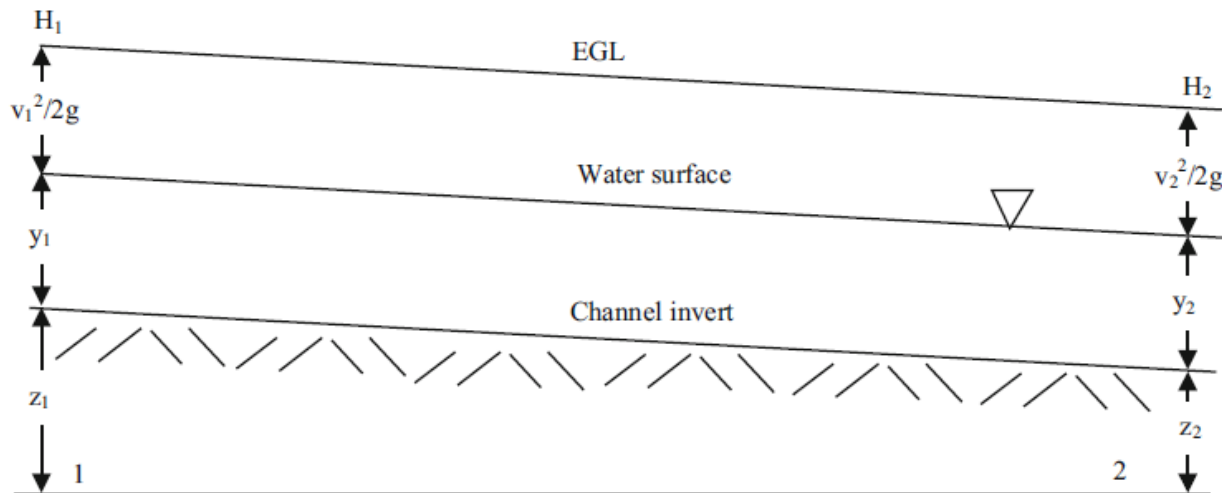


Open Channel Flow

It is a flow with free surface and is subjected to the atmospheric pressure. Figure 1 below shows a sketch of uniform open channel flow.



Where: y =depth of flow in channel,

v = average velocity of water flow in channel,

z = elevation of channel invert (bottom of channel).

The energy (H) in open channel flow is the sum of the elevation of the channel, z , and the depth of water flow, y , and the velocity head $V^2/2g$. as in equation below:

$$H = y + \frac{v^2}{2g} + z$$

Open channel flow is driven by inertial forces (momentum) and gravitational forces (differences in elevation of the water surface).

Types of flow in Open Channels

Open channels could be classified in various ways, based on flow change with time and space.

- Steady flow: A constant in time flow

Uniform flow: A constant in space flow

Nonuniform flow

- Unsteady flow

Unsteady uniform flow (Rare)

Unsteady nonuniform flow

For ***steady uniform flow*** in a channel, the water surface slope is the same as the channel slope (**Fig. 1**) and momentum is constant in time and space. Thus, the slope of the energy grade line (EGL) is equal to the slope of the channel.

The flow ***velocity*** of a steady uniform flow is determined by **Manning's** equation, which is a function of the channel roughness, energy grade line (equal to slope, S), and channel dimension (hydraulic radius).

$$v = \frac{R^{2/3} S_0^{0.5}}{n} \quad Q = \frac{AR^{2/3} S_0^{0.5}}{n}$$

Where: S_0 = channel slope, positive downward in direction of flow, m/m

n = Manning's roughness coefficient,

v = velocity of flow in the channel, m/sec,

Q = flow in channel, m³/sec,

A = cross-sectional area of channel.

The hydraulic radius, **R** , is representative of the length dimension of the channel perpendicular to the direction of flow. It is calculated as the ***cross-sectional area of the channel*** divided by the ***wetted perimeter***.

$$R = A/P$$

The slope is the difference in elevation between two points (Fig. 1) divided by the distance between the points.

For uniform flow, the difference in elevation is also equal to ***the friction loss***, H_f .

$$S_0 = \frac{z_2 - z_1}{L \cos \theta} = \frac{H_f}{L}$$

z_1 = Elevation energy at upper end of channel, m

z_2 = Elevation energy at lower end of channel, m

L = Length of channel, m

θ = channel slope, degrees.

By substituting into equations above, we get:

$$H_f = L \frac{Q^2 n^2}{A^2 R^{4/3}} = kL \frac{Q^2 n^2}{D^{5.3}}$$

State of flow

The state or behaviour of open channel flow is governed basically by the effect of viscosity and gravity relative to the inertial forces of the flow.

- Effect of viscosity. This effect is relative to the inertial forces, which can cause the flow to be *laminar*, *turbulent*, or *transitional*.

The flow becomes laminar when the viscous force is so strong with respect to the inertial force. In this type of flow, the water particles seem to move in definite smooth paths, and infinitesimally thin layers of fluid seem to slide over each other.

The flow is turbulent if the viscous force is so weak with respect to the inertial force. In this flow, the water particles move in irregular path.

Between the laminar and turbulent states, there is a mixed, or transitional state.

The effect of viscosity relative to the inertial forces can be represented by Reynold

number $Re = \frac{V.L}{\nu}$

Where V is the velocity of flow in L/T,

L is a characteristic length in Liters, which is equal to the hydraulic radius R in the case of open channels. Therefore,

$$Re = \frac{V.R}{\nu}$$

ν (nu), is the kinematic viscosity of water in L^2/T .

- Effect of gravity. The effect of gravity on the state of the flow is represented by **Froude number**, defined as $F_r = \frac{V}{\sqrt{g \cdot D}}$

Where, D is the hydraulic depth.

When $F_r = 1$, the flow is said to be critical,

$F_r > 1$, the flow is supercritical,

$F_r < 1$, the flow is subcritical.

Types of open channel

Open channels are classified according to their origin, which may be natural or artificial.

- Natural channels. They include all the watercourses that exist naturally on earth. They could be varying in sizes from tiny hillside rivulets, through brooks, streams, small and large rivers.

Underground water with free surface is also considered an open channel.

The hydraulic properties of natural channels are usually irregular.

- Artificial channels. Those are constructed or developed by human intrusion. They include navigation channels, irrigation canals and flumes, drainage ditches, spillways, etc. They are usually designed with sections of regular shapes. In engineering practice, artificial channels are given different names, such as:

Canals: They are usually long, mild sloped channels built in the ground, which maybe unlined or lined.

Flumes: They could be channels of wood, concrete or masonry. They are usually supported on or above the ground surface to carry water across a depression.

Culverts: They are covered channels of relatively short lengths built to drain water through highways and railroads.

Classification of Canals

Canals are generally trapezoidal in shape constructed on the ground to carry water to the fields either from the river or from a tank or reservoir.

They are generally classified in the following ways:

a) Classification based on the source of supply

- 1- Permanent canals, when they are fed by a permanent source of supply.
- 2- Inundation canals, they draw their supply from rivers when there is a high stage of water.

b) Classification based on the function of the canal

- 1- Irrigation canals, carry water to the agricultural fields.
- 2- Carrier canals, besides doing irrigation, they carry water for other canals.
- 3- Feeder canals, constructed with idea of feeding two or more canals.
- 4- Navigation canals.
- 5- Power canal.

c) Classification based on the boundary surface of the canal

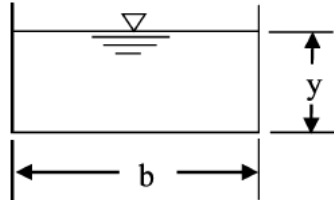
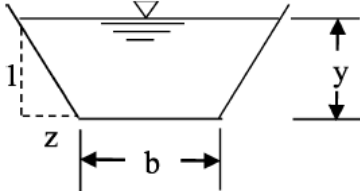
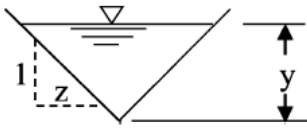
- 1- Alluvial canals, excavated in alluvial soils, such as silt.
- 2- Non-alluvial canals, excavated in non-alluvial soils, such as loam, clay, rock.
- 3- Rigid boundary canals, they have rigid sides and rigid base, such as lined canals.

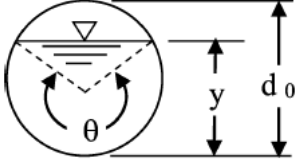
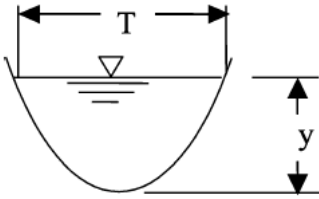
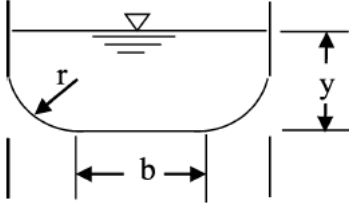
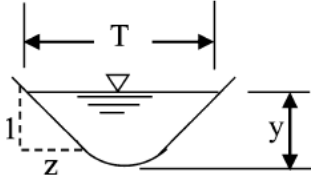
d) Classification based on relative importance in a given network of canals

- 1- Main canal, carries water directly from the river or reservoir and they cannot be used for direct irrigation due to their heavy supplies.

- 2- Branch canals, they are branches of main canals which feed major and minor distributaries, they cannot provide direct irrigation unless exceptional circumstances when direct outlets may be provided.
- 3- Major distributary, they take off branch or main canals but their discharge is usually less than that of branch canals.
- 4- Minor distributary, they take off branch canals or major distributaries. They supply water to the water courses through outlets.
- 5- Water course, they are sometimes called field channels, small canals which ultimately supply water to the field.

Geometric elements of channels

Section	Area A	Wetted perimeter, P	Hydraulic radius R	Top width T
	by	$b + 2y$	$\frac{by}{b + 2y}$	b
Rectangle 	$(b + zy)y$	$b + 2y\sqrt{1 + z^2}$	$\frac{(b + zy)y}{b + 2y\sqrt{1 + z^2}}$	$b + 2zy$
Trapezoid 	zy^2	$2y\sqrt{1 + z^2}$	$\frac{zy}{2y\sqrt{1 + z^2}}$	$2zy$
Triangle				

	$\frac{1}{8}(\theta - \sin \theta)d_0^2$	$\frac{1}{2}(\theta)d_0$	$\frac{1}{4}\left(1 - \frac{\sin \theta}{\theta}\right)d_0$	$\left(\sin \frac{\theta}{2}\right)d_0$
Culvert 	$\frac{2}{3}Ty$	$T + \frac{8y^2}{3T}$ Satisfactory for $0 < x < 1$ where $x = 4y/T$	$\frac{2T^2y}{3T^2 + 8y^2}$	$\frac{3A}{2y}$
Parabola 	$\left(\frac{\pi}{2} - 2\right)r^2 + (b + 2r)y$	$(\pi - 2)r + b + 2y$	$\frac{\left(\left(\frac{\pi}{2} - 2\right)r^2 + (b + 2r)y\right)}{(\pi - 2)r + b + 2y}$	$b + 2r$
Round-cornered rectangle 	$\frac{T^2}{4z} - \frac{r^2}{z} * (1 - z \cot^{-1} z)$	$\frac{T}{z}\sqrt{1 + z^2} - \frac{2r}{z}(1 - z \cot^{-1} z)$	A/P	$2\left[\frac{z(y - r)}{\sqrt{1 + z^2}} + r\right]$
Round-bottomed triangle				

Definitions:

The water area, **A**: is the cross section of the flow normal to the direction of the flow.

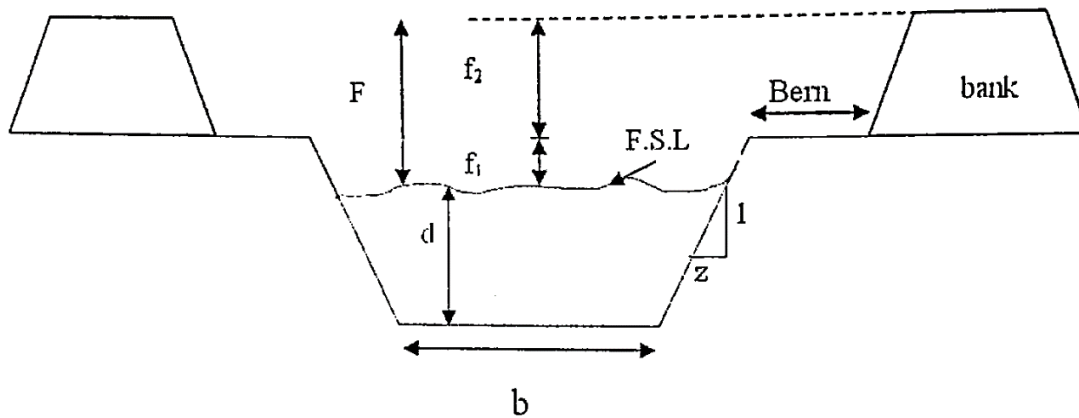
Wetted perimeter, **P**: is the length of the line of intersection of the channel wetted surface with the cross section plan normal to the direction of flow.

Hydraulic radius, **R**: is the ratio of the water area to its wetted perimeter, $R = \frac{A}{P}$.

Top width, **T**: is the width of the channel section at the free surface.

Hydraulic depth, **D**: is the ratio of the water area to the top width $D = \frac{A}{T}$.

Canal cross section elements



F.S.L: Full supply level,

b: Canal bed,

1:Z : Side slope,

Bern or Berm: A narrow strip of land, made on both sides of a channel at G.L. Its width depends on the size of the channel.

Why berms are used?

- 1- They prevent the earthwork from falling inside the canal.
- 2- They provide a scope for future widening of the canal.

Free board (F): It is the vertical distance between F.S.L and top of the lowest bank of the channel.

Free board (f_1): The distance between the water level and canal surface for the protection of over-topping.

Free board (f_2): The distance between the berm level and bank level for the protection of flooding.

$$F = f_1 + f_2, \quad f_1 = 0.2 + 0.1 d, \quad f_1 + f_2 = \frac{1}{3} d,$$

$f_2 = \frac{1}{3} * d - f_1$, should not be less than 0.2m.

Design of open channels

1- Non-alluvial channels

One of the most important equations for channel's design is Manning's formula:

$$Q = \frac{AR^{2/3}S_0^{0.5}}{n}, \text{ which is used in two methods:}$$

- Method of permissible velocity

Procedure:

Step1: Given Q, n, S_o, Z, V

Step2: Based on $Q = V.A$, compute A .

Step3: Compute R from Manning's equation.

Step4: Find $P = \frac{A}{R}$.

Step5: From A and P , find b and d simultaneously by geometry.

Step6: Add free board.

Ex: Design an irrigation channel to carry a discharge of 15 cumecs (m^3/sec) with a permissible velocity of 0.75m/sec, assume bed slope of 1/3600, Manning's coefficient of 0.03 and a side slope of 1:1.

$$\text{Ans./ } A = \frac{Q}{V} = \frac{15}{0.75} = 20 \text{ m}^2 \quad \text{Since } A = b \cdot d + z \cdot d^2,$$

$$\text{then } 20 = b \cdot d + d^2 \dots\dots\dots (1)$$

$$\text{From Manning's eq. } V = \frac{R^{2/3} S_o^{0.5}}{n} \implies 0.75 = \frac{R^{2/3} (1/3600)^{0.5}}{0.03}$$

$$R = 1.568 \text{ m}$$

$$P = \frac{A}{R} = \frac{20}{1.568} = 12.755 \text{ m}$$

$$\text{Since } P = b + 2d \sqrt{1 + z^2} \implies 12.755 = b + 2d \sqrt{1 + 1^2}$$

$$\ast b = 12.755 - 2.828 d \dots\dots\dots (2)$$

$$\text{Substituting in eq. (1)} \implies 20 = (12.755 - 2.828 d) + d^2$$

$$\ast d = 4.6 \text{ m or } d = 2.38 \text{ m.}$$

$$\text{If } d = 4.6 \text{ m then } b = -0.254 \text{ m (Neglected)}$$

$$\text{Or } d = 2.38, \text{ then } b = 6 \text{ m.}$$

$$\text{The free board: } f_1 = 0.2 + 0.1 d = 0.2 + 0.1 \cdot 2.38 = 0.438 \text{ m}$$

$$f_1 + f_2 = \frac{1}{3} d \implies f_2 = \frac{1}{3} \cdot 2.38 - 0.438 = 0.355 \text{ m.}$$

- Method of (b/d) ratio

Procedure:

Step1: Given Q, n, S_o, and Z

Step2: Assume b/d as follows:

$$-Q < 1 \text{ m}^3/\text{sec} \implies \frac{b}{d} = 2 \tan \frac{\theta}{2}$$

$$-Q = (1-10) \text{ m}^3/\text{sec} \implies \frac{b}{d} = (1-3)$$

$$-Q > 10 \text{ m}^3/\text{sec} \implies \frac{b}{d} = (3-10)$$

Step3: Use Manning's eq. $Q = \frac{A R^{2/3} S_o^{0.5}}{n}$

$$\frac{Q \times n}{S^{1/2}} = \left(\frac{b}{d} + Z \right) d^2 \times \left(\frac{\left(\frac{b}{d} + Z \right) d^2}{\left(\frac{b}{d} + 2\sqrt{1+Z^2} \right) \times d} \right)^{2/3}$$

Step4: b is rounded, return to Manning's eq. to find d.

Step5: Check minimum permissible velocity, and Fr.

$$-V_{\min} = 0.46 \text{ m/sec} \quad \text{For lined channels}$$

$$-V_{\min} = 0.76 \text{ m/sec} \quad \text{For unlined channels}$$

$$\text{And } Fr = \frac{V}{\sqrt{gD}} \text{ must be } \leq 0.6$$

If $Fr = 1$, then critical flow,

$Fr < 1$, subcritical flow,

$Fr > 1$, supercritical flow

$$D = \frac{A}{T}$$

Step6: Adding the free board.

Ex: Given the following information about a lined irrigation canal, water level at k0.0 = 32.3m, required water level at k4.8 = 27.5m, max. allowable slope = 75cm/km, min. allowable slope = 15cm/km, side slope 1.5:1, design discharge = 28m³/sec, and n = 0.015. Design the following:

- a- The proper longitudinal slope for the canal.
- b- The canal cross-section if $\frac{b}{d} = 3$.

Ans./

$$a- S_o = \frac{32.3-27.5}{4.8} = 1\text{m/km but max. allowable is } 75\text{cm/km!! Not ok!}$$

Then use $S_o = 75\text{cm/km}$.

$$b- Q = \frac{A R^{2/3} S_o^{0.5}}{n} \implies \frac{Q.n}{S_o^{0.5}} = \left(\frac{b}{d} + Z\right) d^2 * \left[\frac{\left(\frac{b}{d} + Z\right) d^2}{\left(\frac{b}{d} + 2\sqrt{1+Z^2}\right) * d} \right]^{2/3}$$

$$\frac{28 * 0.015}{\left(\frac{75}{1000}\right)^{0.5}} = (3 + 1.5) d^2 * \left[\frac{(3+1.5) d^2}{(3+2\sqrt{1+1.5^2}) * d} \right]^{2/3}$$

$$15.336 = 4.5 d^2 * \left[\frac{4.5 d^2}{(3+2\sqrt{1+1.5^2}) * d} \right]^{2/3} \implies d = 1.74\text{m,}$$

$$\text{and } b = 3 * 1.74 = 5.23\text{m} \approx 5.25\text{m}$$

Substitute value of b= 5.25m in Manning's eq. to find d.

$$Q = \frac{A R^{2/3} S_o^{0.5}}{n} \implies 15.336 = (5.25d + 1.5d^2) * \left[\frac{(5.25d + 1.5d^2)}{(5.25 + 2d\sqrt{1+1.5^2})} \right]^{2/3}$$

$$d = 1.739\text{m} \approx 1.74\text{m.}$$

$$V = \frac{Q}{A} = \frac{28}{5.25 \times 1.74 + 1.5 \times (1.74)^2} = 2.05 > 0.46 \text{ Ok.}$$

$$Fr = \frac{V}{\sqrt{gD}}, \quad \text{and} \quad D = \frac{A}{T},$$

$$Fr = \frac{2.05}{\sqrt{9.81 \times \left(\frac{5.25 \times 1.74 + 1.5 \times 1.74^2}{5.25 + 2 \times 1.5 \times 1.74} \right)}} = 0.5 < 0.6 \text{ Ok.}$$

Add free board:

$$f_1 = 0.2 + 0.1 d = 0.2 + 0.1 \times 1.74 = 0.37 \text{m} \approx 0.4 \text{m}$$

$$f_1 + f_2 = \frac{1}{3} d \implies f_2 = \frac{1}{3} \times 1.74 - 0.4 = 0.18 < 0.2 \text{m, then use } f_2 = 0.2 \text{m}$$

H.W: Given $Q=12.5 \text{m}^3/\text{sec}$, $Z=1$, $n=0.0225$ and $S_o=30 \text{cm/km}$. Design the canal cross-section and check the design if the min permissible velocity $=0.76 \text{m/sec}$.

Ans. ($d=2.69$, $V=0.94 \text{m/sec}$, $Fr=0.227$)

QB: What are the types of open channels? And explain the effect of viscosity on the flow.

QA: A- Classify canals according to their importance in the network.

B- Define culverts.

2- Design of alluvial canals

It is a process of controlling the flow velocity in such a way that the silt flowing in the channel is not dropped in the bed without scouring the channel.

Some of theories towards the design of non-silting non-scouring channel section are presented:

1) Lacey's Theory

The fundamental requirements stated by Lacey for a channel are as follows:

- The channel flows uniformly in incoherent alluvium. Incoherent alluvium is the loose granular material which can scour or deposit with the same ease.
- The characteristics and the discharge of the sediment are constant.
- The water discharge in the channel is constant.

The channel design procedure is as follows:

Step 1: Calculate the silt factor $f = 1.76 \sqrt{m_d}$

Where f = silt factor

m_d = diameter of silt in mm, or it's called mean particle size.

Step 2: Find the velocity $V = \left(\frac{Q f^2}{140} \right)^{1/6}$

Step 3: Determine the area $A = Q/V$

Step 4: Compute $P = 4.75 \sqrt{Q}$, and $R = 0.48 \left(\frac{Q}{f_1} \right)^{1/3}$ and $S_o = 0.0003 \left(\frac{f_1^{5/3}}{Q^{1/6}} \right)$

Ex: Design a stable channel for carrying a discharge of $30\text{m}^3/\text{s}$ using Lacey's method assuming silt factor equal to 1.0.

Ans.

$$P = 4.75 \sqrt{Q} = 4.75 \sqrt{30} = 26.02\text{m}$$

$$R = 0.48 \left(\frac{Q}{f_1}\right)^{1/3} = 0.48 \left(\frac{30}{1}\right)^{1/3} = 1.49\text{m}$$

$$S_o = 3 \times 10^{-4} \left(\frac{f_1^{5/3}}{Q^{1/6}}\right) = 3 \times 10^{-4} \left(\frac{1^{5/3}}{30^{1/6}}\right) = 1.702 \times 10^{-4}$$

*Assume final side slope of the channel as 0.5H:1V (**generally observed field value**)

$$P = b + 2d \sqrt{1 + z^2} \implies 26.02 = b + 2d \sqrt{1 + 0.5^2}$$

$$b = 26.02 - 2.24d \dots\dots\dots (1)$$

$$\text{and } A = b d + z \cdot d^2 = P \cdot R \implies 26.02 \times 1.49 = b d + 0.5 d^2$$

$$38.77 = 26.02d - 2.24d^2 + 0.5d^2 \implies 1.74d^2 - 26.02d + 38.77 = 0$$

Either $d = 13.28\text{m}$, then b would be a negative value!! **Neglected.**

$$\text{Or } d = 1.68\text{m}, \text{ then } b = 26.02 - 2.24 \times 1.68 = \mathbf{22.23\text{m}}$$

Then add free board

H.W: Design an irrigation channel for a discharge of $50\text{m}^3/\text{s}$ adopting the available ground slope of 1.5×10^{-4} . Then find the size of suitable river bed material.

(**Ans.** $d = 1.99\text{m}$, $b = 29.13\text{m}$, $m_d = 0.3\text{mm}$), **then compute the free board.**

2) Kennedy's Theory

The procedure of this theory is

Step 1: Assume (d) by trial and error, where d is the depth of water in (m).

Step 2: Calculate the *critical velocity** in (m/sec) by using eq. $V_o = 0.55 d^{0.64}$

Which is defined as the mean velocity that will not allow scouring or silting in a channel having depth of flow equal to (d)m.

Step 3: Find the area of the section $A = \frac{Q}{V_o}$

Step 4: Since $A = bd + z \cdot d^2$, then find b.

Step 5: Find P and R.

Step 6: Find (V) from Manning's equation, which must equal to (V_o). If not, then assume another (d).

**This critical velocity should be distinguished from the critical velocity of flow in open channels corresponding to Froude number equal to 1.*

Ex: Design a channel carrying a discharge of 30m³/s with **critical velocity ratio** and Manning's n equal to 1.0 and 0.0225, respectively. Assume that the bed slope is equal to 1 in 5000.

Ans. Assume d=2m

$$V_o = 0.55m d^{0.64} = 0.55 (2)^{0.64} = 0.857 \text{ m/sec}$$

$$A = Q/V_o = 30/0.857 = 35.01 \text{ m}^2$$

For a trapezoidal channel with side slope 1H : 2V, or 0.5H:1V

$$A = bd + z.d^2 \implies 35.01 = 2b + 0.5(4) \implies b = 16.51 \text{ m}$$

$$R = \frac{A}{P} = \frac{35.01}{16.51 + 2 \times 2(\sqrt{1 + 0.5^2})} = 1.67 \text{ m}$$

Therefore, from Manning's eq.

$$V = \frac{1}{n} S_o^{1/2} R^{2/3} = \frac{1}{0.0225} \left(\frac{1}{5000}\right)^{1/2} 1.67^{2/3} = 0.885 \text{ m/sec}$$

Since the velocities obtained from Kennedy's equation and Manning's equation are appreciably different, assume $d = 2.25 \text{ m}$ and repeat the above steps.

$$V_o = 0.55 \cdot 2.25^{0.64} = 0.924 \text{ m/sec}$$

$$A = 30 / 0.924 = 32.47 \text{ m}^2$$

$$b(2.25) + (0.5)(2.25)^2 = 32.47 \implies b = 13.31 \text{ m}$$

$$R = 1.77 \text{ m}$$

$$V = \frac{1}{n} S_o^{1/2} R^{2/3} = 0.92 \text{ m/s}$$

Since the two values of the velocities are matching, the depth of flow can be taken as equal to ($d = 2.25 \text{ m}$) and the width of trapezoidal channel ($b = 13.31 \text{ m}$).

Then add the fee board. f_1

***Report.**

Deadline: 11/4/2019

About Open channels. Problems, issues, Maintenance and management.

Min. 8 pages, max 12 pages.

To reduce the number of trials in finding (d), you can use the table below to use recommended b/d values for assumption.

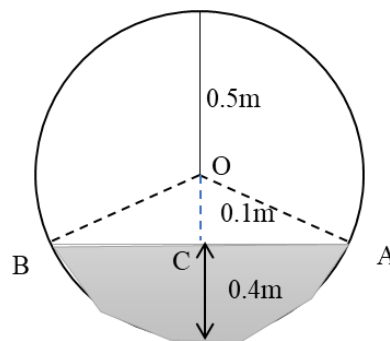
$Q \text{ (m}^3/\text{s)}$	5.0	10.0	15.0	50.0	100.0	200.0	300.0
b/d	4.5	5.0	6.0	9.0	12.0	15.0	18.0

H.W: A stable channel is to be designed for a discharge of $40\text{m}^3/\text{s}$. Calculate the dimensions of the channel using Kennedy's method. Use a Manning's coefficient of 0.0225 and bed slope of $1/5000$.

Ex: A circular concrete culvert of diameter 100cm carries water of depth 40cm to irrigate a 20 hectares field. What could be the discharge of this canal if the slope was 0.002? Use Manning's n of 0.014.

Ans.

$$\text{Length BC} = \sqrt{(0.5)^2 - (0.1)^2} = 0.49\text{m}$$



$$\text{Area BoA} = \frac{2 \times 0.49 \times 0.1}{2} = 0.049\text{m}^2$$

$$\text{Angle CoB} = \cos^{-1} \frac{0.1}{0.5} = 78.46^\circ$$

$$\ast \text{ Angle AoB} = 2 \times 78.46 = 156.92^\circ$$

$$\frac{180^\circ}{\text{half circle area}} = \frac{156.92^\circ}{\text{circle section area}}, \quad \ast \text{ circle section area} = 0.342\text{m}^2$$

$$\ast \text{ water area} = 0.342 - 0.049 = 0.293\text{m}^2$$

$$\frac{360^\circ}{\text{circle perimeter}} = \frac{156.92^\circ}{\text{wetted perimeter}}, \quad \ast \text{ wetted perimeter (P)} = 1.37\text{m}$$

$$\ast Q = \frac{1}{n} R^{2/3} S_o^{1/2} A = \frac{1}{0.014} \left(\frac{0.293}{1.37}\right)^{2/3} (0.002)^{1/2} (0.293)$$

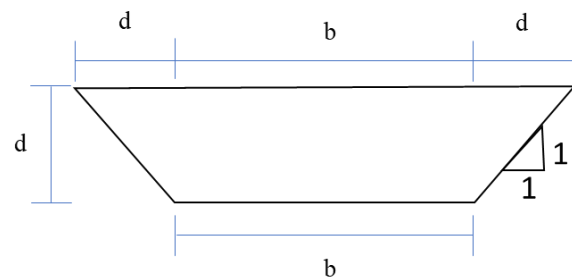
$$\ast Q = 0.335 \text{ m}^3/\text{sec}.$$

H.W: Design the best cross-section of a trapezoidal canal to irrigate a field of 1000 hectare, where the water supply is 100 (m³/hectare/day). Knowing that the side slope is 1:1 and bed slope of 10 cm/km. Use $n = 0.02$.

Ans. (d=1.133m, b=1.602m)

Hint: $Q = \text{water duty} \times \text{area} = (100 \times 1000) / (24 \times 3600) = 1.157 \text{ m}^3/\text{s}$

Find the relationship bet. b & d. then solve..



H.W: The slope of a channel in alluvium is 1/5000; Lacey's silt factor is 0.9. Find the channel section and maximum discharge which can be allowed to flow in it.

(Ans: $b=7.45\text{m}$, $d=0.88\text{m}$, $Q=3.924\text{m}^3/\text{sec}$)

Quiz: Check the adequacy of an irrigation channel of dimensions ($b=9.69\text{m}$, $d=1.7\text{m}$), which is designed to supply 85 ($\text{m}^3/\text{hectare}/\text{day}$) to a 10000 hectares field. Use Kennedy's theory with $n = 0.0225$, $S_o=1/5000$.

Sol. Required Q = $(85 * 10000)/(24*3600) = 9.84 \text{ m}^3/\text{sec}$

$$V_o = 0.55 d^{0.64} = 0.773 \text{ m/sec}$$

$$A = bd + zd^2 = 17.91 \text{ m}^2$$

$$\text{Available Q} = A V = 17.91 * 0.773 = 13.85 \text{ m}^3/\text{sec} > 9.84 \text{ m}^3/\text{sec}$$

❖ The canal is adequate.

Comparisons between Lacey's and Kennedy's theories

- 1- Kennedy introduced the Critical Velocity Ratio (m) but he did not give any idea of how to measure m . While Lacey introduced the silt factor (f) and suggested a method to determine(f) by relating it with particle size.
- 2- Kennedy assumed that silt is kept in suspension because of eddies generated from the bed only and so he proposed a relation between V and d . Lacey assumed that silt is kept in suspension because of eddies generated from the entire perimeter and so he proposed a relation between V and R .
- 3- Kennedy gave no formula of determination of longitudinal slope of the canal, where the slope is to be given based on experience. Lacey gave a formula for the longitudinal slope of the canal.

- 4- Lacey's theory does not involve any trial and error in the design procedure whereas Kennedy's theory involves a trial and error procedure.
 - 5- The basic concept of both theories is the same that the silt remains in suspension due to the force of vertical eddies.
-

Losses in Canals

When water continuously flows in a canal, losses take place due to seepage, deep percolation and evaporation. These losses should be properly accounted for, otherwise lesser quantity of water will be available for cultivation at the tail end. These losses are sometimes called transmission loss.

Water losses in canals can be classified into three categories:

- i) Evaporation losses.

This loss is generally a small percentage of the total loss in unlined canals (1 to 2 percent of the total water entering the canal).

The evaporation losses depend on the following:

- Climate factors, such as temperature, humidity and wind velocity.
- Canal factors, such as water surface area, water depth and velocity of flow.

The evaporation losses are maximum in unlined canals due to the wider water surface area and shallower water depth.

The average evaporation loss per day at hot summers may vary between 4mm to 10mm.

ii) Transpiration losses.

This loss takes place through lots of vegetation and weeds growing along the bank of the canal. This loss forms an extremely small part of total losses.

iii) Seepage losses.

These losses form the major loss in an unlined canal. The seepage losses are due to:

- Absorption of water in the upper layers of soil below the canal bed,
- Percolation of water into water table. Percolation losses are always much more than absorption losses.

The seepage loss varies with the type of the material through which the canal runs. Obviously, the loss is greater in coarse sand and gravel, less in loam, and still less in clay soil.

Rate of water loss

Canal loss maybe expressed in any of the following methods:

- 1- As cumecs per million sq. meter of wetted perimeter (most recommended)
- 2- As depth of water lost per day over the wetted perimeter.
- 3- As percentage of the canal discharge.
- 4- As percentage per kilometer length of the canal.

In absence of any other data, the canal losses (transmission losses) may be taken as:

2.5 cumecs per million sq. meter of wetted perimeter of **unlined canals** (**2.5 m³/sec/10⁶m²**), and

0.60 cumecs per million sq. meter of wetted perimeter of lined canals.

The following empirical relation has also been found to give comparable results

$$q = 1/200 (b + d)^{2/3}$$

where q is the loss expressed in m³/sec per kilometer length of canal and (b and d) are, respectively, canal bed width and depth of flow in metres.

Canal Outlets

An outlet is a small masonry structure which admits water from a distributary channel to a watercourse or a field channel. It also acts as a discharge measuring device. The discharge through an outlet is usually less than $0.085 \text{ m}^3/\text{s}$.

The main objective of providing an outlet is to provide ample supply of water to the fields, whenever needed.

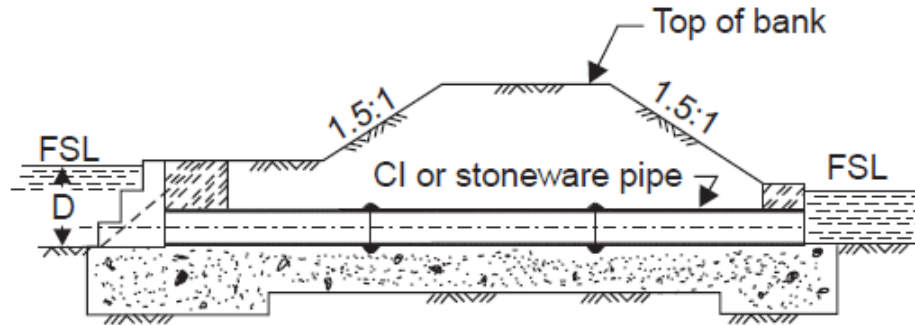
Canal outlets should satisfy the following requirements:

- i- They must be strong and simple with no moving parts, periodic attention and maintenance are necessary.
- ii- Farmers should not be able to tamper with its functioning, and any interference should be easily detectable.
- iii- The cost of outlets should be cheap, because there is a large number of them will be installed in an irrigation network.
- iv- The outlet should draw its fair share of silt carried by the distributary channel.
- v- The outlets should be able to function efficiently even at low heads.
- vi- The design should be simple that they can be constructed or fabricated by local masons or technicians.

Types of outlets

Canal outlets are of the following three types:

(i) *Non-modular outlets*: The discharge capacity in these outlets depends on the difference of water levels in the distributary and the watercourse. Example of these are **submerged pipe outlets, masonry sluices and orifices** which are fixed in the canal bank at right angles to the direction of flow in the distributary. As shown in the submerged pipe outlet in figure below.



(ii) *Semi-modular, or flexible module outlets*: The discharge in this type depends only on the water level in the distributary and is unaffected by the water level in the watercourse. The exit end of the pipe is placed higher than the water level in the watercourse. Examples of this type are **pipe outlets, open flume outlets**.

(iii) *Modular outlets*: Here the discharge is independent of the water levels in the distributary and watercourse. A modular outlet supplies fixed discharge. In case of excess or deficient supplies in the distributary, the tail-end reach of the distributary may either get flooded or be deprived of water. The most common outlets of this type are **Gibb's rigid modules**.

If H is the difference in water levels of the distributary and the watercourse then the discharge Q through the outlet can be obtained from the equation:

$$Q = A.V = \left(\frac{\pi}{4} d^2\right) \sqrt{2gH} \left(\frac{d}{1.5d + fL}\right)^{1/2} = CA \sqrt{2gH}$$

$$\text{Where } H = \frac{V^2}{2g} \left[1.5d + f \frac{L}{d}\right] \implies V = \sqrt{2gH} \left(\frac{d}{1.5d + fL}\right)^{1/2}$$

$$\text{And } C = \left(\frac{d}{1.5d + fL}\right)^{1/2} \text{ (which is called the coefficient of discharge)}$$

d = diameter of pipe outlet, L = length of pipe outlet, f = friction factor of pipe.

Parameters for Studying the Behaviour of Outlets

To understand the criteria of judging the behaviour and function of outlets, the following definitions are useful:

-Flexibility: The ratio of the rate of change of discharge of an outlet to the rate of change of discharge of the distributary channel, which is designated as F .

$$F = \frac{dq/q}{dQ/Q},$$

where F = flexibility, q = discharge through the outlet, Q = discharge through the distributary channel.

Now, for the watercourse:

$$q = kH^m$$

where k =constant, m =outlet index, H = head acting on the outlet.

$$\frac{dq}{q} = m \frac{dH}{H} \dots\dots\dots (1)$$

Similarly, for the distributary channel,

$Q = CD^n$, where Q = discharge of the distr. channel, C =constant, n = canal index, D = depth of the distr. channel.

$$\frac{dQ}{Q} = n \frac{dD}{D} \dots\dots\dots (2)$$

Dividing (1) and (2), we get:

$$F = \frac{dq/q}{dQ/Q} = \frac{m \frac{dH}{H}}{n \frac{dD}{D}} = \frac{m}{n} \frac{D}{H} \frac{dH}{dD}$$

Since any change in the water depth causes equal change in the head causing flow, we have $dH=dD$. Then the flexibility expression becomes:

$$F = \frac{m}{n} \frac{D}{H}$$

Depending upon the value of F , the outlets can be classified as:

- (i) proportional outlets ($F = 1$), where the rate of change of outlet discharge equals that of the distributary discharge.
- (ii) hyper-proportional outlets ($F > 1$), and
- (iii) sub-proportional outlets ($F < 1$).

-Sensitivity: It is the ratio of the rate of change of discharge of an outlet to the rate of change in the water surface level of the distributary channel with respect to the depth of flow in the channel.

$$S = \frac{dq/q}{dG/D}$$

Where: S = sensitivity of the outlet, q = discharge through the outlet, G = gauge reading ($G=0$ when $q=0$), D = depth of water in the distr. channel.

$$\ast dG = dD \implies S = \frac{dq/q}{dD/D} \dots\dots\dots (1)$$

$$\text{But } F = \frac{dq/q}{dQ/Q} \text{ where } \frac{dQ}{Q} = n \frac{dD}{D}$$

$$\ast F = \frac{dq}{q} / n \frac{dD}{D} \dots\dots\dots (2)$$

From (1) and (2), we get:

$$S = n F$$

It is evident from this equation that the sensitivity of **modular outlets** is **zero**.

Ex: A semi-modular pipe outlet of diameter 15cm is to be installed on a distributary with its bed level and full supply level at 100.3m and 101.5m, respectively. The maximum water level in the watercourse is at 101.15 m. Set the outlet for maximum discharge and calculate it. The coefficient C in the discharge equation may be taken as 0.62. Is the setting proportional, sub-proportional or hyper-proportional? Take $m=1/2$, $n=5/3$.

Ans. $D = 101.5 - 100.3 = 1.2\text{m}$

$$H = (101.5 - 101.15) - \frac{0.15}{2} = 0.275 \text{ m}$$

$$Q = CA \sqrt{2gH} = 0.62 * \frac{3.14 * 0.15^2}{4} \sqrt{2 * 9.81 * 0.275} = 0.0254 \text{ m}^3/\text{sec}$$

$$\text{Flexibility, } F = \frac{m}{n} \frac{D}{H} = \frac{1/2}{5/3} \frac{1.2}{0.275} = \mathbf{1.309 > 1}$$

Therefore, the setting is hyper-proportional.

H.W: A submerged pipe outlet has the following data: F.S.L of distributary= 100.00m, F.S.L of watercourse= 99.90m, **length of the pipe= 9m, diameter of pipe= 20cm**, coefficient of friction= 0.005. Find the discharge through the outlet.

(**Ans.** **0.033 m³/sec**)

Ex: Design a submerged pipe outlet for the following data:

Discharge through outlet = 0.04 cumecs,

F.S.L of distr. channel = 100.00m,

F.S.L of watercourse = 99.90m,

Full supply depth of distr. channel = 1.1m

Assume the discharge coefficient = 0.7

Ans. Given $q = 0.04$ cumecs and $C = 0.7$

Available head (H) = $100 - 99.9 = 0.1$ m

$$q = CA \sqrt{2gH} \implies 0.04 = 0.7 A \sqrt{2 * 9.81 * 0.1}$$

$$A = 0.0408 \text{ m}^2 = \frac{3.14 * d^2}{4} \implies d = \sqrt{\frac{4 * 0.0408}{3.14}} = 0.228 \text{m} = 22.8 \text{cm}$$

Hence use a pipe of 25cm diameter.

A very important H.W: A distributary channel having bed width 5.00m and full supply depth of 1.20m carries $3.0 \text{m}^3/\text{sec}$ of discharge. A semi-modular pipe outlet in this channel has a command area requires $0.0165 \text{m}^3/\text{sec}$. Determine the size of the outlet and set it for sub-proportionality with a flexibility of 0.9. Assume the length of the pipe as 3.0m and friction factor as 0.03. The available diameters of the pipe are 150, 125, 100, and 75 mm.

How does this outlet behave if the distributary runs below FSL at 1.0m depth? Take $m=1/2$, $n=5/3$.

Ans.

$$F = m/n * D/H$$

$$H = 0.4 \text{m}$$

$$Q = \left(\frac{\pi}{4} d^2\right) \sqrt{2gH} \left(\frac{d}{1.5d + fL}\right)^{1/2} = \left(\frac{\pi}{4} d^2\right) \left(\frac{d(2gH)}{1.5d + fL}\right)^{1/2} = \left[\left(\frac{\pi}{4}\right)^2 \frac{d^5(2gH)}{1.5d + fL}\right]^{1/2}$$

$$Q = \left[\frac{4.84 d^5}{1.5d + 0.09} \right]^{1/2}$$

Assume $d=0.1\text{m}$, then $Q=0.0142\text{ m}^3/\text{sec}$, which is less than required ($0.0165\text{m}^3/\text{sec}$)

For $d=0.125\text{m}$, then $Q=0.023\text{ m}^3/\text{sec}$, OK!

Therefore, recommended diameter of the outlet is 125 mm.

When the distributary is running at 1.0 m depth (i.e., 0.2m below FSL) then:

$$H = 0.4 - 0.2 = 0.2\text{m and } D = 1.0\text{m.}$$

$$F = \frac{1/2}{3/5} \frac{0.1}{0.2} = 1.5 > 1.0$$

Therefore, the outlet behaves as **hyper-proportional** outlet.

Maintenance of irrigation channels

After completing the construction of an irrigation system, it becomes essential to maintain it for its proper and efficient functioning.

Proper maintenance of an irrigation system is necessary because of the following:

- (i) The delivery system of the irrigation project should be in a good condition for effective water management and to retain system's operation efficiency.
- (ii) Practical, predictable and equitable deliveries to the outlets will increase crop productivity from the existing irrigation systems and such deliveries can be assured by timely inspection and maintenance.
- (iii) Conservation of precious water resource for irrigation is essential. Economically viable project sites are shrinking and exploitation of sites in different terrain for future development with available technology is capital intensive and requires huge public funds.

Irrigation channels may stop functioning properly due to the following reasons:

- i- Silting of canal
- ii- Breaching of canal due to weak banks
- iii- Growth of weed
- iv- Overflow of canal banks

The maintenance will then focus on the following:

- 1- Silt removal

When the silt is deposited on the bed and sides, the capacity of the channel reduces. Therefore, it is better to exclude silt from the channel.

The following methods are used to remove the silt:

- (a) **Flushing.** Clear water is flushed in the canal to lift up deposited silt. Absolutely clear water should be used for flushing, but if this is not available then the water which contains a minimum silt quantity could be used for flushing.
- (b) **Silt scouring fleet.** The method involves agitating the silt by continuous maneuvering of three special boats. This method was only used in Punjab, India and was not successful.
- (c) **Bundle of thorny bushes.** These bushes are tied together and pressed down the canal by stones, then they are moved inside the channel by animals. They are useful in agitating the fine muddy silt.
- (d) **Iron rakes.** They are also dragged down the channel to dislodge silt.
- (e) **Reduction of area of flow.** Special loaded boats are put across the section of the channel to reduce the area of flow and increase the speed of flow.
- (f) **Stirring of silt by water jet.** A pump is fitted with a pipe and nozzle and placed on a boat. The high velocity jet is directed to the bed and stir silt.
- (g) **Dredging.** This method is very costly and rarely used for silt removal of canals because of the high cost.
- (h) **Excavation.** The silt deposited in the canal is cleared by manual labour. This method is quite costly, but it is used in clearing distributaries and minors. The silt must be deposited clear off the channel so that it cannot find its way back to the channel.

2- Weed control.

Water weeds are unwanted plants that grow profusely in water under certain conditions. They tend to reduce the discharge of the canal by reducing the area of canal cross-section and velocity of flow.

Weed growth is not possible in channels having high velocity of flow, but when velocity of flow becomes less than 0.6 m/sec weed growth is generally possible.

The most common method of weed control is called **rush rotation**. In this process, the channel is set to run with full discharge for a specified time then left to dry completely for some time. Long closure duration has killing effect on growth of weeds.

Canal breaches

Canal breach or tearing off the canal banks usually takes place when canals are in fill. During a canal breach, a big gap forms on the side and the large canal discharge rushes out doing unlimited damage.

Reasons for canal breaching:

- 1- ***Breaching due to defective design and construction of the banks***. The canal banks are not strong enough to resist water pressure.
- 2- ***Breaching due to exposure of saturation gradient***. The soil particles on the outer slope are dislodged and flown away along with the seeping water.
- 3- ***Breaching due to rush of water through rat holes***.
- 4- ***Piping in the canal banks***.
- 5- ***Willful cuts in the canal banks***. This happens for unauthorized irrigation or for allowing the flood water to get into the canal to save an area from submergence.

How to repair a breach?

A breach can be repaired by the procedure below:

- i- The canal flow should be diverted to a nearby escape channel or the canal should be closed from the head to stop the outflow completely.
- ii- Provide steps on the sides of the gap so that the deposited soil could form a good bond with the old surface.

- iii- Put two lines of piles in the inner side of the bank and fill the space in between with earth, which should be put in layers and compacted to the optimum moisture content. Then put bushes at top.
 - iv- The new bank should be inspected uniformly and carefully for few days to ensure its proper functioning.
-

Types of maintenance

Maintenance work include operations performed in preserving system and facilities in good or near-original condition. Repairs are part of maintenance.

Maintenance can be of the following types:

1. Normal — usual; done annually.
2. Emergency — done under unusual conditions that are adversely affecting the safety of the system.
3. Deferred — routine/normal ones are deferred because of shortage of funds/machines, affects adversely hydraulic performance.
4. Catch-up — programme that takes care of deferred maintenance
5. Preventive — takes care of the causes creating the maintenance needs when they are only minor.
6. Rehabilitation — renovation needed due to accumulation of deferred maintenance; otherwise, only for ageing structures.
7. Modernisation — updating/improving the system to meet enhanced social, technical and economic activities.

8. 'Walk-thru' — two or three individuals walking along canals, taking notes on maintenance needs

9. Special — due to unforeseen mishaps such as breaches.

Sources of maintenance data

The data to be used in preparing maintenance plans may originate from one of the following:

- reports from field personnel
 - inspection report from superior officers
 - performance measurement data, and
 - research reports on maintenance (material, equipment, method).
-

Evaluation of performance of an irrigation system

Some of the diagnostic analyses that can be considered are:

(i) Farmers operation performance

- adequacy of crop production techniques for irrigated farming
- adequacy of irrigation methods
- farm management and economic results
- soil management and erosion control, and
- on-farm efficiency of water use.

(ii) Delivery operational performance

- water use efficiency in distribution

- water losses (physical including evaporation)
- project overall water use efficiency
- deep percolation
- canal seepage
- spillage from canals
- dam and foundation seepage
- water operational losses (such as leakage from gates, etc.)
- adequacy of delivery scheduling, and
- energy use.

(iii) ***Drainage operational performance***

- drainage requirement
- water table fluctuations by seasons and years
- water quality changes
- soil salinity changes.

Q: Describe the common criteria for judging the performance of an irrigation system.

Lining of irrigation canals

Most of irrigation channels in Iraq are earth channels. The advantage of an earth channel is its low initial cost.

The disadvantages of an earth channel are:

- i- the low velocity of flow to prevent erosion needs larger cross-section of channels,
- ii- excessive seepage loss,
- iii- favourable conditions for weed growth which further retards the velocity,
- iv- breaching of banks due to erosion and burrowing of animals.

These problems of earth channels can be got rid of by lining the channel.

Lining of canals is necessary:

- 1- to minimize the seepage losses in canals
- 2- to increase the discharge in canal section by increasing the velocity
- 3- to prevent erosion of bed and side due to high velocities
- 4- to prevent or minimise the growth of weed
- 5- to reduce the maintenance needs of the canal.

Disadvantages of canal lining:

The canal lining has certain disadvantages although the advantages far exceed them.

The disadvantages can be shown below:

- 1- Canal lining requires heavy initial investment.
- 2- It is difficult to shift the outlet very often because the lining is usually permanent.
- 3- It is very difficult to repair damaged lining.
- 4- Lined canals are usually without berms. This means the safety of vehicular or pedestrian traffic is absent.

Types of canal lining

Concrete lining

Concrete lining is probably the best type of lining.

It is durable, impervious, and requires least maintenance. The smooth surface of the concrete lining increases the conveyance of the channel.

Properly constructed concrete lining can easily last about 40 years.

Concrete linings are suitable for all sizes of channels and for both high and low velocities.

The lining cost is high; however, it can be feasible when compared to the life of lining and the decrease in maintenance.

Some cracks usually develop in concrete linings. These can be sealed with asphaltic compounds.

The concrete lining may be damaged when flow in the canal is suddenly stopped and the surrounding water table is higher than the canal bed.

This damage occurs in excavated channels and can be prevented by providing

weep holes in the lining or *installing drains with outlets in the canal section*.

Values of minimum thickness of concrete lining based on canal capacity have been specified as given in Table below:

Canal capacity (m^3/s)	Thickness of M-150 concrete (cm)		Thickness of M-100 concrete (cm)	
	Controlled	Ordinary	Controlled	Ordinary
0 to less than 5	5.0	6.5	7.5	7.5
5 to less than 15	6.5	6.5	7.5	7.5
15 to less than 50	8.0	9.0	10.0	10.0
50 to less than 100	9.0	10.0	12.5	12.5
100 and above	10.0	10.0	12.5	15.0

Shotcrete lining

Shotcrete lining is constructed by applying cement mortar pneumatically through a nozzle to the canal surface. Cement mortar does not contain coarse aggregates and, therefore, the proportion of cement is higher in shotcrete mix than in concrete lining (1:4). The size of sand particles in the mix should not exceed 0.5 cm.

The thickness of the shotcrete lining may vary from 2.5 to 7.5 cm.

Shotcrete lining is suitable for:

- (a) lining small sections,
- (b) placing linings on irregular surfaces without any need to prepare the subgrade,
- (c) placing linings around curves or other structures,
- (d) repairing seriously cracked and/or leaky old concrete linings.

Shotcrete linings are subject to cracking and may be reinforced or unreinforced. The reinforcement is in the form of **wire mesh**. In order to reduce costs, shotcrete linings are not reinforced these days, particularly on relatively small jobs.

Precast concrete lining

Precast concrete slabs laid properly on subgrades and with joints effectively sealed. The precast slabs are about **5 to 8 cm thick**.

Such slabs may or may not be reinforced.

This type of lining is best suited for repair work as it can be placed rapidly without long interruptions in canal operation.

Stone masonry lining

Stone masonry linings are laid on the canal surface with cement mortar or lime mortar. The thickness of the stone masonry is about **30 cm**.

The surface of the stone masonry may be smooth to increase the hydraulic efficiency of the canal.

Stone masonry linings are stable, durable, erosion-resistant, and very effective in reducing seepage losses.

Brick lining

Bricks are laid in layers of two bricks with **1:3** cement mortar sandwiched in between.

Good quality bricks should be used, and these should be soaked well in water before being laid on the moistened canal surface.

Brick lining is suitable when concrete is expensive and skilled labour is not available.

Asphalt lining

The material used for asphaltic lining is asphalt-based combination of cement and sand mixed in hot condition. The most commonly used asphaltic linings are:

- (a) asphaltic concrete.
- (b) buried asphaltic membrane.

Asphaltic concrete is a mixture of asphalt cement, sand, and gravel mixed at a temperature of about 110°C and is placed either manually or with laying equipment.

Asphaltic linings have short life and are unable to permit high velocity of flow. They have low resistance to weed growth and, hence, it is advisable to sterilise the subgrade to prevent weed growth.

Earth lining

Different types of earth linings have been used in irrigation canals. They are inexpensive but require high maintenance expenditure.

The main types of earth linings are:

- (a) stabilised earth linings.
- (b) loose earth blankets.
- (c) compacted earth linings.
- (d) buried bentonite membranes.
- (e) soil-cement linings.

Economics of canal lining

The lining of canals is to be done on financial considerations besides the technical feasibility. To determine the economic viability, the money gained from canal lining should be more than those spent on lining works.

Calculation of benefit:

The major benefit can be gained from the saving of seepage water.

Let q cumecs be the water saved from seepage when lining a canal, and R_1 dollars be the cost of water per cumecs. Then:

The total saving = $q.R_1$ dollars.

The other benefit is through the reduction of maintenance cost. Let p be the percentage of annual savings in maintenance cost, which is R_2 dollars, then:

The annual maintenance saving = $p.R_2$ dollars.

Then, the total benefit (B) per year = $(q.R_1 + p.R_2)$

Saving in maintenance cost is usually taken to be around 40%. Therefore, $p = 0.4$

Annual cost of extra expenditure on lining

Let the extra expenditure on lining be C dollars, if P_t is the total perimeter of lining, L is the length of lining, and c is the cost of lining in dollars/m².

$$C = c.P_t.L$$

Let the life of lining be N years and i be the percentage rate of interest as a fraction per year. Then, the extra expenditure on lining should be equal to:

$$= C.i (i + 1)^N [(i + 1)^N - 1]^{-1}$$

$$\text{Therefore, the benefit cost ratio (BCR)} = \frac{(q.R_1 + p.R_2)[(i+1)^N - 1]^{-1}}{C.i (i+1)^N}$$

$$= \frac{B[(i+1)^N - 1]^{-1}}{C.i (i+1)^N}$$

If rate of interest is 8% per annum, $i = 0.08$

For the project justification, the benefit cost ratio should be greater than 1.

Ex:

Drainage Engineering

Drainage of irrigated lands can be defined as the removal of excess water and salts from adequately irrigated agricultural lands.

The adequate drainage involves the lowering of the water table.

Drainage systems can be of the following benefits:

- (i) improve soil structure and increase the soil productivity
 - (ii) facilitate early planting
 - (iii) increase the depth of root zone which leads to increasing the available soil moisture and plant food
 - (iv) increases soil ventilation
 - (v) increases water infiltration into the ground then decreasing soil erosion on the surface.
 - (vi) create favourable conditions for growth of soil bacteria
 - (vii) leach excess salts from soil
 - (viii) maintain favourable soil temperature
 - (ix) improve sanitary and health conditions for the residents of the area.
-

The water table can be lowered by eliminating or controlling sources of excess water by natural or artificial drainage systems.

A natural drainage system can be properly maintained at low costs and is a feasible method of protecting irrigated lands.

Artificial drainage also aims at lowering the water table and is accomplished by any of the following methods:

(i) **Open ditch drains:** (or open drains) are suitable and very often economical for surface and subsurface drainage. They permit easy entry of surface flow into the drains.

The longitudinal slope of the drain should be **as large as possible** and is decided on the basis of non-scouring velocities. The bed slope ranges from 0.0005 to 0.0015. Depths of about 1.5 to 3.5m are generally adopted for open drains. The side slopes depend largely on the type of embankment soil and may vary from 1/2H:1V (in very stiff and compact clays) to 3H:1V (in loose sandy formations).

Open drains have the advantages of:

- (a) low initial cost
- (b) simple construction
- (c) large capacity to handle surface runoff caused by precipitation.

*There are *disadvantages* too. Besides **the cost of land** which the open drains occupy and **the need of constructing bridges across them**, open drains cause:

- (a) difficulty in farming operations
- (b) constant maintenance problems resulting from silt accumulation due to rapid weed growth in them.

(ii) **Subsurface drains:** (or underdrainage) involves the creation of *permanent drainage system* consisting of buried pipes (or channels) which remain out of sight and, therefore, do not interfere with the farming operations.

The buried drainage system can remove excess water without occupying the land area. Therefore, there is no loss of farming area. Besides, there is no weed growth

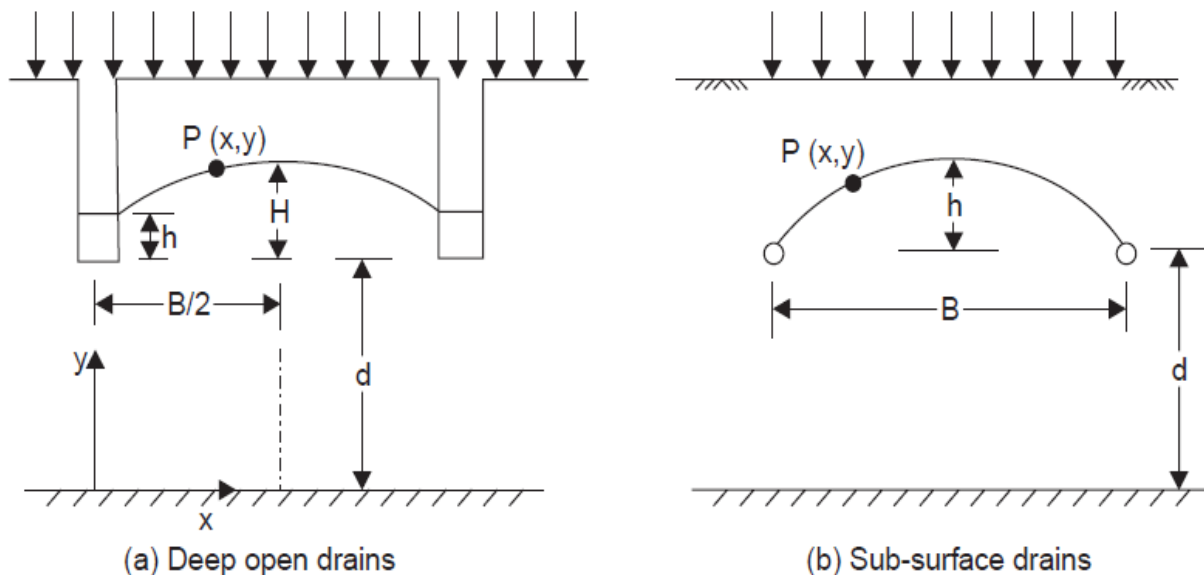
and no accumulation of rubbish and, therefore, the underdrainage system can remain effective for long periods with little or no need for maintenance.

In some situations, however, siltation and blockage may require costly and troublesome maintenance or even complete replacement.

(iii) **Drainage wells:** They offer a very effective method of draining an irrigated land. The soil permeability and economic considerations decide the feasibility of well drainage.

Drainage wells pump water from wells drilled or already existing in the area to be drained. Design of a drainage well system will be based on established principles of well hydraulics **which is out of the scope for this semester.**

The figure below shows a line sketch of two types of drains:



Where d is the depth of the impermeable layer underlying the drain, and B is the spacing between drains in metres.

$$B^2 = \frac{4k}{r_a} [(H + d)^2 - (h + d)^2]$$

The above equation is called **Hooghoudt's equation**, and it can be used for either **open ditch drains** and **subsurface drains**. The value of B is generally between 15 and 45 m (in case of subsurface drains)

If q_d is the discharge per unit length of drain that enters the drain from two sides of the drain, then

$$q_d = r_a * B$$

$$q_d = \frac{4k}{B} [(H + d)^2 - (h + d)^2]$$

In practice, the drain is considered empty (i.e., $h = 0$). Then:

$$q_d = \frac{4k[(H+d)^2 - (d)^2]}{B} \quad \text{Or} \quad q_d = \frac{4kH}{B} (H + 2d) , \text{ and } B^2 = \frac{4kH}{r_a} (H + 2d)$$

Thus, knowing q_d , the spacing B of the drains can be determined.

Where: q_d is the design drain discharge,

r_a is the average rainfall intensity in metres.

k is the coefficient of permeability of soil.

H is the max. water table level between drains (for open surface drains).

h is the level of drain water in drains (for open surface drains), and the max. water table level between drains (for subsurface drains). As shown in figure above.

Ex: Determine the depth of drains below the ground for the following data:

Root zone depth = 1.5m

Capillary rise in soil = 0.3m

Coefficient of permeability of soil = 1.5×10^{-4} m/s

Drainage capacity = $0.11 \text{ m}^3/\text{s}/\text{km}^2$

Spacing of drains = 200m

Depth of impervious stratum below ground = 10.0m

Ans:

$$q_d = 0.11 \times 200 \times 1/10^6 \text{ m}^3/\text{s}/\text{m}$$

$$H + d = 10 - (1.5 + 0.3)\text{m}$$

$$= 8.2\text{m}$$

$$B = 200\text{m}, k = 1.5 \times 10^{-4} \text{ m/s}$$

$$q_d = \frac{4k[(H+d)^2 - (d)^2]}{B} \implies \frac{0.11 \times 200}{10^6} = \frac{4 \times 1.5 \times 10^{-4} [(8.2)^2 - (d)^2]}{200}$$

$$d = 7.74\text{m}$$

Hence, the drains should be located at $10 - 7.74 = 2.26\text{m}$ below the ground.

H.W: Closed drains at a spacing of 16m are located 2m below the ground surface and the position of the water table is 1.7m below the ground surface. Find the discharge carried by a drain if the coefficient of permeability of the soil is 2×10^{-2} cm/s and the depth of the pervious stratum is 8m.