}{A\bar{x}} + \bar{x}$$

Where,

I_G = Moment of inertia of the figure, about horizontal axis through its cg

2) For inclined immersed surface, $\bar{h} = \frac{I_G \sin^2\theta}{A\bar{x}} + \bar{x}$

* Find out the horizontal and vertical force to the following figure -



Sol^n:

Area of ABC $= \frac{1}{6}\pi \times 6^2 - \frac{1}{2} \times 6 \times 5.2$

$= 3.25\ m^2$

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$\therefore P_H = wA\bar{x} = 9.81 \times \underbrace{(6 \times 1)}_{\text{Unit width}} \times \frac{6}{2} = 176.58$ KN

$P_v = w * \text{Volume}$

$= 9.81 * (\text{Area})_{ABC} * 1$

$= 9.81 * 3.25 * 1 = 31.88$ KN

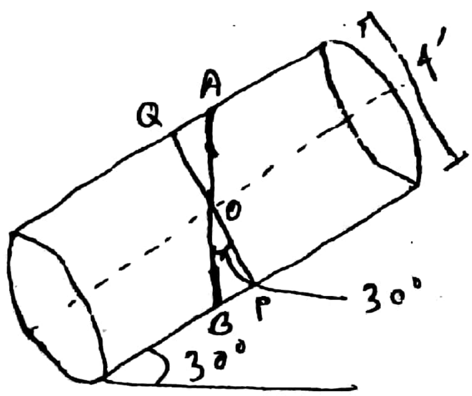
Ans

* Diameter of a pipe is 4 ft. Angle of inclination $\theta = 30°$. The pressure in the centre of pipe = 5 PSI. Determine the pressure at top & bottom level.

Sol^n:

$OA = OB$, $OQ = OP = 2'$

$\therefore OB = \frac{OP}{\cos 30°} = 2.31'$



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$\therefore P_{top} = 5 - \gamma h$

$= 5 - (62.4 \times 2.31)/144$

$= 4$ PSI

$P_{bottom} = 5 + \gamma h$

$= 5 + (62.4 \times 2.31)/144$

$= 6$ PSI Ans.

* **Bernoulli's Equation:** For a perfect incompressible liquid, following in a continuous stream, the total energy of a particle remains the same, while the particle moves from one point to another.

$$z_1 + \frac{v_1^2}{2g} + \frac{P_1}{\gamma} = z_2 + \frac{v_2^2}{2g} + \frac{P_2}{\gamma} + h_f$$

* **Discharge through a venturimeter:**

$$Q = \frac{C a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \sqrt{2gh}$$

where, C = co-efficient of venturimeter

a_1, a_2 = Area of venturimeter at section 1 and 2

* **Discharge through a small orifice:**

$$Q = C_d \, a \sqrt{2gh}$$

where, C_d = co-efficient of discharge for the orifice

a = x-sectional area of orifice.

* **Discharge through a external mouthpiece:**

$$Q = 0.855 \, a\sqrt{2gH} = C_d \cdot a \sqrt{2gH}$$

* **vena contracta:** The liquid particles lose some energy, while taking turn to enter into the orifice. The jet, after leaving the ~~ool~~ orifice, gets contracted. The maximum contraction takes place at a section $d/2$ on the U/s of the orifice, where the jet more/less horizontal, such a section is known as vena contracta.

* A venturimeter with a 150 mm diameter at inlet and 100 mm at throat is laid with its axis horizontal and is used to measuring the flow of oil, sp. gr. 0.9. The oil-mercury differential manometer shows a gauge difference of 200 mm. Assume, co-efficient of the metre as 0.98. Calculate the discharge in litres per minute.

Soln: $a_1 = \frac{\pi}{4} d_1^2 = \frac{\pi}{4} \times .15^2 = 17.67 \times 10^{-3} m^2$

$a_2 = \frac{\pi}{4} d_2^2 = \frac{\pi}{4} \times .1^2 = 7.854 \times 10^{-3} m^2$

$h = 0.2 \left(\frac{13.6 - 0.9}{0.9} \right) = 2.82 m$ of oil

$$\therefore Q = \frac{C a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \sqrt{2gh} = 63.9 \times 10^{-3} m^3/s$$

$= 3834$ litre/min

* A small rectangular orifice 200 mm deep and 500 mm wide is discharging water under a constant head of 400 mm. What will be the discharge through the orifice in litre/s, if the co-efficient of discharge for the orifice is 0.6.

Ans.

Soln: $a = b \times d = 0.5 \times .2 = 0.1 m^2$

$Q = C_d a \sqrt{2gh} = 0.6 \times 0.1 \times \sqrt{2 \times 9.8 \times 0.4}$

$= 0.168 m^3/s = 168$ Litre/s

Ans.

* Darcy weisbach formula/Darcy's formula:

Darcy's formula for loss of head in pipes –

$$h_f = \frac{4flv^2}{2gD}$$

where, h_f = loss of head due to friction

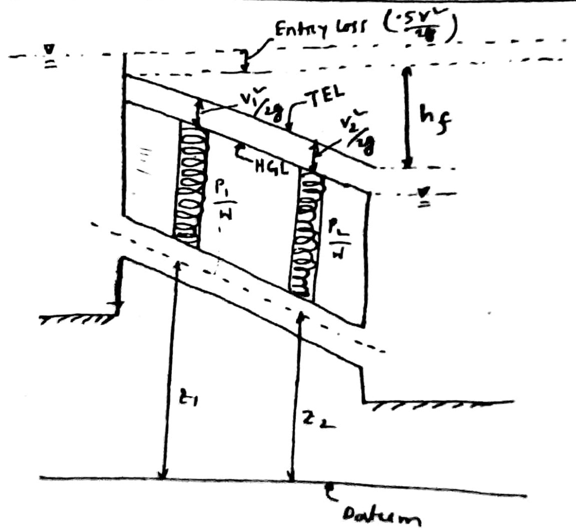
l = length of pipe

D = Diameter of pipe

v = velocity of water in the pipe

f = co-efficient of friction = 0.01

* Hydraulic Grade Line (HGL) and Total Energy Line (TEL):



HGL: If the pressure head at the different sections of the pipe are plotted to a scale as vertical ordinates, then joint by straight line which is reffered as HGL

$$HGL = z + \frac{P}{w}$$

TEL: If the pressure head and velocity head at different sections
84 of the pipe are plotted to a scale as vertical ordinate above the assumed datum, the joint by straight line which is reffered as TEL.

$$TEL = z + \frac{P}{w} + \frac{v^2}{2g}$$

* If the length & height of a measuring tank is 45cm and 30.5cm respectively. Collection time is 74s. Initial Reading (IR) = 0 cm, final reading = 10 cm, Find out the head loss.

Soln:

$A = 45 \times 30.5 = 1372.5\, cm^2$, $l = 45\, cm$.

Difference betn FR & IR $= 10 - 0 = 10\, cm$

volm of water $= 1372.5 \times 10 = 13725\, cm^3$

$\therefore Q = V/t = \frac{13725}{74} = 185.47\, cm^3/sec$

$v = Q/A = \frac{185.47}{1372.5} = 0.135\, cm/sec$

$\therefore$ Head Loss, $h_f = \frac{4 f l v^2}{2 g D} =$

* Water flowing through a pipe of 70 mm diameter under a gauge pressure of $3.5\, kg/cm^2$. and with a mean velocity of 1.5 m/sec. The pipe is 7m above datum line. Neglecting friction, determine total head of water.

Soln:

Butt

Total head, $= z + \frac{v^2}{2g} + \frac{p}{\gamma}$

$= 7 + \frac{1.5^2}{2 \times 9.81} + \frac{3.5 \times 100^2}{1000}$

$= 42.11\, m$ Ans.

* A pipe lying at a head of 525 m one end P and other end Q at elevation is 580 m. Diameter of pipe at P & Q are 50 mm & 70 mm. Discharge, Q = 70 lit/s, if pressure at P is 100 kN/m² then find pressure at Q when head loss is 1m. water is flowing from P to Q.

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Solⁿ:

$$V_p = \frac{Q}{A_p} = 35.71 \text{ m/s}$$

$$V_Q = \frac{Q}{A_Q} = 18.23 \text{ m/s}$$

$$z_1 + \frac{V_P^2}{2g} + \frac{P_1}{\gamma} = z_2 + \frac{V_Q^2}{2g} + \frac{P_2}{\gamma} + h_f$$

$$\Rightarrow 525 + \frac{35.71^2}{2\times 9.81} + \frac{100}{9.81} = 580 + \frac{18.23^2}{2\times 9.81} + \frac{P_2}{9.81} + 1$$

$$\Rightarrow P_2 = 2.25 \times 9.81$$

$$= 22.1 \text{ kN/m}^2$$

Ans.

$$A_p = \frac{\pi}{4} \times \left(\frac{50}{1000}\right)^2$$

$$= 1.96 \times 10^{-3} \text{ m}^2$$

$$A_Q = \frac{\pi}{4} \times \left(\frac{70}{1000}\right)^2$$

$$= 3.84 \times 10^{-3} \text{ m}^2$$

$$Q = 70 \text{ lit/s}$$

$$= 0.07 \text{ m}^3/\text{s}$$

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"Hydreology"

*** Hydrology:** The science which deals with the occurrance, circulation and distribution of water of the earth and earth's atmosphere.

Ans/

*** Objectives:**

- Estimation of water resources
- study of processes such as precipitation
- study of problems such as flood.

*** Hydrologic cycle:** The various aspects of water related to the earth can be explained in terms of a cycle known as hydrologic cycle.

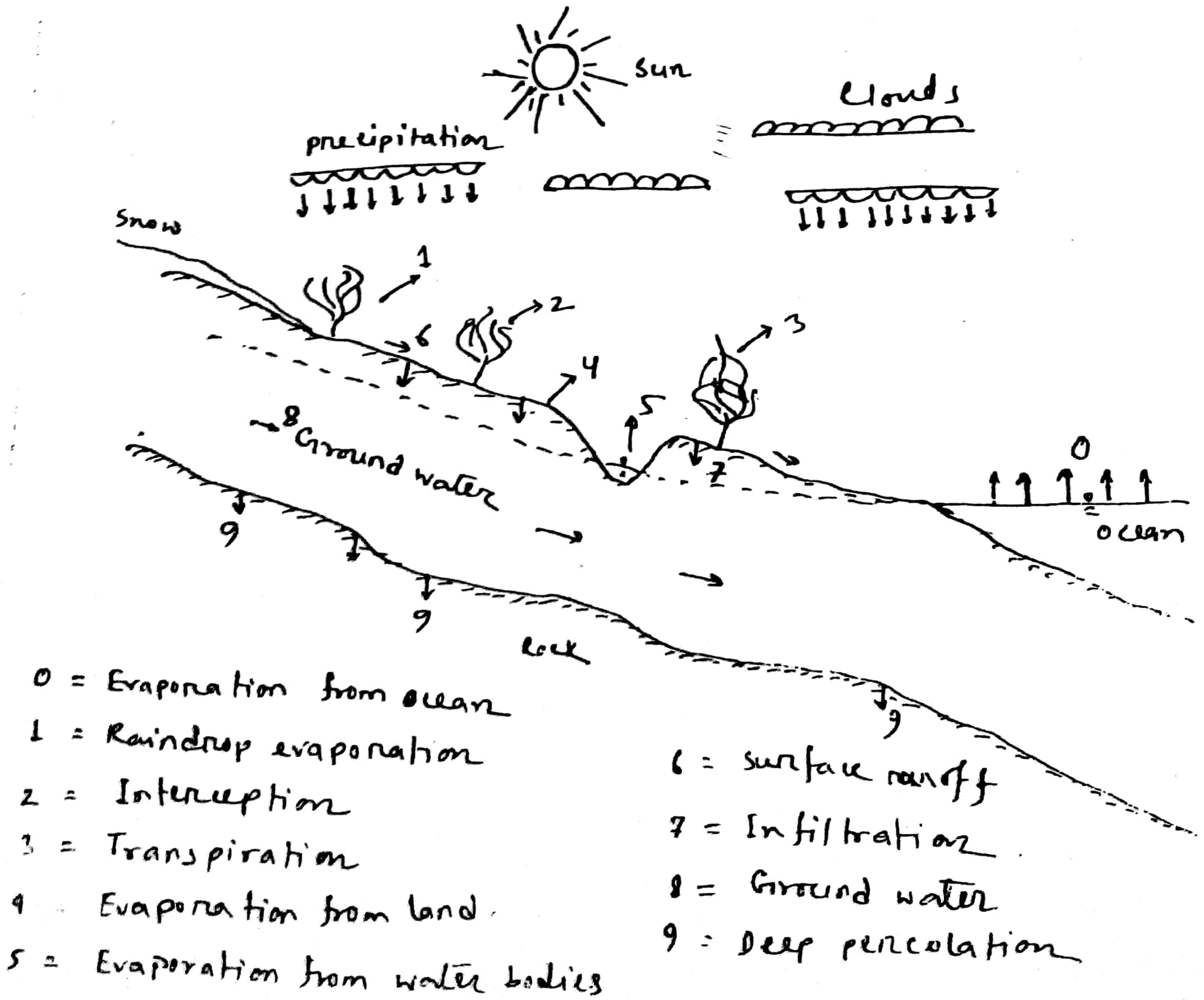

Fig: Hydrologic cycle.

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The area of land draining into a stream or a water course at a given location is known as the catchment area.

Water budget equation:

$$P - R - G - E - T = \Delta S$$

$\Rightarrow$ Mass inflow - Mass outflow = Change in storage

$$\Rightarrow V_i - V_o = \Delta S$$

P = precipitation, R = Surface run off, G = Ground water flow, E = Evaporation, T = Transpiration, ΔS = Change in storage.

* A lake had a water surface elevation of 103.2 m above datum at the beginning of a month. The lake received an inflow 6 m³/s from surface runoff sources. In the same period, the outflow from the lake 6.5 m³/s. The lake received a rainfall 145 mm and the evaporation from the lake surface 6.10 cm. Write water budget equation and calculate the water surface elevation at the end of the month. A = 5000 ha.

Solⁿ: Inflow (volᵐ) - outflow (volᵐ) = change in storage

$$\Rightarrow (\bar{I} \times \Delta t + P \times A) - (\bar{Q} \times \Delta t + E \times A) = \Delta S$$

$$\Rightarrow \Delta S = (6 \times 30 \times 24 \times 3600 + .145 \times 5000 \times 10^4) - (6.5 \times 30 \times 24 \times 3600 + 0.061 \times 5000 \times 10^4)$$

$$= 2.9 \times 10^6 \ m^3$$

$$\therefore \Delta z = \frac{\Delta S}{A} = \frac{2.9 \times 10^6}{5000 \times 10^4} = 0.0508 \ m$$

$\therefore$ Elevation = 103.2 + .0508 = 103.2508 m. Ans.

$$R = P - L$$

where, L = losses (water not available to runoff due to infiltration, evaporation, transpiration and surface storage.

* A small catchment of 150ha area received a rainfall of 10.5 cm in 90 min due to a storm. At outlet, the stream draining the catchment was dry before storm and experienced a runoff lasting for 10 hours with an average discharge of 1.5 m³/s. The stream was again dry after the runoff event. (a) What is the amount of water which was not available to runoff due to combined effect of infiltration, evaporation and transpiration? (b) What is the ratio of runoff to precipitation?

Solⁿ:

a) $P = 150 \times 10^4 \times \frac{10.5}{100} = 157500 \, m^3$

$R = 1.5 \times 10 \times 3600 = 54000 \, m^3$.

$\therefore L = P - R = 103500 \, m^3$

Ans.

b) $\frac{\text{Runoff}}{\text{Rainfall}} = \frac{54000}{157500} = 0.343$

↳ This ratio is known as runoff co-efficient.

Forms of precipitation:

① **Rain:** size - 0.5mm ~ 6mm

Type	Intensity
Light Rain	Trace to 2.5 mm/h
Moderate Rain	2.5 ~ 7.5 mm/h
Heavy Rain	> 7.5 mm/h

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② **Snow:** Consists of ice crystals which usually combine to form flakes.

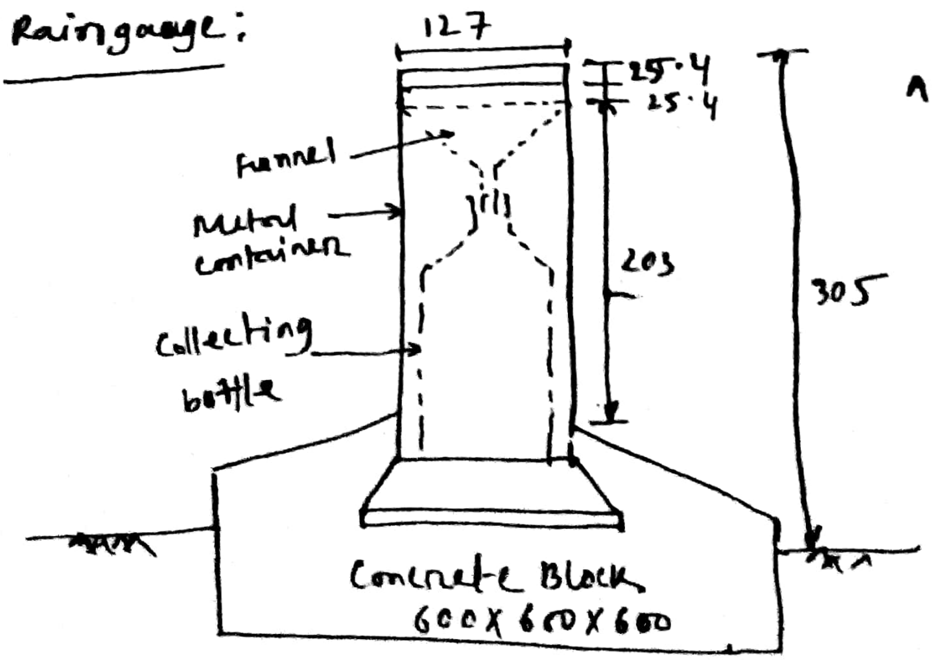
③ **Drizzle:** A fine sprinkle of numerous water droplets of size <0.5mm & intensity 1mm/h.

④ **Glaze:** When rains come contact to cold ground at 0°c, water drops freeze to form an ice coating called glaze.

⑤ **Sleet:** precipitation of snow & rain simultaneously.

⑥ **Hail:** It is a showery precipitation in the form of irregular lumps of ice of size more than 8mm.

Raingauge:



All dimension is in mm

Fig: Non-recording gauge (symons' gauge)

Adequacy of raingauge station:

$$N = \left(\frac{C_v}{\varepsilon}\right)^2 \quad ; \quad \varepsilon = \frac{C_v}{\sqrt{m}}$$

where,
N = optimal Nos. of stations.
ε = Allowable degree of error in the estimation of the mean rainfall.
C_v = co-efficient of variation.

$$C_v = \frac{100 \times \sigma_{m-1}}{\bar{P}}$$

$$\sigma_{m-1} = \sqrt{\frac{\sum_1^m (P_i - \bar{P})^2}{m-1}} = \text{standard deviation.}$$

P_i : precipitation in the ith station.

$$\bar{P} = \frac{1}{m}\left(\sum_1^m P_i\right) = \text{mean precipitation.}$$

* A Catchment has 6 raingauge stations. In a year, the annual rainfall recorded by the gauges are as follows—

Station →	A	B	C	D	E	F
Rainfall (cm) →	82·6	102·9	180·3	110·3	98·8	136·7

a) Determine the standard error in the estimation of mean rainfall in the existing set of raingauges.

b) For a 10% error in the estimation of the mean rainfall, calculate the optimum Nos. of raingauge.

Soln: $m = 6$, $\bar{P} = \frac{82.6 + 102.9 + 180.3 + 110.3 + 98.8 + 136.7}{6} = 118.6 \text{ cm}$

$$\sigma_{m-1} = \sqrt{\frac{(82.6-118.6)^2 + (102.9-118.6)^2 + (180.3-118.6)^2 + (110.3-118.6)^2 + (98.8-118.6)^2 + (136.7-118.6)^2}{6-1}}$$

$= 35.04$ cm

$$C_v = \frac{100 \times \sigma_{m-1}}{\bar{P}} = 29.54$$

a) $\varepsilon = \frac{C_v}{\sqrt{m}} = 12.06\%$

b) $N = \left(\frac{C_v}{\varepsilon}\right)^2 \quad [\varepsilon = 10\%]$

$= \left(\frac{29.54}{10}\right)^2 \approx 9$

∴ Additional raingauge $= 9 - 6 = 3$ Ans.

Estimation of missing data:

Station = 1, 2, 3, ..., m

Annual precipitation = P_1, P_2, P_3 ....., P_m

Missing station = X

Missing precipitation = P_x

normal rainfall values

$$D = \frac{\text{Max}^m \text{ precipitation} - \text{Min}^m \text{ precipitation}}{\text{Max}^m \text{ precipitation}} \times 100$$

↳ normal rainfall

If $D \leq 10\%$

$$P_x = \frac{1}{m}\left[P_1 + P_2 + P_3 + \dots + P_m\right]$$

If $D > 10\%$

$$P_x = \frac{N_x}{m}\left(\frac{P_1}{N_1} + \frac{P_2}{N_2} + \dots + \frac{P_m}{N_m}\right)$$

* The normal annual rainfall at stations A, B, C and D in a basin are 81·97, 67·59, 76·28 and 92·01 cm. In the year 1985, the station D was inoperative and the station A, B and C recorded annual precipitation of 91·11, 72·23 and 79·89 cm respectively. Det^n the rainfall at station D.

Sol^n:

$$D = \frac{92.01 - 67.59}{92.01} \times 100 = 26.54\%$$

$\because D > 10\%$

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$$\boxed{m = 3}$$

$$\therefore P_D = \frac{N_D}{m}\left(\frac{P_A}{N_A} + \frac{P_B}{N_B} + \frac{P_C}{N_C}\right)$$

$$= \frac{92.01}{3}\left(\frac{91.11}{81.97} + \frac{72.23}{67.59} + \frac{79.89}{76.28}\right)$$

$$= 99.49 \text{ cm}$$

Ans.

* precipitation: It denotes all forms of water that reach the earth from the atmosphere.

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Frequency of point rainfall:

The probability of occurrence of a rainfall of magnitude equal to in excess of a specified magnitude x is denoted by P.

$$T = 1/P$$

T = Recurrence interval / Return period.

The probability of occurrence of a event r times in n successive years –

$$P_{r,n} = {}^{n}C_r \, P^r q^{n-r} = \frac{n!}{(n-r)!\,r!} P^r q^{n-r}$$

(a) occurring 2 times in n successive years.

$$P_{2,n} = \frac{n!}{(n-2)!\,2!} P^2 q^{n-2}$$

(b) not occurring at all in n successive years –

$$P_{0,n} = q^n = (1-P)^n$$

(c) occurring at least once in n successive years –

$$P_1 = 1 - q^n = 1 - (1-P)^n$$

* Analysis of data on maximum one-day rainfall depth at Chennai indicated that a depth of 300mm had a return period of 50 years. Determine the probability of a one-day rainfall depth equal to or greater than 300mm occurring (a) once in 20 successive years (b) two times in 15 successive years (c) at least once in 20 successive years.

Soln:

$$T = \frac{1}{P} \Rightarrow P = \frac{1}{T} = \frac{1}{50} = 0.02$$

$$\therefore q = 1 - P = 0.98$$

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(a) $P_{1,20} = \dfrac{20!}{(20-1)!\,1!} p^1 q^{20-1}$.

$= \dfrac{20!}{19!\,1!} (0 \cdot 02)^1 (0 \cdot 98)^{19} = 0 \cdot 272$

(b) $P_{2.15} = \dfrac{15!}{(15-2)!\,2!} (0 \cdot 02)^2 (0 \cdot 98)^{15-2} = 0 \cdot 323$

(c) $P_1 = 1 - q^n = 1 - (0 \cdot 98)^{20} = 0 \cdot 32$

Ans.

Evaporation: It is the process in which a liquid changes to the gaseous state at the free surface, below the boiling point through the transfer of heat energy.

The rate of evaporation depends on —

i) vapour pressure
ii) Air & water temperature
iii) Wind Speed
iv) Atmospheric pressure.
v) Quality of water.
vi) Size of water body.

Empirical Evaporation Equations:

① Dalton-type Equation:

$$E_L = K f(u) (e_w - e_a)$$

where,

E_L = lake evaporation in mm/day.

e_w = Saturated vapour pressure in mm of mercury

e_a = Actual " " " " "

$f(u)$ = wind-speed correction function.

② Meyer's Formula:

$$E_L = K_m (e_w - e_a) \left(1 + \frac{u_9}{16}\right)$$

u_9 = monthly mean wind velocity in km/h at about 9m above ground.

$K_m = 0.36$

* A reservoir with a surface area of 250 hectares had the following average values of climate parameters during a week. water temperature = 20°c, Relative humidity = 40%, wind speed at 1m above ground surface = 16 km/h. Estimate the average daily evaporation from the lake by using Meyer's formula.

Sol^n:

$$e_w = 4.589 \times e^{\frac{17.27t}{237.3+t}}$$

$$= 17.55 \text{ mm of Hg}$$

$$e_a = 0.4 \times e_w \quad 40\%$$

$$= 7.02 \text{ mm of Hg}$$

$$t = 20°c$$

$$u_9 = u_1 * (9)^{1/7}$$

$$= 16 \times (9)^{1/7}$$

$$= 21.9 \text{ km/h}$$

$$\therefore E_L = 0.36\,(e_w - e_a)\left(1 + \frac{u_9}{16}\right)$$

$$= 8.97 \text{ mm/day}$$

Ans.

Water Harvesting: It is the process of collecting and concentrating runoff water from a run-off area into a run-on area, where the collected water is either directly applied to the cropping area and stored in the soil profile for immediate use by the crop.

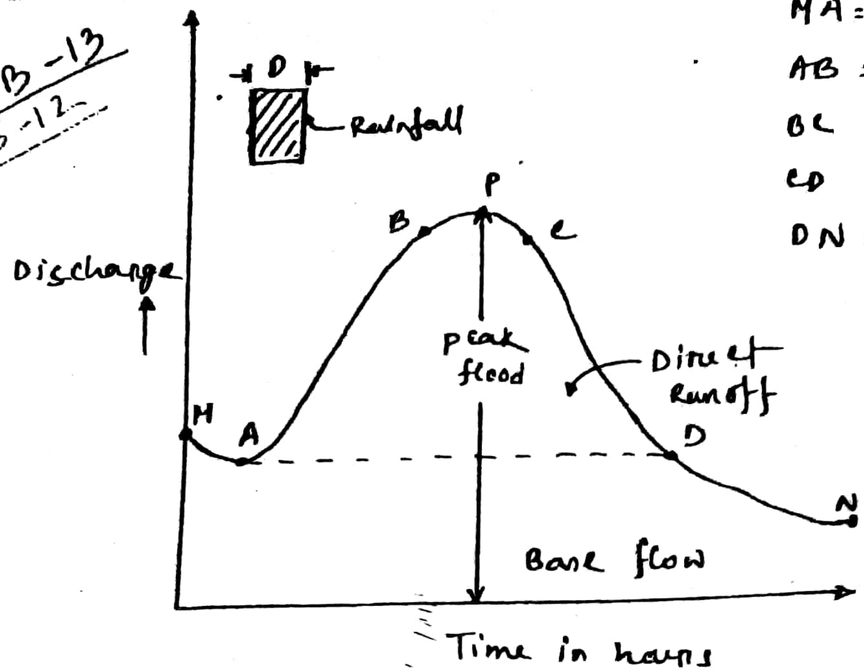
Water Harvesting

- Rain water harvesting
- Flood water harvesting.

Hydrograph: It is a graphical representation of discharge in a stream plotted against time chronologically.

Elements of a flood hydrograph:

BWDB-13
PCB-12



MA = Base flow recession
AB = Rising limb
BC = Crest segment
CD = Falling limb
DN = Base flow recession

Fig: Elements of flood hydrograph.

Factors affecting run off / flood hydrograph:

A. Physiological Factors:

1. Basin characteristics:

a) Shape b) size c) slope d) elevation e) nature of valley.

RCL-14 2. Infiltration characteristics:

a) land use and cover b) soil type & geological conditions.

3. channel characteristics:

a) cross-section b) roughness.

B. Climatic factors:

1. precipitation, intensity, duration & magnitude of storm.

2. Initial loss.

3 Evapotranspiration.

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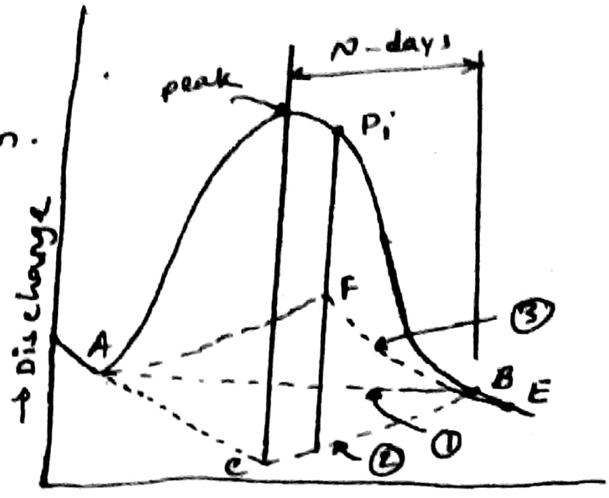
Base Flow Separation Method:

① Straight line method

② Fixed base length separation.

③ Variable slope separation



Time

Effective Rainfall / Excess Rainfall: It is that part of the rainfall that becomes direct runoff at the outlet of the watershed.

Hyetograph: A hyetograph is a plot of the intensity rainfall against the time interval. It is usually represented as a bar chart.

Point Rainfall / Station Rainfall: It refers to the rainfall data of a station.

Rainfall of magnitude 3.8 cm and 2.8 cm occurring on two consecutive 4-h durations on a catchment of area 27 km² produced the following hydrograph of flow at the outlet of the catchment. Find rainfall excess & φ index.

Wasa-17

Time (h) →	−6	0	6	12	18	24	30	36	42	48	54	60	66
Flow (m^3/s) →	6	5	13	26	21	16	12	9	7	5	5	4.5	4.5

Soln: Here, Base flow = 5 m^3/s

∴ ~~Flow~~

Time (h) →	0	6	12	18	24	30	36	42	48
Flow without Base Flow (m^3/s) →	0	8	21	16	11	7	4	2	0

$\therefore$ Runoff vol^m $= (6\times60\times60)\left[\frac{1}{2}(0+8)+\frac{1}{2}(8+21)+\frac{1}{2}(21+16)\right.$
$\left.+\frac{1}{2}(16+11)+\frac{1}{2}(11+7)+\frac{1}{2}(7+4)+\frac{1}{2}(4+2)+\frac{1}{2}(2)\right]$

$= 1.4904\times10^6\ m^3$

$\therefore$ Runoff depth $= \dfrac{\text{Run off vol}^m}{\text{Catchment Area}} = \dfrac{1.4904\times10^6}{27\times10^6} = 0.0552\,m$.

$= 5.52\ cm$ (rainfall excess).

Ans.

Total rainfall $= 3.8+2.8 = 6.6\ cm$

Duration $= 4+4 = 8$ hrs.

$\therefore \phi$ index $= \dfrac{6.6-5.52}{8} = 0.135\ cm/h$

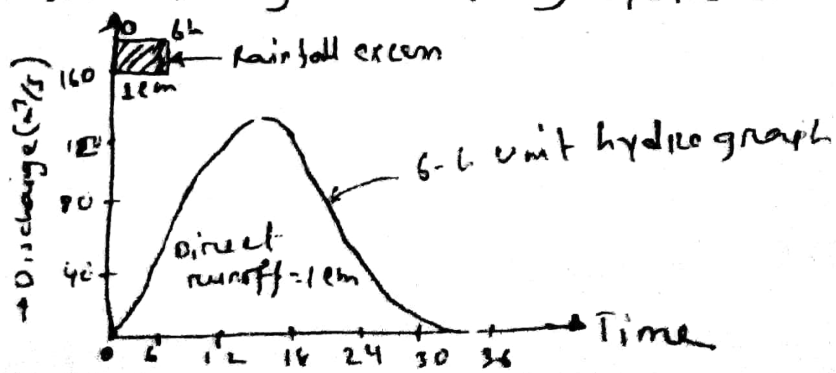
Ans.

 # Unit Hydrograph: A unit hydrograph is defined as the hydrograph of direct runoff resulting from one unit depth (1cm) of rainfall excess occurring uniformly over the basin and at a uniform rate for a specified duration (D hrs).

 # S-curve / s-hydrograph: It is a hydrograph produced by a continuous effective rainfall at a constant rate for an infinite period.

 # Uses of unit hydrograph:

1. The development of flood hydrographs for extreme rainfall magnitudes for use in the design of hydraulic structures.
2. Extension of flood flow records based on rainfall records.
3. Development of flood forecasting & warning system based on rainfall.



* The peak of flood hydrograph due to a 3h duration isolated storm in a catchment is 270 m^3/s. The total depth of rainfall is 5.9 cm. Assuming an avg. infiltration loss of 0.3 cm/h and a constant base flow of 20 m^3/s, estimate the peak of the 3-h unit hydrograph of this catchment.

(b) If the area of the catchment is 567 km^2, determine the base width of the 3-h unit hydrograph by assuming it to be triangular in shape.

Soln: Rainfall excess = Total rainfall − Loss

$= 5.9 - (.3 \times 3)$

$= 5$ cm

peak of DRH $= 270 - 20 = 250\ m^3/s$

∴ peak of 3h unit hydrograph $= \dfrac{\text{peak of DRH}}{\text{rainfall excess}}$

$= \dfrac{250}{5} = 50\ m^3/s$ Ans.

(b) Let, Base width = B in hours.

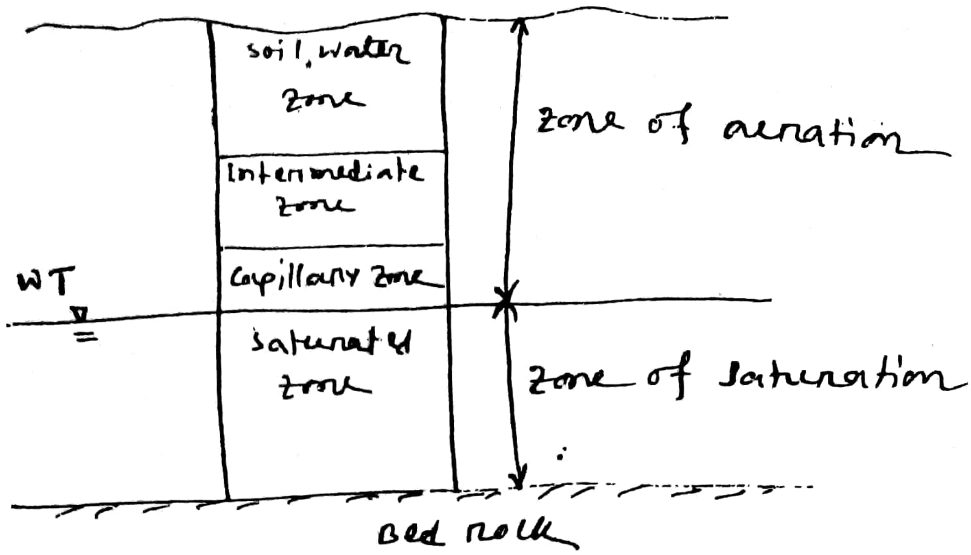
volm represented by the area of UH = Area of catchment × 1 cm

$\Rightarrow \frac{1}{2} B \times 60 \times 60 \times 50 = 567 \times 10^6 \times \frac{1}{100}$

$\Rightarrow B = 63$ hrs

Ans.

Forms of subsurface water:



Zone of aeration: Soil pores are occupied partially by water & partially by air. The space between the land surface and the water table marks the extent of this zone.

Zone of saturation: The space in which all the pores of the soil filled with water is called zone of saturation or ground water zone.

 # Aquiclude: A saturated but relatively impermeable formation which transmit little water. Example – Clay.

 # Aquitard: It is a formation through which only seepage is possible. It is partly permeable. Ex – Sandy clay.

 # Aquifuge: It is a formation which is neither porous nor permeable. It cannot transmit water. Ex – Rock.

 # Aquifer: It is a formation having an ability to store and transmit water.

* The value of peak of a 6-h flood hydrograph is 35 m^3/s and constant base flow is 10 m^3/s. If the rainfall depth is 3.25 cm and infiltration rate is 0.125 cm/h, Determine peak of 6-h unit hydrograph and run off co-efficient ?

Soln: Rainfall excess $= 3.25 - 6 \times .125$

$= 2.5$ cm

BWDB-..6

peak of DRH $= 35 - 10 = 25\ m^3/s$

peak of 6-h unit hydrograph $= \dfrac{25}{2.5}$

$= 10\ m^3/s$

Run-off co-efficient $= \dfrac{\text{Run off}}{\text{Rainfall}} = \dfrac{2.5}{3.25}$

$= 0.77$

Ans.

* The hourly ordinates of 6-h unit hydrograph for a particular basin is 36 cumec. The flood peak due to 6-h strom was 130 cumec. The constant base flow is 6 cumec and avg. strom loss 6 mm/hr. Determine the depth of storm rainfall and the stream flow at successive 3hr interval. Assume the wall surface.

Soln:

peak of DRH $= 130 - 6 = 124\ m^3/s$

peak of UH $= 36\ m^3/s$

Buet

∴ peak of 6-h UH $= \dfrac{\text{peak of DRH}}{\text{Rainfall excess}}$

⇒ Rainfall excess $= \dfrac{124}{36} = 3.44$ cm

∴ storm Rainfall depth $= 3.44 + (0.6 \times 6)$

$= 7.04$ cm

Ans.

∴ Direct Runoff $= 36 \times 3.44 = 123.84\ m^3/s$ [1 cm → 36, 3.44 cm → 123.84]

∴ Stream flow $= 123.84 + 6$

$= 129.84\ m^3/$.

02

Open channel Flow

 # Open channel Flow/ Free Surface Flow: It may be defined as the flow of water with a free surface (subjected to atmospheric pressure).

Example:

1. Flows in river, lake, canals, etc.
2. Flow in a submerged sewer or in a culvert may be open channel flow, if the flow occurs with a free surface.
3. The flow of ground water with free surface.

 # Flow Mechanism: Open channel flow occurs under the action of gravity and atmospheric pressure. Basically, all open channels have a bottom slope and flow occurs downstream along the slope. The component of gravity force on the weight of water along the slope acts as the driving force. For open channel flow to occur, total energy at upstream must be greater than the total energy at down stream.

Difference between openchannel flow & pipe flow.

Open Channel Flow	Pipe Flow
1. Flow occurs due to gravity.	1. Flow occurs due to difference in pressure.

Geometric Elements of OCF:

B = Top width.

P = wetted perimeter

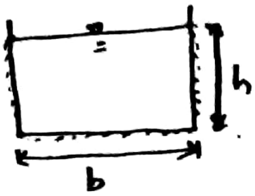
b = bottom width.

h = water height.

R = Hydraulic radius $= A/P$

D = Hydraulic depth $= A/B$

① Rectangular channel:



$A = bh$

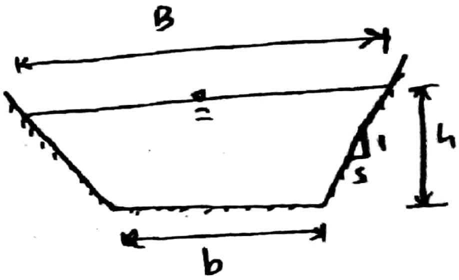
$P = b + 2h$

$B = b$

$D = A/B = \frac{bh}{b} = h$

$R = A/P$

② Trapezoidal channel:



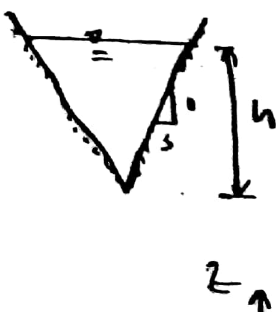
$B = b + 2sh$

$A = (b + sh)h$

$P = b + 2\sqrt{1+s^2}\,h$

$R = A/P$, $D = A/B$

③ Triangular channel:



$B = 2sh$

$A = sh^2$

$P = 2\sqrt{1+s^2}\,h$

$R = A/P$, $D = A/B$

④ parabolic channel:

$A = \frac{2}{3}Bh$

$B = 2\sqrt{h/c}$

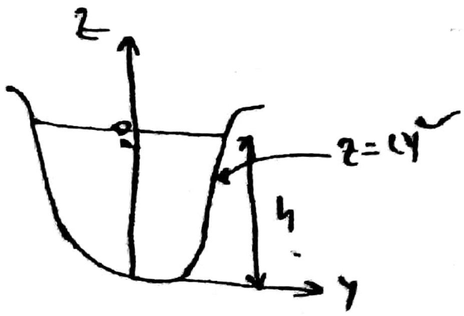
$P = B + \frac{8h^2}{3B} \quad \left(0 < \frac{4h}{B} \leq 1\right)$

let,

$y^2 = 4z$

$\Rightarrow z = \frac{1}{4}y^2$

$\therefore c = \frac{1}{4}$



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(4) Circular channel:

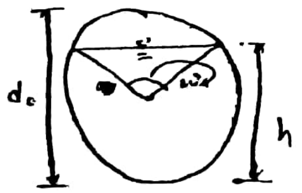
$B = d_0 \sin(w/2)$

$A = (w - \sin w)\, d_0^2/8$

$h = d_0 \left[1 - \cos(w/2)\right]/2$

$P = w d_0/2$

$w = 2\cos^{-1}(1 - 2h/d_0)$



Wide channel: when the width of a rectangular channel is very large compared to the depth i.e. $b >> h$ $(b \geq 10h)$, the sides of channel have no influence on the velocity distribution in the central region. Such a channel is known as wide channel.

For wide channel.

$$R = A/P = \frac{bh}{b+2h} \approx \frac{bh}{b} \approx h$$

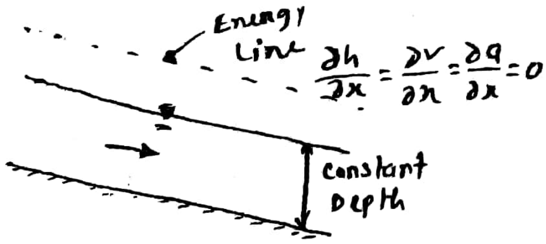
Classification of OCF:

- open channel flow
  - Steady →
    - Uniform Flow
    - Gradually varied flow
    - Rapidly varied flow
    - Spatially varied flow.
  - Un Steady →
    - Gradually varied Flow
    - Rapidly varied Flow
    - Spatially varied Flow

(with respect to time) (with respect to space)

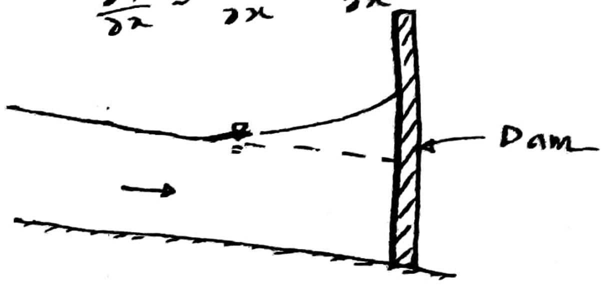
Steady Flow: $\left(\frac{\partial h}{\partial t} = \frac{\partial v}{\partial t} = \frac{\partial Q}{\partial t} = 0 \text{ for a fixed section}\right)$

BWDB-12

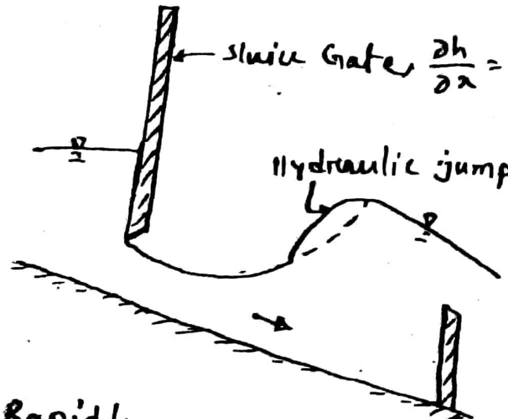


1) Uniform Flow - Flow in a prismatic channel with constant discharge & velocity

$\frac{\partial h}{\partial x} \approx \frac{\partial v}{\partial x} \approx \frac{\partial Q}{\partial x} \approx 0$

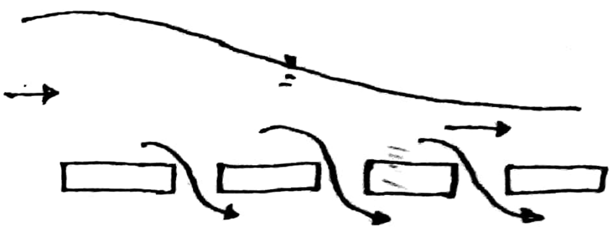


2) Gradually varied Flow - Flow behind a dam



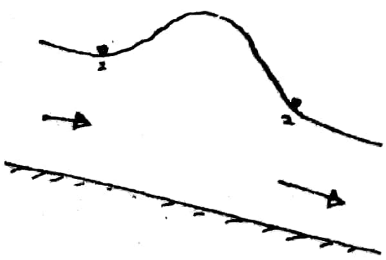
3) Rapidly varied Flow - Hydraulic jump

$\frac{\partial Q}{\partial x} \neq 0$

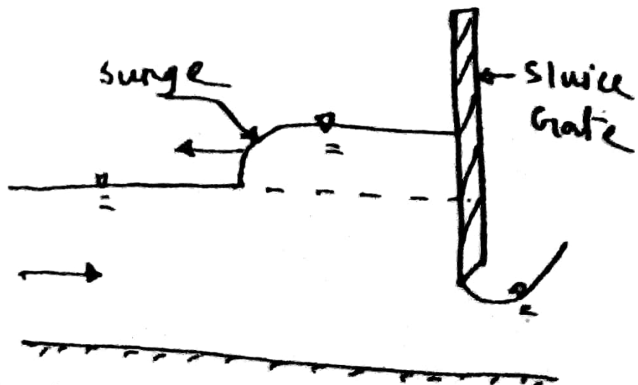


4) Spatially varied Flow - Flow over a bottom rack.

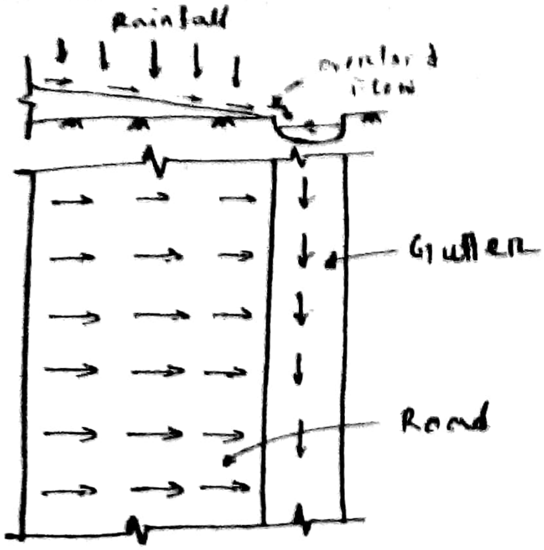
Unsteady Flow: $\left(\frac{\partial h}{\partial t} \neq \frac{\partial v}{\partial t} \neq \frac{\partial Q}{\partial t} \neq 0 \text{ for a fixed section}\right)$



1. Unsteady gradually varied Flow - Flood wave



2. Unsteady rapidly varied Flow - Surge produced by sudden closure of gate.



3) Unsteady spatially varied Flow – overland Flow and Flow in a Roadside gutter.

Effect of viscosity:

Reynold's number, $Re = \frac{VR}{\nu} = \frac{\text{Inertia Force}}{\text{Viscous Force}}$

For water at 20°c, $\nu = 10^{-6} m^2/s$

Classification:

1. $Re < 500$, Flow is laminar
2. $500 \leq Re \leq 12500$, Flow is transitional.
3. $Re > 12500$, Flow is turbulent.

prove that, most of flow in nature are turbulent.

proof:

$Re = \frac{VR}{\nu}$

$= 10^6 >> 12500$

$V = 1 m/sec$

$\nu = 10^{-6} m^2/s$

$R = 1 m$

$\therefore$ natural flows are turbulent.

(proved)

Effect of Gravity:

Froude Number, $F_r = \sqrt{\dfrac{\text{Inertia force}}{\text{Gravity force}}} = \dfrac{V}{\sqrt{gD}}$ → 1.00

where, D = Hydraulic depth $= A/B$

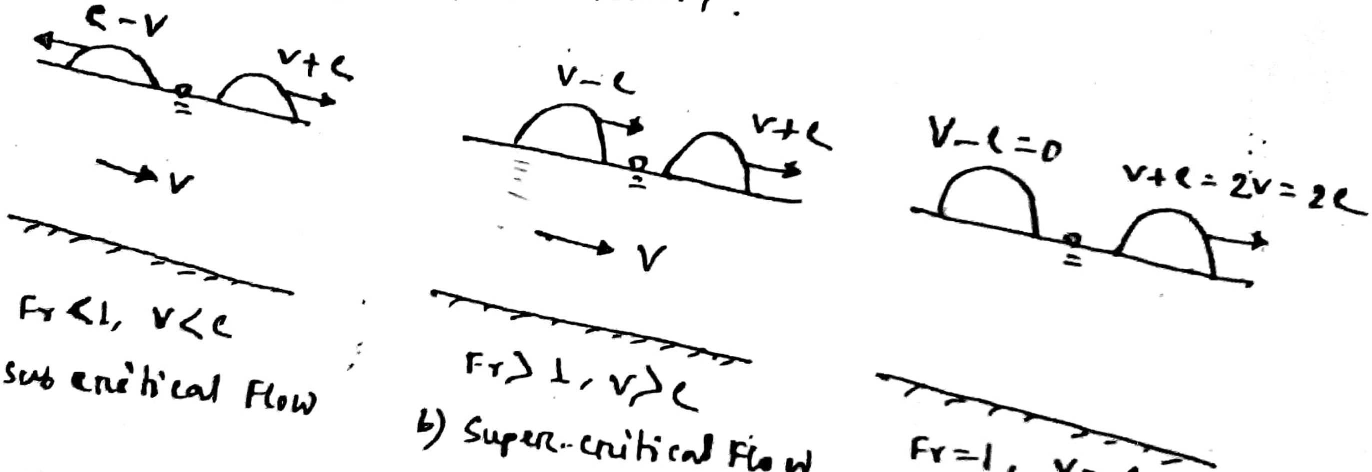
Classification:

1. Sub-critical Flow, $F_r < 1$; $V < \sqrt{gD}$; $V < c$
2. Critical Flow, $F_r = 1$; $V = \sqrt{gD}$; $V = c$
3. Super-critical Flow, $F_r > 1$; $V > \sqrt{gD}$; $V > c$

Again,

$$F_r = \frac{V}{c} \qquad [\because c = \sqrt{gD}]$$

where, c = wave celerity.



a) Sub critical Flow

b) Super-critical Flow

c) Critical Flow.

* An oil ($s = 0.85$, $\nu = 1.8 \times 10^{-5}\ m^2/s$) flows in a 10cm dia pipe at 0.5 L/sec Is the flow laminar or turbulent?

Soln.

$$Re = \frac{VR}{\nu}$$

$$= \frac{0.0637 \times 0.1}{1.8 \times 10^{-5}}$$

$$= 353.33 < 500$$

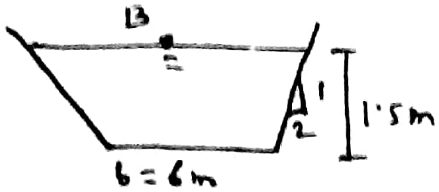
∴ Flow is laminar.

Ans.

$$V = \frac{Q}{A} = \frac{.5 \times 10^{-3}}{\frac{\pi}{4}(.10)^2} = 0.0637\ m/s$$

$R = 10cm = 0.1m$

* A trapezoidal channel has a bottom width of 6m and side slopes of 2:1. Compute the discharge and determine the state of flow in this channel if the depth of flow is 1.5m and the mean velocity of flow is 2.30 m/s. If elementary waves are created in this channel, determine the speed of the wave fronts upstream & downstream.



solⁿ: $B = b + 2sh = 6 + 2\times2\times1.5 = 12\,m$

$A = (b+sh)h = 13.5\,m^2$

$P = b + 2\sqrt{1+s^2}\,h = 12.71\,m$

$R = A/P = 1.06\,m$

$D = A/B = 1.13\,m$

$\therefore Q = AV = 13.5 \times 2.3 = 31.05\ m^3/s$

$Re = \dfrac{VR}{\nu} = \dfrac{2.3\times1.06}{10^{-6}} = 2.44\times10^6 > 12500$

$Fr = \dfrac{V}{\sqrt{gD}} = \dfrac{2.3}{\sqrt{9.81\times1.13}} = 0.69 < 1$

$\therefore$ Flow is subcritical turbulent.

Again, $c = \sqrt{gD} = 3.33\ m/s$

$\therefore$ Speed of wave fronts upstream $= c - V = 3.33 - 2.3$
$= 1.03\ m/s$

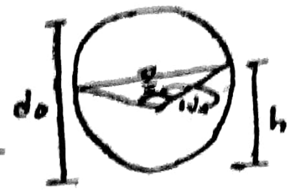
" " " " downstream $= c + V = 3.33 + 2.3$
$= 5.63\ m/s$

Ans.

\# Combined effect of viscosity & Gravity:

* A Circular channel 2.75 m in diameter carries a discharge of 6.55 m^3/s at a depth of 1.1 m. Determine the State of flow.

<u>Sol^n</u>: $d_0 = 2.75$ m, $Q = 6.55\ m^3/s$, $h = 1.1$ m



$$w = 2\cos^{-1}\left(1 - \frac{2h}{d_0}\right) = 157^\circ = 2.74 \text{ rad}$$

$$A = (w - \sin w)\, d_0^2/8 = (2.74 - \sin 2.74) \times \frac{2.75^2}{8}$$

$$= 2.22\ m^2$$

$$P = w d_0/2 = \frac{2.74 \times 2.75}{2} = 3.77\,m$$

$$B = d_0 \sin(w/2) = 2.75 \times \sin\left(\frac{2.74}{2}\right) = 2.69\,m$$

$$R = A/P = 0.59\,m, \quad D = A/B = 0.82\,m$$

$$V = Q/A = \frac{6.55}{2.22} = 2.95\ m/s$$

$$Re = \frac{VR}{\nu} = \frac{2.95 \times 0.59}{10^{-6}} = 1.74 \times 10^6 > 12500$$

$$Fr = \frac{V}{\sqrt{gD}} = \frac{2.95}{\sqrt{9.81 \times 0.82}} = 1.09 > 1$$

$\therefore$ The Flow is supercritical turbulent.

* Flow modulus of a city for 25% area is 2.84, for 50% area is 3.12, for 15% area is 1.32, for 10% area is 1.82. Find the combined flow modulus of the city.

<u>Sol^n</u>:

Bult

$$\text{Combined flow modulus} = \frac{25 \times 2.84 + 50 \times 3.12 + 15 \times 1.32 + 10 \times 1.82}{25 + 50 + 15 + 10}$$

$$= 2.65$$

<u>Ans.</u>

velocity Distribution:

If $h < 0.61m\,(2ft)$; Avg. velocity, $V \simeq V_{0.6h}$

If $h > 0.61m\,(2ft)$; Avg. velocity, $V = \frac{V_{0.2h}+V_{0.8h}}{2}$

velocity Distribution Co-efficient:

$$\alpha = \frac{\int_A u^3 dA}{U^3 A} = \frac{\Sigma u^3 \Delta A}{U^3 A}$$

$$\beta = \frac{\int u^2 dA}{U^2 A} = \frac{\Sigma u^2 \Delta A}{U^2 A}$$

$\alpha > \beta > 1$

For uniform velocity distribution, $\alpha = \beta = 1$

Where,

α = Kinetic energy co-efficient / coriolis co-efficient

β = Momentum co-efficient.

For wide channel:

① $q = \int_0^h u\,dz$ ③ $\alpha = \frac{\int_0^h u^3 dz}{U^3 h}$

② $U = \frac{1}{h}\int_0^h u\,dz$ ④ $\beta = \frac{\int_0^h u^2 dz}{U^2 h}$

For one dimensional flow, take h in lieu of A

* In a wide channel the velocity varies along a vertical as $u = 1 + 3z/h$, where h is the total depth and u is the velocity at a depth z from the channel bottom.

① Compute the discharge per unit width.

② Determine the state of flow, h = 5m.

③ Compute velocity distribution co-efficient.

Sol^n:

① $U = \frac{1}{h}\int_0^h u\,dz = \frac{1}{h}\int_0^h \left(1+\frac{3z}{h}\right)dz = \frac{1}{h}\left[z + \frac{3}{h}\frac{z^2}{2}\right]_0^h$

$= \frac{1}{h}\left(h + \frac{3}{2}h\right) = \frac{1}{h} \times 2.5h = 2.5\ m/s$

$\therefore q = Uh$

$= 2.5 \times 5 = 12.50\ m^2/s$

Ans.

(ii) $Re = \frac{VR}{\nu} = \frac{2.5 \times 5}{10^{-6}}$ [For wide channel $R = h$]

$= 12.5 \times 10^{6} > 12500$

$Fr = \frac{U}{\sqrt{gD}} = \frac{2.5}{\sqrt{9.81 \times 5}} = 0.36 < 1$

$\therefore$ Flow is subcritical turbulent.

(iii) $\alpha = \frac{\int_0^h u^3 dz}{U^3 h}$

$= \frac{1}{U^3 h} \int_0^h \left(1 + 3\frac{z}{h}\right)^3 dz$

$= \frac{1}{U^3 h} \int_0^h \left(1 + 9\frac{z}{h} + 27\frac{z^2}{h^2} + 27\frac{z^3}{h^3}\right) dz$

$= \frac{21.25h}{U^3 h}$

$= \frac{21.25}{2.5^3}$

$= 1.36$

$\beta = \frac{\int^h u^2 dz}{U^2 h}$

$= \frac{1}{U^2 h} \int_0^h \left(1 + 3\frac{z}{h}\right)^2 dz$

$= \frac{1}{U^2 h} \int_0^h \left(1 + 6\frac{z}{h} + 9\frac{z^2}{h^2}\right) dz$

$= \frac{7h}{U^2 h}$

$= \frac{7}{2.5^2}$

$= 1.12$

$\therefore \alpha$ & β doesn't depen on h if u is in terms of z/h.

Pressure Distribution in Curvilinear Flow:

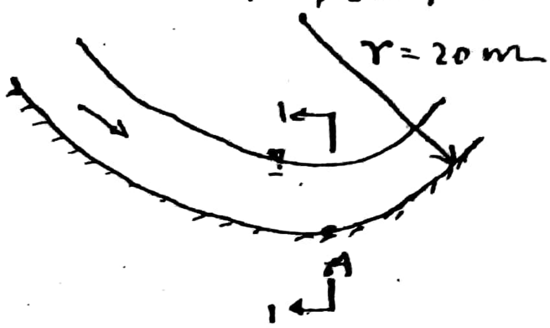
Total pressure head $= h \pm \frac{1}{g} h \frac{U^2}{r} = h\left(1 \pm \frac{1}{g}\frac{U^2}{r}\right)$

Total pressure $= \rho g h \left(1 \pm \frac{U^2}{gr}\right)$

(+) sign for concave flows

(−) sign for convex flows.

* A spillway flip bucket has a radius of curvature of 20 m. If the flow depth at section 1-1 is 3 m and the discharge per unit width is 66 m^2/s, compute pressure at A.



Soln:

$U = \frac{q}{h} = \frac{66}{3} = 22\ m/s$

$\therefore P = \gamma h\left(1 + \frac{U^2}{gr}\right) = \rho g h\left(1 + \frac{U^2}{gr}\right)$ → For concave

$= 1000 \times 9.81 \times 3 \times \left(1 + \frac{22^2}{9.81 \times 20}\right)$

$= 102030\ N/m^2$ Ans.

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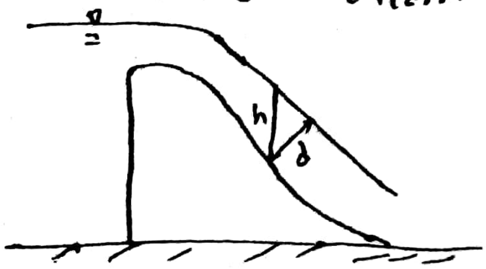
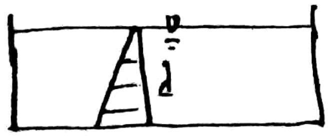
* prove that, the shear force and the overturning moment on the side walls of a steep rectangular channel are $\frac{1}{2}\gamma h^2 \cos^3\theta$ and $\frac{1}{6}\gamma h^3 \cos^4\theta$, where h is the depth of flow and θ is the bottom slope of the channel.



Sol^n:



$\gamma d \cos\theta$

$d = h\cos\theta$

$\therefore$ Shear Force $= \frac{1}{2}\gamma d \cos\theta * d$

$= \frac{1}{2}\gamma (h\cos\theta)\cos\theta \times h\cos\theta$

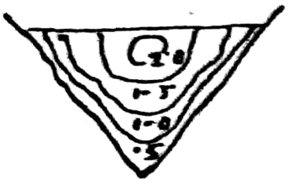
$= \frac{1}{2}\gamma h^2 \cos^3\theta$ (proved)

overturning moment $= f \times d/3$

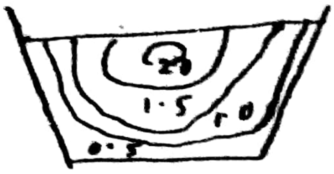
$= \frac{1}{2}\gamma h^2 \cos^3\theta \times \frac{h\cos\theta}{3}$

$= \frac{1}{6}\gamma h^3 \cos^4\theta$ (proved)

* Typical curves of equal velocity in various channel sections

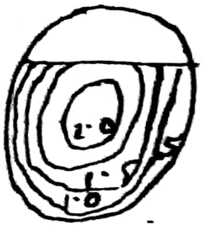


Triangular channel



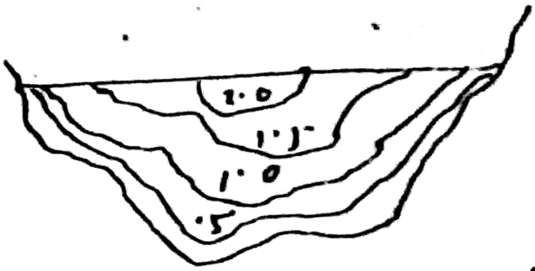
Trapezoidal channel

Narrow rectangular channel



Pipe

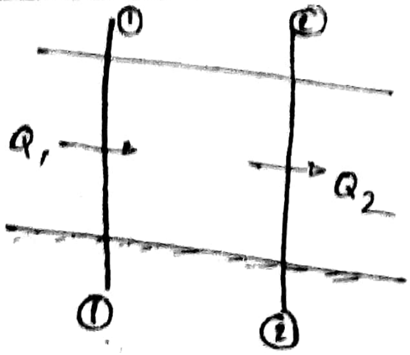
Shallow ditch



Natural irregular channel.

"Governing Equations"

Continuity Equation:



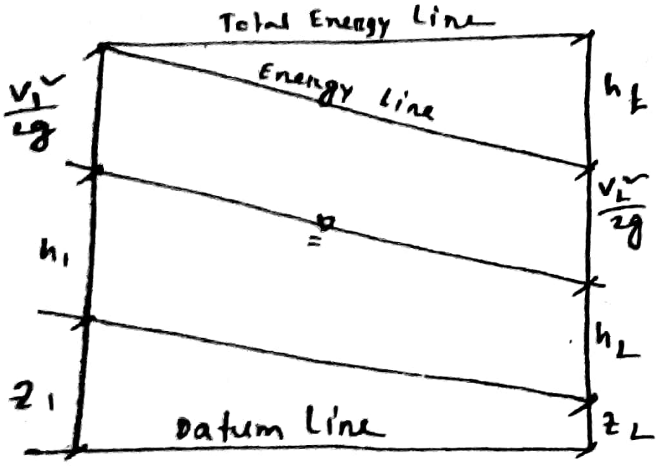
At different section, discharge is constant.

$$Q_1 = Q_2$$

$$\Rightarrow A_1V_1 = A_2V_2 = A_3V_3 \cdots\cdots$$

Energy Equation:

For a perfect incompressible liquid, flowing in a continuous stream, the total energy of a particle remains the same, while the particle moves from one point to another.



$$Z_1 + h_1 + \frac{V_1^2}{2g} = Z_2 + h_2 + \frac{V_2^2}{2g} + h_L$$

→ Bernoulli energy equation.

Momentum Equation:

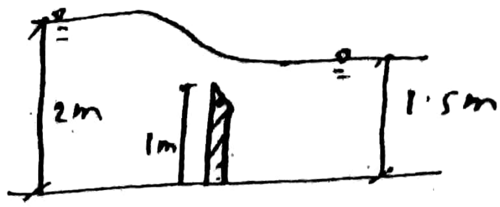
The momentum equation is based on Newton's second law of motion which states that the algebraic sum of all the external forces acting on a fluid mass in any particular direction is equal to the time rate of change of momentum in that direction.

$$\rho Q(\beta_2 V_2 - \beta_1 V_1) = P_1 - P_2 + W\sin\theta - F_f$$

where,

$\rho Q \rightarrow$ mass
$V \rightarrow$ velocity } → momentum

* Figure shows a sharp crested weir in a rectangular channel. If the discharge per unit width of the weir is 4 m^2/s. Estimate the energy loss due to the weir and force on the weir plate for the submerged flow condition.



Soln:

$$V_1 = \frac{q}{h_1} = \frac{4}{2} = 2\ m/s$$

$$V_2 = \frac{q}{h_2} = \frac{4}{1.5} = 2.67\ m/s.$$

we know,

$$z_1 + h_1 + \frac{V_1^2}{2g} = z_2 + h_2 + \frac{V_2^2}{2g} + h_L \qquad [z_1 = z_2 = 0]$$

$$\Rightarrow h_L = h_1 - h_2 + \frac{V_1^2}{2g} - \frac{V_2^2}{2g}$$

$$= 0.34\ m \text{ of water}$$

Ans.

Applying momentum equation.

$$\rho q (V_2 - V_1) = \frac{1}{2}\gamma h_1^2 - \frac{1}{2}\gamma h_2^2 - F$$

$$\Rightarrow 4 \times 1000 (2.67 - 2) = \frac{1}{2} \times 1000 (2^2 - 1.5^2) - F$$

$$\Rightarrow F = 5917.18\ N$$

Ans.

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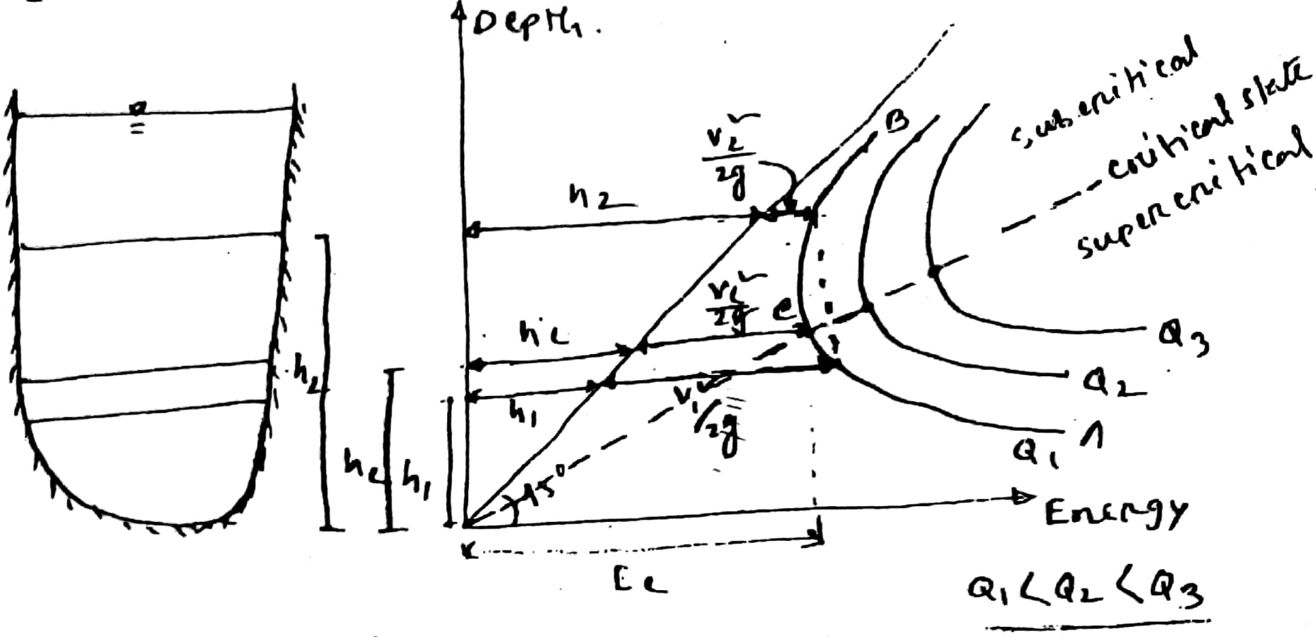
Specific Energy: sp. energy at a channel section may be defined as the energy measured with respect to the channel bottom.

$$E = h + \alpha \frac{V^2}{2g} = h + \alpha \frac{Q^2}{2gA^2}$$

Specific Energy Curve: The variation of sp. energy with the depth of flow for a given section and constant discharge is called sp. energy curve or E.h. Curve.



→ E-h Curve always in the right side of 45° line.

→ E-h curve moves towards right with increase in discharge because $E = h + \alpha \frac{Q^2}{2gA^2}$.

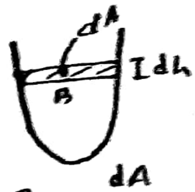
→ For all point on E-h curve except c, there are two values of h for a particular E.

→ At point c, Energy is minimum, flow is critical.

* prove that, at critical state of flow, the energy is minimum. Also prove that, it's value is $1.5h_c$ for wide rectangular channel.

proof: We know,

$$E = h + \alpha \frac{V^2}{2g} = h + \alpha \frac{Q^2}{2gA^2}$$



$$B = \frac{dA}{dh}$$

$$\therefore \frac{dE}{dh} = 1 + \frac{Q^2}{2g} \cdot \frac{-2}{A^3} \cdot \frac{dA}{dh}$$

$$= 1 - \frac{Q^2}{gA^2 . A} B$$

$$= 1 - \frac{Q^2}{gA^2 . A/B}$$

$$= 1 - \frac{v^2}{gD} \quad [A/B = D]$$

$$= 1 - F_r^2$$

For min^m energy

$$\frac{dE}{dh} = 0 \quad \therefore \; 1 - F_r^2 = 0 \; , \Rightarrow F_r = 1$$

$\therefore$ At critical state of flow, energy is min^m (proved)

Again, $E = h + \frac{v^2}{2g}$

For critical,

$$F_r = 1$$

$$\Rightarrow \frac{V_c}{\sqrt{gD_c}} = 1 \Rightarrow \frac{V_c^2}{g} = D_c \Rightarrow \frac{V_c^2}{2g} = \frac{D_c}{2}$$

$$\Rightarrow \frac{V_c^2}{2g} = \frac{h_c}{2}$$

For rectangular channel,

$$D_c = \frac{A_c}{B_c} = \frac{B_c h_c}{B_c} = h_c$$

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$\therefore E_c = h_c + \frac{V_c^2}{2g}$

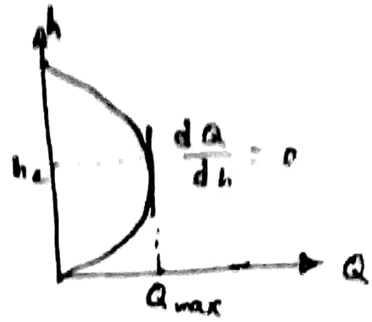
$= h_c + \frac{h_c}{2} = 1.5 h_c$ (proved)

Discharge - Depth Curve:

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

$= h + \frac{Q^2}{2gA^2}$

$\Rightarrow Q^2 = 2gA^2(E-h)$



* prove that, Maximum discharge is obtained at critical state of flow.

proof: $E = h + \frac{V^2}{2g}$

$\Rightarrow E = h + \frac{Q^2}{2gA^2} \Rightarrow Q^2 = 2gA^2(E-h) \Rightarrow E-h = \frac{V^2}{2g}$

Differentiate with respect to h

$$2Q\frac{dQ}{dh} = 2g\left[2A\frac{dA}{dh}(E-h) + A^2(-1)\right]$$

$$= \cancel{4}^{2}gAB \cdot \frac{V^2}{\cancel{2g}} - 2gA^2$$

$$= -2gA^2\left(1 - \frac{V^2}{gA/B}\right)$$

$$= -2gA^2\left(1 - \frac{V^2}{gD}\right)$$

$$= -2gA^2(1 - F_r^2)$$

For max^m discharge, $\frac{dQ}{dh} = 0$

$1 - F_r^2 = 0$

$\Rightarrow F_r = 1$

(proved)

Computation of critical depth:

① wide Channel:

$$h_c = \sqrt[3]{\frac{\alpha q^2}{g}}$$

② Rectangular Channel:

$$h_c = \sqrt[3]{\frac{\alpha Q^2}{g b^2}}$$

③ Triangular Channel:

$$h_c = \sqrt[5]{\frac{2\alpha Q^2}{g s^2}}$$

④ parabolic channel:

$$h_c = \sqrt[4]{\frac{27 \alpha c Q^2}{32 g}}$$

* Compute critical depth and velocity in a (i) wide rectangular channel with $q = 3\ m^2/s$ (ii) rectangular channel with $b = 6m$ and $Q = 20\ m^3/s$ (iii) triangular channel with $s = 2$ and $Q = 10\ m^3/s$ and (iv) parabolic channel whose profile is given by $y^2 = 4z$ with $Q = 20\ m^3/s$, $\alpha = 1.12$

Soln:

(i) wide channel:

$$h_c = \sqrt[3]{\frac{\alpha q^2}{g}} = \sqrt[3]{\frac{1.12 \times 3^2}{9.81}} = 1.01m$$

$$V_c = \frac{q}{h_c} = \frac{3}{1.01} = 2.97\ m/s$$

(ii) rectangular channel:

$$h_c = \sqrt[3]{\frac{\alpha Q^2}{g b^2}} = \sqrt[3]{\frac{1.12 \times 20^2}{9.81 \times 6^2}} = 1.08m$$

$$A_c = b h_c = 6 \times 1.08 = 6.50\ m^2$$

$$V_c = \frac{Q}{A_c} = \frac{25}{6.52} = 3.68\ m/s$$

(iii) Triangular Channel:

$$h_c = \sqrt[5]{\frac{2\alpha Q^2}{g s^2}} = \sqrt[5]{\frac{2\times 1.12\times 10^2}{9.81\times 2^2}} = 1.42\,m$$

$$A_c = s h_c^2 = 2\times 1.42^2 = 4.01\ m^2$$

$$V_c = \frac{Q}{A_c} = \frac{10}{4.01} = 2.49\ m/s$$

(iv) $y^2 = 4z \Rightarrow z = 0.25y^2 \quad \therefore c = 0.25,\ Q = 20\ m^3/s$

$$h_c = \sqrt[4]{\frac{27\alpha c Q^2}{32g}} = \sqrt[4]{\frac{27\times 1.12\times .25\times 20^2}{32\times 9.81}} = 1.76\,m$$

$$A_c = \frac{4h_c^{3/2}}{3\sqrt{c}} = \frac{4\times 1.76^{3/2}}{3\times\sqrt{0.25}} = 6.24\ m^2 \quad \left[A = \tfrac{2}{3}bh,\ b = 2\sqrt{\tfrac{h}{c}}\right]$$

$$V_c = \frac{Q}{A_c} = 3.21\ m/s$$

Hydraulic exponent for critical flow computation:

$$M = \frac{h}{A}\left(3B - D\frac{dB}{dh}\right)$$

where, M = Hydraulic exponent.

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* Determine the numerical value of the hydraulic exponent for critical flow computation M for a rectangular channel.

Sol^n: For rectangular channel.

$$A = bh,\ B = b,\ D = A/B = h,\ \frac{dB}{dh} = 0$$

$$\therefore M = \frac{h}{A}\left(3B - D\frac{dB}{dh}\right)$$

$$= \frac{h}{bh}\left(3b - h \times 0\right) = 3 \quad \text{Ans:}$$

* Compute the hydraulic exponent for critical flow computation for a trapezoidal channel with b = 6.1m, s = 2 and h = 2m.

Sol^n: $A = h(b + sh) = 2(6.1 + 2\times 2) = 20.2\,m^2$

$$D = A/B = \frac{20.2}{b + 2sh} = \frac{20.2}{6.1 + 2\times 2\times 2} = 1.43\,m$$

$$\frac{dB}{dh} = \frac{d}{dh}(b + 2sh) = 2s = 2\times 2 = 4.$$

$$\therefore M = \frac{h}{A}\left(3B - D\frac{dB}{dh}\right) = \frac{2}{20.2}\left(3\times 14.1 - 1.43\times 4\right)$$

$$= 3.62 \quad \text{Ans.}$$

* Compute the hydraulic exponent for critical flow computation for a triangular channel.

Sol^n: $B = 2sh,\ A = sh^2,\ D = A/B = h/2,\ \frac{dB}{dh} = 2s$

$$\therefore M = \frac{h}{A}\left(3B - D\frac{dB}{dh}\right)$$

$$= \frac{h}{sh^2}\left(6sh - sh\right)$$

$$= 5 \quad \text{Ans.}$$

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* Water flows at a velocity of 1 m/s and a depth of 1.5 m in a rectangular channel of 3m wide. Compute

a) The height of smooth upward step in the channel bed to produce critical flow.

b) The change in depth and water level produced by

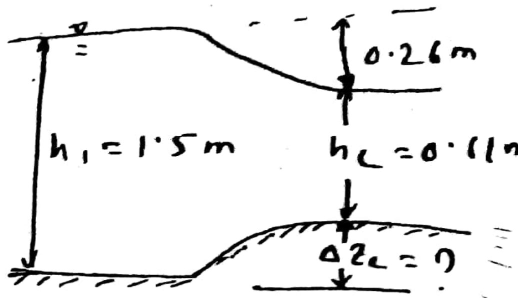
i) A smooth upward step of 0.45 m

ii) A " " " " 0.80 m

iii) A " downward " " 0.45 m

Assuming that, the discharge in the channel does not change. Neglect energy losses and take $\alpha = 1$.

Soln:



a) $v = 1\ m/s$, $h_1 = 1.5\ m$, $b = 3\ m$

$\therefore Q = av = 3 \times 1.5 \times 1 = 4.5\ m^3/s$

$$Fr = \frac{v}{\sqrt{gD}} = \frac{1}{\sqrt{9.81 \times 1.5}}$$

$= 0.26 < 1$

Hence the flow is subcritical.

$$h_c = \sqrt[3]{\frac{\alpha Q^2}{g b^2}} = \sqrt[3]{\frac{1 \times 4.5^2}{9.81 \times 3^2}} = 0.61\ m$$

Here,

$$E_1 = h_1 + v_1^2/2g$$

$$= 1.5 + \frac{1^2}{2 \times 9.81} = 1.55\ m$$

$E_c = 1.5 h_c = 1.5 \times .61 = 0.92\ m$

we get,

$$E_1 = E_c + \Delta z_c$$

$\Rightarrow \Delta z_c = 0.63\ m$

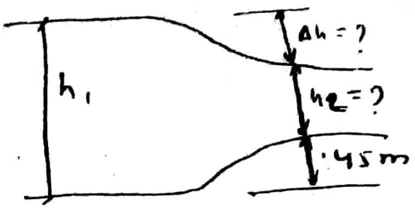
Ans.

Change in water level $= h_1 - h_c - \Delta z_c$

$= 0.26\ m$

b) (i) $E_1 = 1.55\,m$, $E_c = 0.92\,m$

$\Delta z = 0.45\,m$



By energy equation

$$h_1 + \frac{v_1^2}{2g} = h_2 + \frac{v_2^2}{2g} + \Delta z$$

Note: $E \geq E_c \rightarrow$ Flow occur

$E < E_c \rightarrow$ Flow doesn't occur

$$\Rightarrow E_1 = E_2 + \Delta z$$

$$\Rightarrow E_2 = 1.1 > E_c$$

$$\therefore h_2 + \frac{v_2^2}{2g} = 1.1$$

$$V_2 = \frac{Q}{b h_2} = \frac{4.5}{3h_2} = \frac{1.5}{h_2}$$

$$\Rightarrow h_2 + \frac{2.25}{2g h_2^2} = 1.1$$

$$\Rightarrow h_2 + \frac{0.115}{h_2^2} = 1.1$$

$$\Rightarrow h_2^3 - 1.1h_2^2 + 0.115 = 0$$

$$\therefore h_2 = -.29,\ 0.98,\ 0.4175$$

flow super-critical হবে, because, $F_r = 1.84$

since, the flow is subcritical, therefore the depth is 0.98 m.

$$\therefore \Delta h = h_1 - h_2 - \Delta z$$

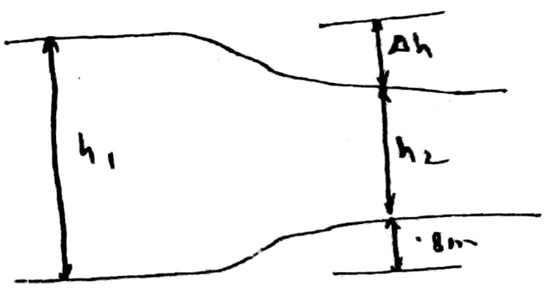
$$= 0.07\,m \quad \text{Ans.}$$

(ii)

$$E_1 = E_2 + \Delta z$$

$$\Rightarrow E_2 = 1.55 - 0.8$$

$$= 0.75 < E_c$$



i.e. Flow will not occur.

Therefore, the upstream flow will be affected. Assume, h_1' is the new depth at upstream. The flow will occur for minimum sp. energy at downstream.

Hence, $E_2 = E_c$

$$h_1' + \frac{v_1'^2}{2g} = E_c + \Delta z$$

$$V_1' = \frac{Q}{b h_1'} = 1.5/h_1'$$

$$\Rightarrow h_1' + \frac{2.25}{2g h_1'^2} = 0.92 + 0.8$$

$\Rightarrow h_1' + \frac{0.115}{h_1'^2} = 1.72$

$\Rightarrow h_1'^3 - 1.72\, h_1'^2 + 0.115 = 0$

$\Rightarrow h_1' = -0.24,\ 0.28,\ 1.18$

since the flow is subcritical, $h_1' = 1.68$ m

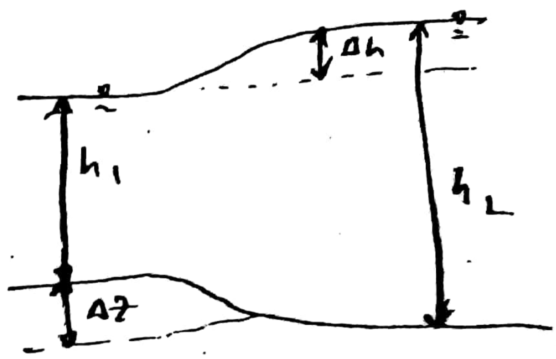
$\therefore$ Drop in water level $= h_1' - h_c - \Delta z$

$= 1.68 - 0.61 - 0.80$

$= 0.27$ m Ans.

(iii)

$h_1 + \frac{v_1^2}{2g} + \Delta z = h_2 + \frac{v_2^2}{2g}$



$\Rightarrow E_1 + \Delta z = E_2$

$\Rightarrow E_2 = 2.00$ m

$\Rightarrow h_2 + \frac{v_2^2}{2g} = 2.00$

$V_2 = \frac{Q}{bh_2} = \frac{4.5}{3h_2} = \frac{1.5}{h_2}$

$\Rightarrow h_2 + \frac{2.25}{2g h_2^2} = 2$

$\Rightarrow h_2 + \frac{.115}{h_2^2} = 2$

$\Rightarrow h_2^3 - 2h_2^2 + .115 = 0$

$\therefore h_2 = 1.97,\ .25,\ -.23$

$\therefore$ subcritical depth $= 1.97$ m

$\Delta h = h_2 - h_1 - \Delta z$

$= 0.02$ m

Ans.

Note: $b < b_c \rightarrow$ Flow is not possible

$b \geq b_c \rightarrow$ Flow occur.

* Water flows at a velocity of 1 m/s and a depth of 1.5 m in a long rectangular channel 3 m wide. compute -

a) the contraction in width of the channel for producing critical flow.

b) the depth and change in water level produced by -

i) a smooth contraction in width to 2m.

ii) a " " " " to 1m.

iii) a " expansion " " to 4m

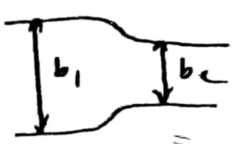
Assume that, discharge is constant. Neglect energy loss and take $\alpha = 1$.

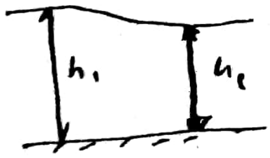
Soln:

(a) $h_1 = 1.5\,m$, $V = 1\,m/s$

$b_1 = 3m$.

$Q = AV = 3 \times 1.5 \times 1 = 4.5\ m^3/s$





By energy equation,

$$E_1 = h_1 + \frac{V_1^2}{2g} = 1.5 + \frac{1^2}{2 \times 9.8} = 1.55$$

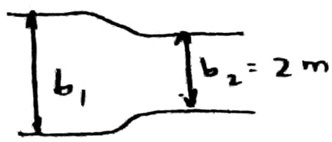
$$E_1 = E_c = 1.5 h_c \Rightarrow h_c = \frac{1.55}{1.5} = 1.03\ m$$

$$\therefore h_c = \sqrt[3]{\frac{\alpha Q^2}{g b_c^2}} \Rightarrow b_c = 1.37\ m \quad \text{Ans,}$$

(b) i)

Drop in water level $= h_1 - h_c = 0.47\,m$ Ans.

$E_1 = E_2$



$$\Rightarrow 1.55 = h_2 + \frac{V_2^2}{2g}$$

$$V_2 = \frac{Q}{b_2 h_2} = \frac{4.5}{2 h_2} = \frac{2.25}{h_2}$$

$$\Rightarrow 1.55 = h_2 + \frac{5.06}{2g\, h_2^2}$$

$$\Rightarrow 1.55 = h_2 + \frac{.258}{h_2^2}$$

$$\Rightarrow h_2^3 - 1.55 h_2^2 + .258 = 0$$

$\therefore h_2 = 1.42,\ .49 \quad \therefore h_2 = 1.42m$.

Drop of water level $= 1.5 - 1.42 = 0.08\ m$ Ans.

(ii) $b = 1m < b_c$; so, flow is not possible. To continue the flow $b = b_c = 1.37m$.

$$h_1 + \frac{v_1^2}{2g} = h_2 + \frac{v_2^2}{2g}$$

$$V_2 = \frac{Q}{b_c h_2} = \frac{4.5}{1.37 \times h_2} = \frac{3.28}{h_2}$$

$$\Rightarrow E_1 = h_2 + \frac{10.76}{2g h_2^2}$$

$$\Rightarrow 1.55 = h_2 + \frac{.55}{h_2^2}$$

$$\Rightarrow h_2^3 - 1.55 h_2^2 + .55 = 0$$

$$\Rightarrow h_2 = 1.07, 1 \quad \therefore h_2 = 1.07m.$$

Drop in water level $= 1.5 - 1.07 = .43$ m

Ans.

(iii)
$$E_1 = h_2 + \frac{v_2^2}{2g} \qquad V_2 = \frac{Q}{b h_2} = \frac{4.5}{4 h_2} = \frac{1.125}{h_2}$$

$$\Rightarrow 1.55 = h_2 + \frac{1.265}{2g h_2^2}$$

$$\Rightarrow 1.55 = h_2 + \frac{0.0645}{h_2^2}$$

$$\Rightarrow h_2^3 - 1.55 h_2^2 + 0.0645 = 0$$

$$\Rightarrow h_2 = 1.52, 0.22$$

so, $h_2 = 1.52$ m

Rise in water level $= 1.52 - 1.5$

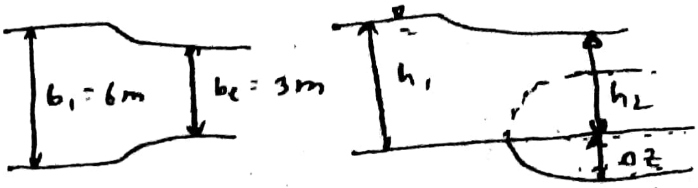
$= 0.02$ m

Ans

* Water flows in a 6m wide rectangular channel at a depth of 2m and a velocity of 2 m/s. The channel is contracted to a width of 3m. How much the channel bottom is to be simultaneously raised or lowered for the flow to be possible as specified? Neglect energy losses, take $\alpha = 1$.

Soln:

$Q = AV = 6 \times 2 \times 2 = 24 \ m^3/s$



$$Fr = \frac{V}{\sqrt{gh}} = \frac{2}{\sqrt{9.81 \times 2}}$$

$$= 0.45 < 1.$$

Upstream flow is subcritical.

$$E_1 = h_1 + \frac{v_1^2}{2g} = 2 + \frac{2^2}{2 \times 9.81} = 2.2 \ m.$$

For flow to be occur: minimum energy = E_c

$$h_c = \sqrt[3]{\frac{\alpha Q^2}{g b_c^2}} = \sqrt[3]{\frac{1 \times 24^2}{9.81 \times 3^2}} = 1.87 \ m$$

$$E_c = 1.5 h_c = 2.80 \ m$$

since $E_c > E_1$

$$\therefore E_1 + \Delta z = E_c$$

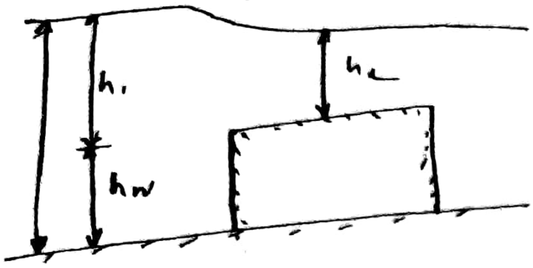
$$\Rightarrow \Delta z = 2.8 - 2.2$$

$$= 0.6 \ m$$

Therefore, the bed is needed to be lowered by 0.6 m.

Ans.

Flow measurement for Broad-crested weir:



$$h_c = \sqrt[3]{\frac{\alpha Q^2}{g b^2}}$$

$$h_1 = h_c + \frac{V_c^2}{2g}$$

$$= h_c + \frac{h_c}{2}$$

$$= \frac{3}{2} h_c$$

$$Q = 1.705\, b\, h_1^{1.5}$$

* A broad crested weir is built in a rectangular channel of width 2m. The height of the weir crest above the channel bed is 1.20m and the head over the weir is 0.80m. Calculate discharge

Soln: $h_w = 1.2m$, $h_1 = 0.8m$. $b = 2m$.

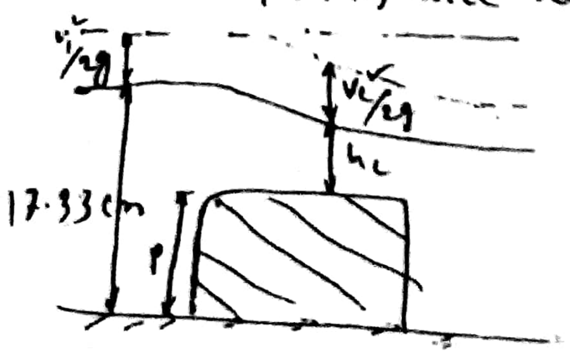
$$Q = 1.705\, b\, h_1^{1.5}$$

$$= 1.705 \times 2 \times (0.8)^{1.5}$$

$$= 2.44\ m^3/s$$

Ans.

* Determine the height of broad crested weir (P) from given data. width of flume = 25cm. actual discharge, = 7020.5 cm³/s. Ignore head loss.



Jandra-16

Sol^n: $h_c = \sqrt[3]{\frac{\alpha Q^2}{g b^2}} = \sqrt[3]{\frac{1 \times 7070.5^2}{981 \times 25^2}} = 4.31\,cm.$

$h_1 = 1.5 h_c = 6.47\,cm$

$\therefore\ P = 17.33 - h_1$

$= 17.33 - 6.47 = 10.86\,cm$ Ans.

* A trapezoidal channel has a bottom width of 6 m & side slope 1:1. The depth of flow is 1.5 m at a discharge of 15 m³/s. Determine specific ~~gravity~~ Energy in terms of head of water.

Sol^n:

$Q = AV$

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

$= \frac{15}{11.25}$

$= 1.33\,m/sec.$

$A = 6h + 5h^2$

$= 6 \times 1.5 + 1 \times 1.5^2$

$= 11.25\,m^2$

$\therefore\ E = h + \frac{v^2}{2g}$

$= 1.5 + \frac{1.33^2}{2 \times 9.81} = 1.59\,m$

Ans.

Meghna 17.
Great

* Calculate the sp. energy of the trapezoidal canal 2h:2v. and Q = 2700 l/s, b = 6m and depth = 1.5m

Sol^n:

sp. energy $= h + \frac{v^2}{2g}$

$= 1.5 + \frac{.2^2}{2 \times 9.81}$

$= 1.502\,m$

$Q = 2.7\,m^3/s$

$A = 6h + 5h^2$

$= 6 \times 1.5 + 2 \times 1.5^2$

$= 13.5\,m^2$

$V = Q/A = \frac{2.7}{13.5} = 0.2\,m/s$

Great

* Calculate the total head for $z_1 = 7m$, $v = 25 m/s$, $P = 25$ [illegible] neglect friction.

<u>soln</u>:

$$\text{Total head} = z_1 + \frac{P}{\gamma} + \frac{v^2}{2g}$$

$$= 7 + \frac{25}{9.81} + \frac{25^2}{2 \times 9.81} = 41.40\,m$$

<u>Ans</u>

"Uniform Flow"

Cross-Sectional Mean velocity:

1. Hydraulically smooth surface, $\left(\frac{k_s u^*}{\nu} \leq 5\right)$

$$\frac{U}{u^*} = 5.75 \log\left(\frac{3.64\, u^* R}{\nu}\right)$$

2. Hydraulically rough surface $\left(\frac{k_s u^*}{\nu} \geq 70\right)$

$$\frac{U}{u^*} = 5.75 \log\left(\frac{12.2R}{k_s}\right)$$

3. Transition regime $\left(5 < \frac{k_s u^*}{\nu} < 70\right)$

$$\frac{U}{u^*} = 5.75 \log\left(\frac{12.2R}{k_s + 3.35\frac{\nu}{u^*}}\right)$$

where,

u^* = shear/friction/drag velocity

$= \sqrt{gRS_0}$ → for any channel.

$= \sqrt{gyS_0}$ → for wide channel.

R = Hydraulic Radius $= A/P$

K_s = roughness height.

$\nu = 10^{-6}$ m²/s

* A rectangular channel is 6 m wide and laid on a slope of 0.25%. The channel is made of concrete (k_s = 2mm) and carries water at a depth of 0.50m. Compute the mean velocity of flow.

Soln: $k_s = 2\,mm = 0.002\,m$, $S_0 = \cdot25/100 = 0.0025$

$$R = A/P = \frac{6\times\cdot5}{6+2\times\cdot5} = 0.4286\,m.$$

$$u^* = \sqrt{gRS_0} = \sqrt{9.81\times\cdot4286\times\cdot0025} = 0.1025\,m/s.$$

$$\frac{k_s u^*}{\nu} = \frac{\cdot002\times\cdot1025}{10^{-6}} = 205 > 70.$$

∴ The boundary is hydraulically rough.

$$\frac{U}{u^*} = 5.75\log\frac{12.2R}{k_s}$$

$$\Rightarrow U = 5.75\log\frac{12.2\times\cdot4286}{\cdot002}\times0.1025$$

$$= 2.074\,m/s$$

Ans.

Uniform Flow formula:

① **Chezy Formula:** $V = C\sqrt{RS}$

where, C = Chezy's constant.

$= 50\ m^{1/2}/s$

$C = \sqrt{\frac{8g}{f}}$; $C = \frac{1}{n} R^{1/6}$.

② **Darcy – Weisbach Formula:**

For pipe flow,

$$h_f = \frac{fLV^2}{2gd}$$

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$$\Rightarrow V = \sqrt{\frac{2gd\,h_f}{fL}}$$

$$\Rightarrow V = \sqrt{\frac{8g}{f}}\ R^{1/2} S^{1/2}$$

$d = 4R$

$S = \frac{h_f}{L}$

where,
- h_f = frictional loss
- f = frictional factor
- d = diameter of pipe
- L = length of pipe
- V = mean velocity of flow.
- g = Acceleration due to gravity.

③ **Manning Formula:**

$$V = \frac{\phi}{n} R^{2/3} S^{1/2} \quad ; \quad n = 0.047\, d_{50}^{1/6}$$

where, ϕ = conversion factor

= 1 for SI unit

= 1.486 for FPS unit.

Note: যদি width, depth ft এ থাকে ϕ এর মান ব্যবহার্য হবে।

* Factors affecting manning's n:

1. Roughness of the surface
2. vegetation
3. channel irregularity.
4. Channel alignment.
5. silting & scouring.
6. obstruction
7. stage & discharge
8. Suspended materials & bed Load.

* An open channel lined with concrete ($d_{50} = 1.5$ mm) is laid on a slope of 0.1%. The channel is trapezoidal with $b = 6$m, $s = 2$. compute the uniform flow in the channel if the depth of flow is 2m. Also compute the numerical values of chezy's C & friction factor f.

Sol^n: $S = \frac{.1}{100} = 0.001$,

$\phi = 1$

$A = (b + sh)h = (6 + 2\times2)\times2$
$= 20m^2$

$P = b + 2\sqrt{1+s^2}\,h$
$= 6 + 2\sqrt{1+2^2} \times 2$
$= 14.94m$

$R = A/P = 1.34m$

$d_{50} = 1.5mm$
$= .0015m$

$n = 0.047\, d_{50}^{1/6}$
$= 0.016$

$v = \frac{\phi}{n} R^{2/3} S^{1/2}$
$= \frac{1}{0.016} \times 1.34^{2/3} \times .001^{1/2}$
$= 3.066$ m/s.

$\therefore Q = Av$
$= 20 \times 3.066$
$= 61.31$ m^3/s

Ans.

$C = \frac{1}{n} R^{1/6}$
$= \frac{1}{.016} \times 1.34^{1/6}$
$= 65.62$ m$^{1/2}$/s

Ans.

$C = \sqrt{\frac{8g}{f}}$

$\Rightarrow f = \frac{8g}{C^2} = \frac{8\times9.81}{65.62^2} = 0.0182$

Ans

Computation of normal depth:

① For wide channel:

By manning formula,

$$h_n = \left(\frac{nq}{\sqrt{S_0}}\right)^{3/5}$$

By chezy's formula,

$$h_n = \left(\frac{q}{C\sqrt{S_0}}\right)^{2/3}$$

② For other channel:

By trail method

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

$$\Rightarrow A_n R_n^{2/3} = \frac{Qn}{\sqrt{S}}$$

n, Q, S is given from which normal depth need to be calculate.

Here, $A_n R_n^{2/3}$ is known as section factor.

* A wide channel with $S_0 = 0.0025$ carries a discharge of $3 \, m^2/s$. Compute the normal depth and velocity (i) using the manning formula when $n = 0.020$, and (ii) using the chezy formula when $C = 45 \, m^{1/2}/s$.

soln: ① $h_n = \left(\frac{nq}{\sqrt{S_0}}\right)^{3/5} = \left(\frac{0.020 \times 3}{\sqrt{0.0025}}\right)^{3/5} = 1.12 m.$

$$\therefore V = \frac{q}{h_n} = \frac{3}{1.12} = 2.69 \, m/s \text{ Ans,}$$

(ii) $h_n = \left(\frac{q}{c\sqrt{S_0}}\right)^{2/3} = \left(\frac{3}{45\sqrt{.0025}}\right)^{2/3} = 1.21\ m$

$V = \frac{q}{h_n} = \frac{3}{1.21} = 2.48\ m/s$ Ans.

* For a triangular channel with side slopes of 2 : 1, a longitudinal slope of 0.0016, n = 0.015, determine the normal depth if Q = 10 m^3/s.

Ans:

$A_n R_n^{2/3} = \frac{Qn}{\sqrt{S_0}}$

$\Rightarrow S h_n^2 \left(\frac{S h_n}{2\sqrt{1+S^2}}\right)^{2/3} = \frac{10 \times .015}{\sqrt{.0016}}$

$\Rightarrow 2 h_n^2 \left(\frac{2h_n}{2\sqrt{1+2^2}}\right)^{2/3} = 3.75$

$A_n = S h_n^2$

$P = 2\sqrt{1+S^2}\ h_n$

$R_n = \frac{A_n}{P} = \frac{S h_n}{2\sqrt{1+S^2}}$

$S = 2$

$\therefore h_n = 1.55\ m$

Ans.

* For a trapezoidal channel with b = 6 m, S = 2, n = 0.025, S_0 = 0.001, compute the normal depth and velocity if Q = 14 m^3/s

Soln:

$A_n R_n^{2/3} = \frac{Qn}{\sqrt{S_0}} = \frac{14 \times .025}{\sqrt{.001}} = 11.068$

For trapezoidal channel.

$A_n = (b + S h_n) h_n = (6 + 2h_n) h_n$

$P = b + 2\sqrt{1+S^2}\ h_n = 6 + 2\sqrt{5}\ h_n$

$R_n = \frac{A_n}{P} = \frac{(6+2h_n)h_n}{6+2\sqrt{5}\ h_n}$

$\therefore (6+2h_n) h_n \left\{\frac{(6+2h_n)h_n}{6+2\sqrt{5}\ h_n}\right\}^{2/3} = 11.068$

$\Rightarrow h_n = 1.32\ m$

so, $A_n = (6+2\times1.32)\times1.32$
$= 11.405\ m^2$

$\therefore V = \frac{Q}{A} = \frac{14}{11.405} = 1.23\ m/s$ Ans.

* compute the normal depth and velocity in a parabolic channel with $Q = 20\ m^3/s$, $n = 0.025$, $S_0 = 0.0025$ when the profile of the channel is given by $y^2 = 4z$.

Soln:

$$AR^{2/3} = \frac{Qn}{\sqrt{S_0}} = \frac{20\times .025}{\sqrt{.0025}} = 10$$

For parabolic channel.

$$A = \frac{2}{3}Bh = \frac{8}{3}h^{3/2}$$

$$B = 2\sqrt{\frac{h}{c}} = 4\sqrt{h}$$

$y^2 = 4z$
$\Rightarrow z = \frac{1}{4}y^2$
$\therefore c = \frac{1}{4} = 0.25$

$$P = B + \frac{8h^2}{3B} = 4\sqrt{h} + \frac{8h^2}{12\sqrt{h}}$$

$$\therefore \frac{8}{3}h^{3/2} \cdot \left(\frac{\frac{8}{3}h^{3/2}}{4\sqrt{h} + \frac{8h^2}{12\sqrt{h}}}\right)^{2/3} = 10$$

$\Rightarrow h_n = 2.304\ m.$

$\therefore A = \frac{8}{3}(2.304)^{3/2} = 9.33\ m^2$

$\therefore V = \frac{Q}{A} = 2.14\ m/s$ Ans.

$$N = \frac{2h}{3A}\left(5B - 2R\frac{dP}{dh}\right)$$

* compute the hydraulic exponent for uniform flow computation N of a trapezoidal channel with b = 6.1m, s = 2, h = 2m based on the manning formula.

Soln:

$A = (b+sh)h = (6.1+2\times2)\times2 = 20.2\ m^2$

$B = b + 2sh = 6.1 + 2\times2\times2 = 14.1m$

$P = b + 2\sqrt{1+s^2}\,h = 6.1 + 2\sqrt{5}\times2 = 15.044m$

$R = A/P = 1.342m.$

$\frac{dP}{dh} = 2\sqrt{5}$

$\therefore N = \frac{2h}{3A}\left(5B - 2R\frac{dP}{dh}\right)$

$= \frac{2\times2}{3\times20.2}\left(5\times14.1 - 2\times1.342\times2\sqrt{5}\right) = 3.86$

Ans.

Computation of normal & critical slope:

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

$$\Rightarrow S_n = \frac{n^2 Q_n^2}{A_n^2 R_n^{4/3}} \rightarrow [A, P, B, \text{ need to calculate with } h_n]$$

critical slope,

$$S_c = \frac{n^2 Q^2}{A^2 R^{4/3}} \rightarrow [A, P, B \text{ need to calculate with } h_c]$$

* A rectangular channel has a bottom width of 6 m, $\alpha = 1.12$ and $n = 0.02$. (i) For $h_n = 1$m and $Q = 11\ m^3/s$. determine the normal slope. (ii) Determine the critical slope for $Q = 11 m^3/s$. (iii) Determine the critical slope for $h_n = 1$m.

sol^n: (i) $h_n = 1m$, $Q = 11\ m^3/s$

$A = bh = 6 \times 1 = 6m^2$, $P = b + 2h = 6 + 2 \times 1 = 8m$

$R = A/P = 0.75m$.

$$S_n = \frac{n^2 Q_n^2}{A_n^2 R_n^{4/3}} = \frac{.02^2 * 11^2}{6^2 * (.75)^{4/3}} = 0.002$$

Ans.

(ii) $Q = 11 m^3/s$

$$h_c = \sqrt[3]{\frac{\alpha Q^2}{g b^2}} = \sqrt[3]{\frac{1.12 \times 11^2}{9.81 * 6^2}} = 0.73m.$$

$A = bh_c = 6 \times .73 = 4.36m^2$, $P = b + 2h_c = 6 + 2 \times .73$
$= 7.45m$

$R = A/P = 0.58m$.

$$S_c = \frac{n^2 Q^2}{A^2 R^{4/3}} = \frac{.02 \times 11^2}{4.36^2 \times (.58)^{4/3}} = 0.0053$$

Ans.

(iii) $h_n = 1m = h_c$

$A = bh = 6\times1 = 6\ m^2$, $P = b+2h = 6+2\times1 = 8m$

$R = A/P = 0.75m$

$$V_n = V_c = \sqrt{\frac{gD_c}{\alpha}} = \sqrt{\frac{gh_c}{\alpha}} = \sqrt{\frac{9.81\times1}{1.12}} = 2.96\ m/s$$

$\therefore Q = AV_0 = 6\times2.96 = 17.76\ m^3/s$

$$S_c = \frac{n^2Q^2}{A^2R^{4/3}} = \frac{.02^2\times17.76^2}{6^2\times(.75)^{4/3}} = 0.0051$$

Ans.

Channel section with composite roughness:

$$n = \left(\frac{P_1 n_1^{3/2} + P_2 n_2^{3/2} + P_3 n_3^{3/2}}{P}\right)^{2/3}$$

where, P_1 = bottom perimeter

P_2, P_3 = side perimeter.

* The sides of a laboratory flume are made of glass ($n = 0.010$) and the bottom is made of wood ($n = 0.015$). The flume is rectangular with $b = 1m$ and is laid on a slope of 0.001. Compute the discharge in the flume if $h_n = .4m$.

Soln: $P_1 = 1m$, $P_2 = P_3 = 0.4m$, $P = P_1+P_2+P_3 = 1.8m$

$n_1 = 0.015$, $n_2 = n_3 = 0.010$, $A = bh = 1\times0.4 = 0.4\ m^2$

$$R = A/P = \frac{0.4}{1.8} = 0.222m.$$

$$n = \left(\frac{P_1 n_1^{3/2} + P_2 n_2^{3/2} + P_3 n_3^{3/2}}{P}\right)^{2/3}$$

$= 0.013$

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

$= 0.4 \times \frac{1}{0.013} * (0.222)^{2/3} * 0.001^{1/2} = 0.36\ m^3/s$

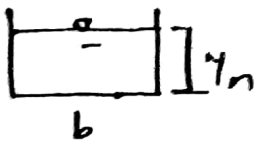
Ans.

* A wide channel having $n = 0.026$, $S_0 = 0.0028$. calculate the normal depth & velocity of flow if the channel discharge is $30\ m^3/s$.

soln: we know,

$Q = 30 m^3/s$

$Q = AV$



For wide channel $R = y_n$

$\Rightarrow Q_b = \frac{1}{n} b y_n s^{1/2} y_n^{2/3}$

BPDB-16

$\Rightarrow 30 = \frac{1}{n} s^{1/2} y_n^{5/3}$

$\Rightarrow 30 = \frac{1}{.026} \times .0028^{1/2} \times y_n^{5/3}$

$\Rightarrow y_n = 5.02\ m$

$\therefore V = \frac{Q}{A} = \frac{q_b}{b y_n} = \frac{30}{5.02} = 5.97\ m/s$

Ans.

*** Best Hydraulic Section:** A channel section that conveys the maximum discharge for a given area is known as the best hydraulic section.

BWDB-16

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

$$= \frac{\phi}{n} A \left(\frac{A}{P}\right)^{2/3} S^{1/2}$$

$$= \frac{\phi}{n} \frac{A^{5/3}}{P^{2/3}} S^{1/2}$$

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

where, K = Conveyance $= \frac{\phi}{n} \frac{A^{5/3}}{P^{2/3}}$

Q is maximum when K is maximum, P is minimum.

* Show that, the best hydraulic rectangular section is one-half of a square.

Soln: For rectangle, $A = bh \Rightarrow b = Ah^{-1}$

$$\therefore P = b + 2h = Ah^{-1} + 2h$$

For minm value of P, $\frac{dP}{dh} = 0$

$$\frac{d}{dh}\left(Ah^{-1} + 2h\right) = 0$$

$$\Rightarrow -Ah^{-2} + 2 = 0$$

$$\Rightarrow Ah^{-2} = 2$$

$$\Rightarrow bh \cdot h^{-2} = 2 \Rightarrow b = 2h.$$

so, the best hydraulic rectangular section is one-half of a square.

Best hydraulic section of a trapezoidal channel is one-half of a regular hexagon.

Best hydraulic section of triangular channel is found when side slope is 1:1.

Design of channel by best hydraulic section:

A trapezoidal channel carrying $20 m^3/s$ is built with non-erodible bed having a slope of 1 in 1000, $n = 0.025$. Design the channel by the concept of best hydraulic section. Side slope 1:1.

Soln:

$$AR^{2/3} = \frac{Qn}{\sqrt{S}} = \frac{20 \times .025}{\sqrt{.001}} = 15.81$$

For trapezoidal, $A = \sqrt{3}h^2$, $R = 0.5h$ [Table]

$$\therefore AR^{2/3} = \sqrt{3}h^2 (.5h)^{2/3} = 15.81$$

$$\Rightarrow h = 2.73 \text{ Ans.}$$

Check: $\therefore A = \sqrt{3}h^2 = 12.86 \; m^2$; $(b+3h)h = 12.86 \Rightarrow b = 1.98 m$ Ans.

permissible velocity, $V = Q/A = \frac{20}{12.86} = 1.56 m/s$

Froude No, $F_r = \frac{V}{\sqrt{gD}} = \frac{1.56}{\sqrt{9.81 \times 2.73}}$

$$= 0.31 < 1$$

So, subcritical flow.

Add a free board of 0.77m.

Total depth $= 2.73 + .77$

$= 3.5$ m

Ans.

Design of non-erodible / lined channel:

* A trapezoidal channel lined with concrete ($n = 0.013$) and laid on a slope of 1 in 3600 carries a discharge of $100 \, m^3/s$. Determine the section dimensions of the channel (a) taking $b = 6m$ and side slope 1:1.

Soln: $Q = 100 \, m^3/s$, $s = 1$, $S = 0.00028$, $b = 6m$, $n = .013$

For trapezoidal channel,

$$A = (b + sh)h = (6 + h)h$$

$$P = b + 2\sqrt{1+s^2}\, h = 6 + 2\sqrt{2}\, h$$

Again;

$$AR^{2/3} = \frac{Qn}{\sqrt{S}} = \frac{100 \times .013}{\sqrt{.00028}} = 78$$

$$\therefore (6+h)h \times \left\{ \frac{(6+h)h}{6+2\sqrt{2}h} \right\}^{2/3} = 78$$

$$\Rightarrow h = 4.24 \, m$$

Check for minimum permissible velocity:

$$V = \frac{Q}{A} = \frac{100}{(6+4.24)4.24} = 2.30 \, m/s \text{ ok}$$

Check for Froude Number:

$$F_r = \frac{V}{\sqrt{gD}} = \frac{2.30}{\sqrt{9.81 \times 7.24}} = 0.27 < 1$$

$$D = \frac{(6+h)h}{b+2sh} = 7.24$$

subcritical.

* prove that, the best hydraulic section of triangle is half of rectangular.

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proof: for triangle,

$$A = sh^2 \Rightarrow s = Ah^{-2}$$

$$P = 2\sqrt{1+s^2}\,h \Rightarrow P^2 = 4h^2(1+s^2) \Rightarrow P^2 = 4h^2(1+A^2h^{-4})$$

Now $\cancel{\frac{dP}{dh}}$ $\Rightarrow 2P\frac{dP}{dh} = \frac{d}{dh}(4h^2+4A^2h^{-2})$

$$\therefore \frac{dP}{dh} = 0$$

$$\therefore \cancel{\Rightarrow \frac{d}{dh}(4h^2+4s^2h^2)=0}$$

$$\cancel{\Rightarrow 8h+8s^2}$$

$$\Rightarrow 8h - 8A^2\frac{1}{h^3} = 0$$

$$\Rightarrow 8h - 8\times s^2h^4 \times \frac{1}{h^3} = 0$$

$$\Rightarrow 8h = 8s^2h$$

$$\Rightarrow s^2 = 1$$

$$\therefore s = 1$$

so, side slope is 1 : 1

best hydraulic section: Area of triangular channel, $A = sh^2 = h^2$

Area of rectangular channel, $A = bh$ $[b=2h]$ (best hydraulic section of rectangle)

$= 2h^2$

$$\therefore A_{triangular} = \frac{1}{2} A_{rectangular}$$

(proved)

"Hydraulic Jump"

 # Hydraulic jump: In a open channel, when a supercritical flow is made to change abruplly to subcritical flow, the result is also an abrupt rise of the water surface. This feature is known as hydraulic jump.

 # Application:

- to increase the discharge of sluice.
- to remove air pockets from water supply line.
- to dissipate energy in water flowing over dams, weirs and other hydraulic structures.
- to prevent scouring downstream from the structures

 # Types of jump:

1. $F_r = 1$; No Jump (Flow is critical)
2. $F_r = 1 \sim 1.7$; Undular Jump.
3. $F_r = 1.7 \sim 2.5$; Weak Jump.
4. $F_r = 2.5 \sim 4.5$; oscillating Jump.
5. $F_r = 4.5 \sim 9.0$; Steady Jump.
6. $F_r = > 9.0$; strong Jump.

 # Formula for Hydraulic Jump:

① Relation between Initial & sequent depth:

$$\frac{Y_2}{Y_1} = \frac{1}{2}\left(\sqrt{1+8F_1^2} - 1\right)$$

and,

$$\frac{Y_1}{Y_2} = \frac{1}{2}\left(\sqrt{1+8F_2^2} - 1\right)$$

$Y_1 \rightarrow$ Initial Depth

$Y_2 \rightarrow$ sequent depth.

$$F_1 = \frac{V_1}{\sqrt{gh_1}}$$

$$F_2 = \frac{V_2}{\sqrt{gh_2}}$$

② Length of the jump

$$\frac{L_j}{Y_1} = 9.75\,(F_1 - 1)^{1.01}$$ | $L_j \rightarrow$ Length of jump.

③ Energy loss in jump,

$$\Delta E = \frac{(Y_2 - Y_1)^3}{4 Y_1 Y_2}$$

④ Relative loss

$$\frac{\Delta E}{E_1} = 1 - \frac{E_2}{E_1}$$

⑤ Height of the jump

$$h_j = Y_2 - Y_1$$

⑥ Relative height of jump,

$$\frac{h_j}{E_1} = \frac{\sqrt{1+8F_1^2} - 3}{2 + F_1^2}$$

⑦ Efficiency of jump:

$$\frac{E_2}{E_1} = \frac{(1+8F_1^2)^{3/2} - 4F_1^2 + 1}{8F_1^2\,(2 + F_1^2)}$$

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Submerged Jump: A Hydraulic jump is formed when the initial & sequent depths satisfy the equation -

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1+8F_1^2} - 1\right)$$

If the downstream depth is increased, the jump moves U/s. Thus by increasing the downstream depth a jump can be moved U/s. Now, if there is an U/s control like a sluice gate, the jump cannot move U/s & a submerged jump is formed.

$$\frac{y_3}{y_4} = \left[1 + 2F_4^2\left(1 - \frac{y_4}{y_1}\right)\right]^{1/2}$$

where, y_1 = height of sluice gate opening.

y_4 = tailwater depth.

Length of Submerged Jump

$$\frac{L_j}{y_2} = 4.9S + 6.1$$

where, $S = \frac{y_4 - y_2}{y_2}$ called submergence factor.

* Water flows in a horizontal rectangular channel 6m wide at a depth of 0.52m and a velocity of 15.2 m/s. If a hydraulic jump forms in this channel, determine

1. The type of jump.
2. The downstream depth needed to form the jump.
3. The horse-power dissipation in the jump.
4. The efficiency of the jump.
5. The relative hight of the jump.
6. The length of the jump.

soln: $b = 6m$, $y_1 = 0.52m$, $v_1 = 15.2 m/s$

$Q = b y_1 v_1 = 6 \times .52 \times 15.2 = 47.42 m^3/s$.

① $F_1 = \frac{v_1}{\sqrt{g h_1}} = \frac{15.2}{\sqrt{9.81 * .52}} = 6.73$

so, the jump is steady jump.

② $\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1+8F_1^2} - 1\right) = \frac{1}{2}\left(\sqrt{1+8 * 6.73^2} - 1\right)$

$\Rightarrow y_2 = 9.03 * 0.52 = 4.70 m$

③ $AE = \frac{(y_2 - y_1)^3}{4 y_1 y_2} = \frac{(4.70 - .52)^3}{4 * 4.7 * .52} = 7.47 m - kg/kg$

$\therefore$ Horse power dissipation $= \frac{7.22 \rho Q AE}{550}$

$= \frac{7.22 \times 1000 \times 47.42 \times 7.47}{550}$

$= 4650.4$

④ $\frac{E_2}{E_1} = \frac{(1+8F_1^2)^{3/2} - 4F_1^2 + 1}{8F_1^2(2+F_1^2)} = 0.3937 = 39.37\%$.

⑤ $\frac{h_j}{E_1} = \frac{\sqrt{1+8F_1^2} - 3}{2+F_1^2} = 0.3396 = 33.96\%$.

⑥ $\frac{L_j}{y_1} = 9.75 (F_1 - 1)^{1.01}$

$\Rightarrow L_j = 9.75 \times .52 (6.73 - 1)^{1.01}$

$= 29.56 m$

Ans.

* A horizontal trapezoidal channel with $b = 6m$, $s = 2$ carries a discharge of $120\ m^3/s$. If the upstream depth of flow is 1m, compute the relative energy loss that will occur if there is a hydraulic jump in the channel. $h_2 = 5.14m$, $A_2 = 83.68m^2$

$h_2 = 5.14$, $A_2 = 83.68\ m^2$

sol^n: $h_1 = 1m$, $b = 6m$, $s = 2$, $Q = 120\ m^3/s$

$A_1 = (b + sh_1)h_1 = 8\ m^2$

$V_1 = \frac{Q}{A_1} = 15\ m/s$, $V_2 = \frac{Q}{A_2} = 1.434\ m/s$.

$E_1 = h_1 + \frac{V_1^2}{2g} = 12.47\ m$

$E_2 = h_2 + \frac{V_2^2}{2g} = 5.24\ m$

$\therefore$ Relative energy loss $= \frac{E_1 - E_2}{E_1} \times 100 = 57.98\%$.

Ans.

* The sequent & initial depth ratio of a hydraulic jump in a rectangular channel is 16.48. Find the froude No. at the beginning of jump & type of jump.

sol^n:

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1 + 8F_1^2} - 1\right)$$

Buet BSDB-14

$$\Rightarrow 16.48 = \frac{1}{2}\left(\sqrt{1 + 8F_1^2} - 1\right)$$

$$\Rightarrow F_1 = 12 > 9;\ \text{so, strong jump.}$$

* A rectangular channel 6 m wide and discharge 1200 l/sec of water into a 6m wide apron with zero slope with a mean velocity of 16 m/sec. What is the height of the jump?

Soln:

$Q = 1.2\ m^3/s$, $v = 16\ m/sec$, $b = 6\ m$

$A = \frac{Q}{v} = 0.075\ m^2$, $h_1 = \frac{A}{b} = 0.0125\ m = Y_1$

$F_1 = \frac{v}{\sqrt{gh_1}} = 45.69 > 9 \rightarrow$ Strong Jump

$\frac{Y_2}{Y_1} = \frac{1}{2}\left(\sqrt{1+8F_1^2} - 1\right) = \frac{1}{2}\left(\sqrt{1+8*45.69^2} - 1\right)$

$\Rightarrow Y_2 = 64.12 * 0.0125 = 0.801\ m$

$\therefore$ Height of jump $= Y_2 - Y_1$

$= 0.801 - 0.0125$

$= 0.78897\ m$ **Ans**

"Gradually Varied Flow"

* classification of flow profile:

channel category	symbol	characteristics condition	Remarks
Mild	M	$Y_n > Y_c$; $S_o < S_c$	Subcritical flow
critical	C	$Y_n = Y_c$; $S_o = S_c$	critical flow
steep	S	$Y_n < Y_c$; $S_o > S_c$	super critical flow
Horizontal	H	$S_o = 0$	can't sustain uniform flow.
Adverse	A	$S_o < 0$	can't sustain uniform flow.

Zone - 1 : $Y > Y_n > Y_c$; $Y > Y_c > Y_n$

Zone - 2 : $Y_n > Y > Y_c$ for mild.

$Y_c > Y > Y_n$ for steep.

Zone - 3 : $Y_c > Y_n > Y$

* prove that, critical depth is independent of slope.

proof: For critical flow, $Fr = 1 = \frac{V}{\sqrt{gD}}$

$$\Rightarrow V^2 = gD$$

$$\Rightarrow Q^2 = \frac{gA^3}{B}$$

$$\Rightarrow Q^2 = \frac{g\,b^3 Y_c^3}{b} \quad [\text{rectangular}]$$

$$\Rightarrow Y_c = \sqrt[3]{\frac{Q^2}{g b^2}}$$

so, critical depth is independent of slope.

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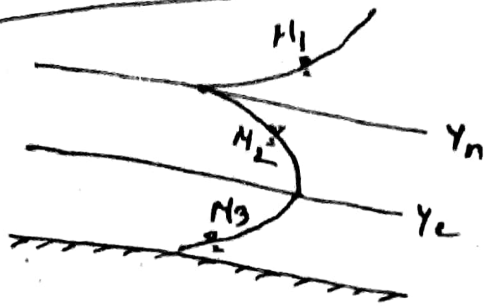
Equation of GVF:

$$\frac{dy}{dx} = S_0 \frac{1-\left(\frac{K_n}{K}\right)^2}{1-\left(\frac{Z_c}{Z}\right)^2}$$

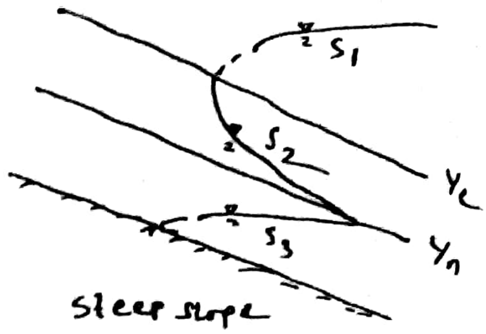
$\therefore \frac{dy}{dx} > 0 \longrightarrow$ Backwater curve

$\frac{dy}{dx} < 0 \longrightarrow$ Drawdown curve.

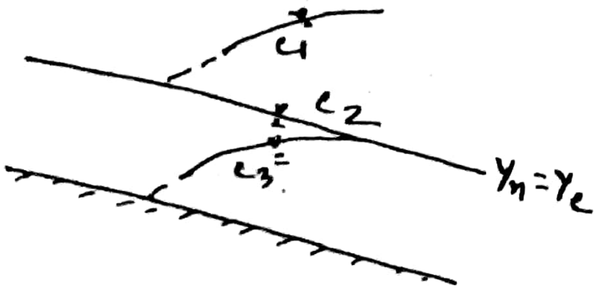
flow profile:



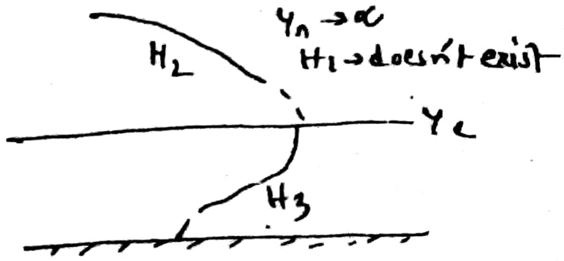
Mild slope.



Steep slope

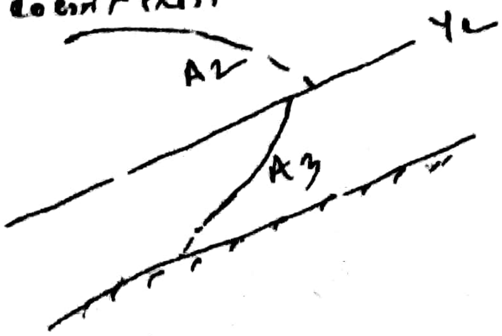


Critical slope



Horizontal slope.

$Y_n \to$ imaginary
$A_1 \to$ doesn't exist



Adverse slope.

* Computation of GVF profile:

① Depth of flow (given)

② Find normal depth, Y_n or h_n

③ Find critical depth, Y_c or h_c

* A trapezoidal channel $b = 20$ ft, $S = 2$, $S_o = 0.0016$, $n = .025$, $Q = 400$ cfs. Compute the backwater profile created by a dam ($\alpha = 1.10$) that backs up the water to a depth of 5 ft.

Soln: $A = (b + Sh)h = (20h + 2h^2)$; $R = A/P = \dfrac{20h + 2h^2}{20 + 2\sqrt{5}h}$

$P = b + 2\sqrt{1+S^2}\, h = 20 + 2\sqrt{5}\, h$; $D = A/B = \dfrac{20h + 2h^2}{20 + 4h}$

$B = b + 2Sh = 20 + 4h$

Computation of Y_n:

$$AR^{2/3} = \frac{Qn}{\phi\sqrt{S_o}} = \frac{400 \times .025}{1.486 \times \sqrt{.0016}} = 168.24$$

$$\Rightarrow (20h + 2h^2)\left(\frac{20h + 2h^2}{20 + 2\sqrt{5}h}\right)^{2/3} = 168.24$$

$\therefore h_n = 3.36$ ft

Computation of Y_c:

$$F_r = \frac{V}{\sqrt{\frac{gD}{\alpha}}} = 1$$

$$\Rightarrow \frac{Q^2}{A^2} = \frac{gD}{\alpha}$$

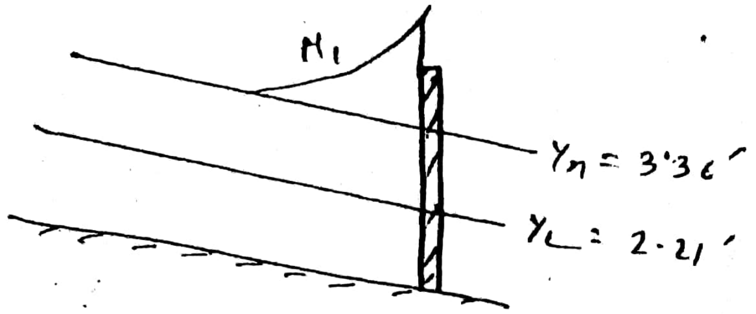
$$\Rightarrow \alpha Q^2 = \frac{gA^2 \cdot A}{B}$$

$$\Rightarrow \alpha Q^2 B - gA^3 = 0$$

$$\Rightarrow 1.1 * 400^2 (20 + 4h) - 32.2\,(20h + 2h^2)^3 = 0$$

$\therefore h_c = 2.21$ ft

$y = 5\ ft > y_n > y_c \Rightarrow M_1$ profile



Mild slope.

* A wide channel with Chezy's $C = 47\ m^{1/2}/s$ and $S_0 = 0.0001$ carries a discharge of $2\ m^2/s$. A dam raises the water level by 0.5 m above normal depth at the dam site. Compute flow profile.

Soln:

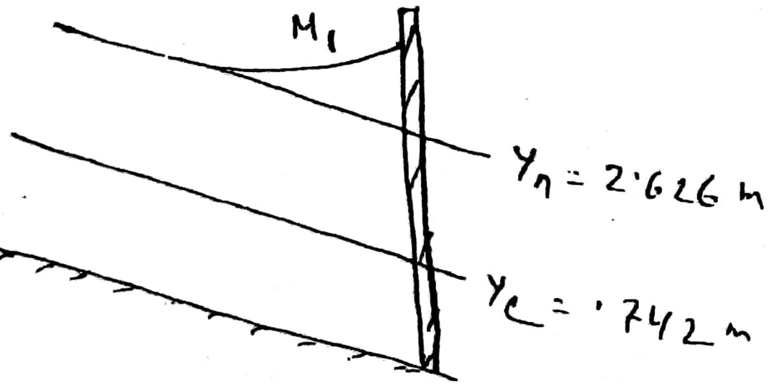
...ezy's eqn ...se করে আগের মত করতে হবে।

$$y_n = \left(\frac{q}{C\sqrt{S_0}}\right)^{2/3} = \left(\frac{2}{47\sqrt{.0001}}\right)^{2/3} = 2.626\,m$$

$$y_c = \sqrt[3]{\frac{\alpha q^2}{g}} = \sqrt[3]{\frac{1 \times 2^2}{9.81}} = 0.742\,m.$$

$$y = y_n + .5 = 3.126\ m.$$

$y > y_n > y_c \rightarrow M_1$ profile



Mild Slope.

* A sluice gate discharges 11 m^3/s. into trapezoidal channel. side slope 1:1, b = 6m, S = 0.01, n = 0.014, the depth at vena-contracta = 0.17m, compute profile d/s of gate.

<u>sol^n:</u> $Q = 11 m^3/s$, $b = 6m$, $s = 1$, $S_0 = 0.01$, $n = 0.014$

~~$y_n = 0.17m$~~. $h = 0.17m$

$$A = (b+sh)h = 6h + h^2, \quad P = b + 2\sqrt{1+s^2}\,h = 6 + 2\sqrt{2}h$$

$$B = b + 2sh = 6 + 2h, \quad R = A/P = \frac{6h+h^2}{6+2\sqrt{2}h}$$

$$D = A/B = \frac{6h+h^2}{6+2h}$$

<u>computation of y_n:</u>

$$AR^{2/3} = \frac{Qn}{\phi\sqrt{S_0}} = \frac{11 \times .014}{1\sqrt{.01}} = 1.54$$

$$\Rightarrow (6h+h^2)\left(\frac{6h+h^2}{6+2\sqrt{2}h}\right)^{2/3} = 1.54$$

$$\therefore h_n = 0.444m$$

<u>computation of y_c:</u>

$$Fr = \frac{V}{\sqrt{\frac{gD}{\alpha}}} = 1$$

$$\Rightarrow \frac{Q^2}{A^2} = \frac{gD}{1}$$

$$\Rightarrow Q^2 = \frac{gA^2 \cdot A}{B}$$

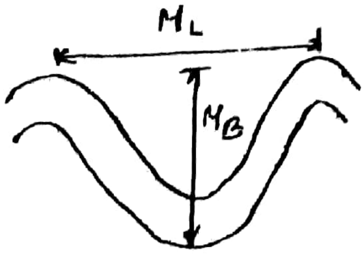
$$\Rightarrow Q^2B - gA^3 = 0$$

$$\Rightarrow 11^2(6+2h) - 9.81(6h+h^2)^3 = 0$$

$$\therefore h_c = 0.673m$$

$\therefore h_c > h_n > h \rightarrow S_3$ profile

"River Training work"



Meandering River: When a consecutive curves of reverse order connected with short straight reaches called crossing are developed in a river reach, the river is stated to be a meandering river.

Meander Length: The tangential distance between the corresponding points of a meander (M_L).

Meander belt (M_B): It is the distance between the outer edges of clockwise and anti-clockwise loops of the meander.

Meander Ratio: M_B/M_L

causes of meandering:

4 variables

① valley slope ② silt grade and silt charge.

③ Discharge ④ Bed and side materials.

cut off:

In a excessively meandering river, a particular bend may sometimes be abandaned by the formation of a shorter channel, then the abandaned channel itself is called cut off.

l area of flow and the square
d section factor.

$$Z = A\sqrt{D}$$

$$Z_c = A_c\sqrt{D_c} = \frac{Q}{\sqrt{g/\alpha}}$$

Trail for critical depth, h_c.

zoidal channel $b = 6m$, $s = 2$, Ca
for $Q = 50\ m^3/s$, $\alpha = 1$

b + sh), $B = b + 2sh$, $D = A/B$

$$= \frac{50}{\sqrt{\frac{9.81}{1}}} = 15.969$$

A (m^2)	B (m)	D (m)	$Z = A\sqrt{D}$	Remarks
8	10	.8	7.155	h [illegible] increase
20	14	1.43	23.90	h need to decrease
13.5	12	1.125	14.32	h need to increase
14.72	12.4	1.187	16.04	very close

* Using the trapezoidal rule of numerical integration, compute the discharge per unit width, the mean velocity and the numerical values of α & β for the following velocity measurements along a vertical in a wide channel, when the depth of flow is 6m.

z(m) →	0	1	2	3	4	5	6
u(m/s) →	0	2.95	3.31	3.62	3.95	4.12	4.51

Soln:

$$U = \frac{\Sigma u \Delta z}{h} = \frac{\left(\frac{0+4.51}{2} + 2.95 + 3.31 + 3.62 + 3.95 + 4.12\right) \times 1}{6}$$

$$= 3.37 \text{ m/s} \quad [\Delta z = 1 \text{ m}]$$

$$\therefore q = Uh = 3.37 \times 6 = 20.21 \text{ m}^2/\text{s}.$$

$$\alpha = \frac{\Sigma u^3 \Delta z}{U^3 h} = \frac{\left(\frac{0^3 + 4.51^3}{2} + 2.95^3 + 3.31^3 + 3.62^3 + 3.95^3 \times 4.12^3\right) \times 1}{3.37^3 \times 6}$$

$$= 1.25$$

$$\beta = \frac{\Sigma u^2 \Delta z}{U^2 h} = \frac{\left(\frac{0^2 + 4.51^2}{2} + 2.95^2 + 3.31^2 + 3.62^2 + 3.95^2 + 4.12^2\right) \times 1}{3.37^2 \times 6}$$

$$= 1.11$$

Ans.