Design Discharge → Full Supply Discharge.

$\dfrac{3.5}{\text{Distributary actually opened / Crop period}}$

Now, Time Factor = 0·7 (given);

∴ Full Supply Discharge (based on average demand or duty)

Actual operating hrs of distributary → 3.5 cumecs. (x)

per hr → $\dfrac{3.5}{x}$

In a crop period → $\dfrac{3.5}{x}$ × Cropping

$= \dfrac{3.5}{0.7} = 5.0$ cumec

Assuming the peak demand discharge to be 25% more than the average,

Design Duty or Design Discharge → $\dfrac{3.5}{\text{Time factor}}$

Nominal Discharge Nominal Duty = x × Time Factor

FSD = Nominal Discharge / Time Factor

Full Supply Discharge on peak demand = 5×1·25 cumecs

= 6·25 cumecs.

(a) Hence the F.S.Q at the head of the main canal (assuming negligible seepage losses from the head of the main canals to the fields) = <u>6.25 cumecs</u> (Ans)

(b) To find out the gross storage capacity of the reservoir, let us work out the volume of water required by various crops as follows:

1. Water required by Sugarcane = $0.5 \times 280 \times 24 \times 3600 \; m^3$.
2. Water required by Overlap Sugarcane = $0.11 \times 100 \times 24 \times 3600 \; m^3$.
3. Water required by Jowar = $3.0 \times 120 \times 24 \times 3600 \; m^3$.
4. Water required by Bajra = $2.0 \times 120 \times 24 \times 3600 \; m^3$.
5. Water required by vegetables = $0.5 \times 120 \times 24 \times 3600 \; m^3$.

Total Water required = $\Sigma\, 24 \times 60 \times 60 \, [(0.5 \times 280) + (0.11 \times 100) + (3 \times 120) + (2 \times 120) + (0.5 \times 120)]$

$= 24 \times 60 \times 60 \, [140 + 11 + 660] \; m^3$

$= 24 \times 60 \times 60 \times 811 \; m^3$

$= 70.07 \times 10^6 \; m^3 = 70.07 \; Mm^3$

Assume reservoir losses due to absorption & evaporation as 10% of 70·07, i.e., = 7·01 $M.m^3$

To allow for late & irregular monsoon, let us assume a carry over storage, equal to 5% of 70·07 Mm^3 i.e., 3·35 Mm^3.

∴ The live storage of reservoir = (70·07 + 7·01 + 3·35)

= 80·43 $M.m^3$.

Now Gross Storage = Live storage + Dead Storage.

Assuming dead storage at 10% of gross storage, we get

$= G = 80.43 + 0.1 \times$ Gross Storage.

or $G = 80.43 + 0.1G$

or $0.9G = 80.43$

or $G = \dfrac{80.43}{0.9} = 89.4 \; M.m^3$.

Hence, the Gross Storage of the Reservoir is <u>89·4 $M.m^3$</u> (Ans)

LOSSES OF WATER IN CANALS.

- During the passage of water from the main canal to the outlet at the head of the watercourse, water may be lost either by evaporation from the surface or by seepage through the peripheries of the channel.
- These losses are sometimes very high, of the order of 25 – 50 % of the water diverted into the main canal.
- In determining the designed channel capacity, a provision for these water losses must be made. The provision for the water lost in the watercourses & in the fields is however, already made in the outlet discharge factors and hence, no extra provision is made on that account.

(1) EVAPORATION :-

- Evaporation losses are generally of the order of 2 to 3 % of total losses.
- In summer season, these losses may be more but seldom exceed about 7 % of the total water diverted into the main canal.

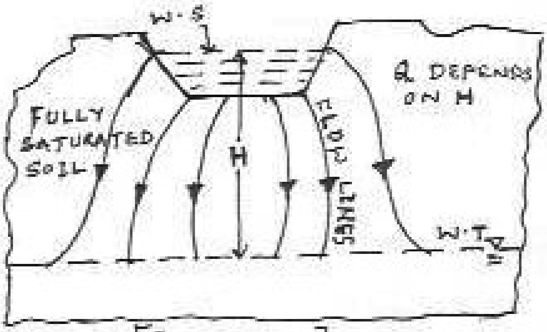
(2) SEEPAGE :-

There may be two different conditions of seepage, i.e.,

- (i) Percolation
- (ii) Absorption.

(i) $\boxed{\text{Percolation}}$:-

- In percolation, there exist a zone of continuous saturation from the canal to the watertable & a direct flow is established.
- Almost all the water lost from the canal, joins the groundwater reservoir.



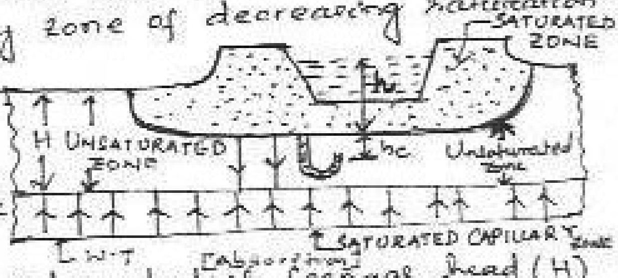
[Percolation]

- The loss of water depends upon the difference of top water surface level of the channel and level of the watertable.
  H → Seepage Head

  $Q \propto H$

(ii) $\boxed{\text{Absorption}}$:-

- In absorption, a small saturated soil zone exists round the canal section, and is surrounded by zone of decreasing saturation.
- A certain zone just above the water-table is saturated by capillarity. Thus there exist an unsaturated soil zone between the two saturated zones.



[Absorption]

- In this case, the rate of loss is independent of seepage head (H) but depends only upon the water head $[(h) + (h_c)]$

  Water Head, h = distance between water surface level of canal & bottom of the saturated zone.

  \+

  Capillary Head, h_c.

  Q Depends on $(h + h_c)$

Flow Lines :- or **Streamlines** – The path along which the individual particles of water seep through the soil is called Streamlines or Flow Lines.

The seepage losses depend upon the following factors,

(i) Type of Seepage i.e., whether 'percolation' or 'absorption'.

(ii) Soil permeability.

(iii) Condition of the canal; the seepage through a silted canal is less than that from a new canal.

(iv) Amount of silt carried by the canal; the more the silt, lesser are the losses.

(v) Velocity of canal water; the more the velocity, the lesser will be the losses.

(vi) Cross-section of the canal & its wetted perimeter.

EMPIRICAL FORMULAS FOR CHANNEL LOSSES

(a) $\boxed{\Delta Q = \frac{1}{200}(B+D)^{2/3}}$; where ΔQ = channel losses in cumecs per km length of channel;

D = Depth of water in channel (m); B = Bed width of the channels (m).

This formula is generally used in U.P.

(b) $\boxed{\Delta Q = 1.9\, Q^{1/6}}$, where ΔQ = Losses in cumecs per million sq.m. of wetted perimeter.

Q = Discharge in cumecs.

The above formula is commonly used in Punjab.

ABSORPTION	PERCOLATION
- When the underground water table is at a considerable depth, the water entering the soil is unable to join the saturated zone & wets the sub-soil locally immediately below the canal bed.	- When the underground water table is nearer to the natural surface, the water which has entered the sub-soil may join the saturated zone or underground reservoir to maintain a continuous direct flow
- The soil layer which is in immediate contact with the channel section is completely saturated due to the absorbed water. It forms a bulb of saturated soil below the channel.	- The portion b/w the two extreme flow lines is completely saturated from the canal section to the underground water table
- The soil layer below the saturated bulb is not fully saturated. Thus the extent of saturation goes on decreasing from the ground level below in the soil with depth. There is now exist a zone of unsaturation b/w the underground saturated zone & saturated bulb.	- Here water directly flows from the canal to the underground reservoir ~~through~~ the soil pores under pressure.
- Thus there is no chance of continued & constant flow from the canal to the groundwater reservoir.	- The pressure which is responsible for this flow is due to the diff. in level b/w the underground reservoir & the water level in the canal. It can be measured in units of water depth.
- It is observed that loss of due to absorption is more when canal is in cutting reach.	

Under what condition unstability occurs?

DESIGN OF STABLE CHANNELS IN INDIA

Most of the canal bed (unlined) are made up of sands, silts & fine silts which may interfere canal flow & make the channel unstable

Why Kennedy and Lacey?

Average Shear Stress (τ_0); $\tau_0 = \gamma_w R S$ < Critical Shear Stress (τ_c)
acting on the boundary of an alluvial channel

Pull force of water on the wetted area is known as tractive force.

$\tau_0 < \tau_c$ ⇒ the channel shape remains unchanged (rigid boundary).

The resistance equations, such as those given by Chezy's formula & Manning's formula, remain well applicable to such channels.

Chezy's formula: $V = C\sqrt{RS}$; (i)

Manning's formula: $V = \frac{1}{n} R^{2/3} S^{1/2}$ (ii)

However, as soon as the sediment movement starts, undulations develop on the bed, which increases the boundary resistance of the channel. Also energy is spent to move the grains/sediments in alluvial canals. The suspended load, carried due to the turbulence in flow, further affects the resistance of alluvial streams.

All these factors renders the evaluation of resistance of alluvial stream to be a very complex problem, and the complexity further increases if one includes the effects of the channel shape, non-uniformity of sediment size, discharge variation, & other such factors.

None of the resistance equations developed so far, takes all these factors into account. The direct accurate mathematical solution to the design of channels in alluvial soils is, therefore, not an easy job; and hence in India, alluvial channels are designed on the basis of hypothetical theories given by Kennedy & Lacey. These theories are based on experiments & experience gained on the existing channels over the past many years.

- Water moving with a given velocity and a certain depth can carry in suspension, only a certain amount of silt of a certain nature.

SCOURING :- If water of a given velocity and depth is not fully charged with silt (that it can carry in suspension) it will scour the bed & sides of the channel, till it is fully charge with silt.

- Scouring lowers the full supply level & causes loss of command.
- It may also cause breaching of canal banks & failure of foundations of irrigation structures.

SILTING :- If the velocity is less, the silt which was formerly carried in suspension is likely to be dropped, hence causing silting of the channel.

- Silting interferes with the proper working of a channel, as the channel gets reduced by siltation, thereby reducing the discharging capacity of the channel.

"While thinking to design a properly functioning channel, one must think to design such a channel in which neither silting nor scouring takes place. Such channels are known as stable channels or regime channels."

Silting & Scouring are the phenomenon very inst. to the unlined canals.

IS: 5968-1987 Indian Standard (Guide for planning & layout of canal system for irrigation) (First Revision)

IS: 7112: 2002 Indian Standard (Criteria for design of cross-section for unlined canals in alluvial soil) (First Revision)

REGIME CHANNELS [Design of unlined canals in alluvial soil by Kennedy's Theory]

A Channel is said to be in a state of 'Regime', if the flow is such that 'Silting & Scouring' need no special attention.

"The basis for designing such an ideal, non-silting, non-scouring channel is that, whatever silt has entered the channel at its head is kept in suspension, so that it doesnot settle down and deposit at any point of the channel. Moreover, the velocity of the water should be such that it doesnot produce local silt by erosion of channel bed & slopes".

- In artificial channels, such a state can be obtained by properly designing the channels.

KENNEDY'S THEORY

[There will be friction b/w the flowing water and bed of the channel while water flows it has to overcome this friction and this results in the formation of the eddies.]

Concept. (1895) E.E. R.G. Kennedy.

He concluded that silt supporting power in a channel cross-section was mainly dependent upon the generation of eddies, rising to the channel surface.

These eddies are generated due to friction of flowing water with the channel surface. The vertical component of these eddies try to move the sediment up, while the weight of the sediment tries to bring it down, thus keeping the sediment in suspension. So if the velocity is sufficient to generate these eddies, so as to keep the sediment just in suspension, silting will be avoided. (He assumed the horizontal components varying from the edes, cannot support silt)

Based on the above concept, he defined the critical velocity (V_0) in a channel as the mean velocity (across the section) which will just keep the channel free from silting or scouring, and related it to the depth of flow by the equation,

$$V_0 = C_1 \cdot y^{C_2}$$

Silt Supporting Power $\propto$ bed width of the canal (B) (it doesnot depend upon its wetted perimeter)

where C_1 & C_2 are constants depending on silt charges.

$C_1 = 0.55$; $C_2 = 0.64$.

$$\therefore \boxed{V_0 = 0.55\, y^{0.64}}$$

V_0 = Critical Velocity in channel (ms^{-1})

y = water depth in channel (m)

This formula was worked out especially for the Upper Bari Doab canal System, it couldnot have been applicable into other canal systems due to variation in type of soil (or silt) at various canal sites.

Realising this lacuna, Kennedy later introduced a factor 'm' in this equation, to account for the type of soil through which the canal was to pass.

This factor, which was dependent upon the silt grade, was named as critical velocity ratio (C.V.R) and denoted by m,

$$\boxed{V_0 = 0.55\, m y^{0.64}}$$

m = Critical Velocity Ratio.

For sands coarser than the standard, $1.0 < m < 1.2$

For sands finer than the standard, $0.7 < m < 1.0$.

The value of 'm' is assumed as higher in head reaches of canal than in its tail reaches.

$V_0 \rightarrow$ The critical velocity which doesnot produce silting or scouring is called critical velocity. $m = \frac{V}{V_0}$

DESIGN PROCEDURE

1 Determine the critical velocity V_0 by the above equation (1) by assuming a trial depth.

2 Determine area by dividing discharge by velocity.

3 Determine channel dimensions. (R & P)

4 Finally compute the actual mean velocity (V) that will prevail in the channel of this cross-section, by using kutter's or Manning's formula etc.

5. If $V_0 = V$; then the assumed depth is all right, Otherwise change it and repeat the procedure., till $V_0 = V$.

Kutter's formula,

$$V = \left[\frac{\frac{1}{n} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{n}{\sqrt{R}}} \right] \sqrt{RS}$$

Manning's Formula,

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

V = Velocity of flow (ms^{-1})
R = Hydraulic mean depth (m)
S = Bed slope of the channel
n = Rugosity Coefficient.

Chezy's Formula,

$$V = C\sqrt{RS}$$

C = a constant depending upon the shape & surface of the channel.

NOTE :- The actual mean velocity (V) generated in the channel can be computed by any of these 3 resistance equations, but generally kutter's equation is used with kennady's theory.

Problem

Design an irrigation channel to carry 50 cumecs of discharge. The channel is to be laid at a slope of 1 in 4000. The critical velocity ratio for the soil is 1.1. Use Kutter's rugosity coefficient as 0.023.

Solution. $Q = 50$ cumecs, $S = \frac{1}{4000}$

$m = 1.1$, $n = 0.023$

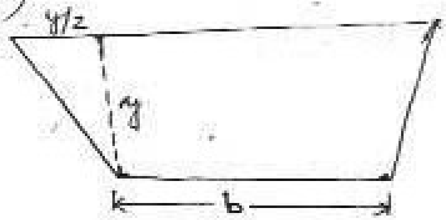
1st Trial. $V_0 = 0.55\, m y^{0.64}$

Assume a depth = 2m

$\therefore V_0 = 0.55 \times 1.1 \times (2)^{0.64} = 0.605 \times 1.558 = 0.942\ ms^{-1}.$

$$A = \frac{Q}{V_0} = \frac{50}{0.942} = 53.1\ m^2.$$

Assume side slopes as $\frac{1}{2} : 1$ $\left(\frac{1}{2}H : 1V\right)$



Now, $A = \frac{1}{2} \times y \left(b + b + y\right)$

$\Rightarrow A = y\left(b + y/2\right)$

or $53.1 = 2\left(b + 1\right)$

$\Rightarrow b = 25.55$ m.

and $P = b + 2\sqrt{y^2 + \frac{y^2}{4}}$

or $P = b + 2\sqrt{\left(1 + \frac{1}{4}\right)}\, y$

$\Rightarrow P = b + 2\frac{\sqrt{5}}{2} y = b + \sqrt{5} y = 25.55 + \sqrt{5} \times 2 = 30.03$ m

$R = \frac{A}{P} = \frac{53.1}{30.03} = 1.77$ m.

Using Kutter's formula to find mean velocity V.

$$V = \left[\frac{\frac{1}{n} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{n}{\sqrt{R}}}\right]\sqrt{RS}$$

$m = \frac{V}{V_0} = 1.1$

$V = 1.1 V_0$

$$V = \left[\frac{\frac{1}{0.023} + \left(23 + \frac{0.00155}{1/4000}\right)}{1 + \left(23 + \frac{0.00155}{1/4000}\right)\frac{0.023}{\sqrt{1.77}}}\right]\sqrt{1.77 \times \frac{1}{4000}}$$

or $V \doteq 1.016\ ms^{-1} > 0.942\ ms^{-1}\ (V_0)$ or $V > V_0$

In order to increase the critical velocity (V_0), we have to increase the depth. So increase the depth

* Use 3m depth (2nd Trial):

$V_0 = 0.605 \times (3)^{0.64} = 0.605 \times 2.02 = 1.22\ ms^{-1}$.

$A = \frac{Q}{V_0} = \frac{50}{1.22} = 40.8\ m^2$.

$A = y\left(b + y/2\right)$

$\Rightarrow 40.8 = 3\left(b + 1.5\right)$

or $b = 12.1$m

$P = b + \sqrt{5}\, y = 12.1 + \sqrt{5} \times 3 = 12.1 + 6.72 = 18.82$m

$R = \frac{A}{P} = \frac{40.8}{18.82} = 2.17$

$R = 2.17$m

Using Kutter's formula to find mean velocity V (for 2nd Trial)

$$V = \left[\frac{\frac{1}{0.023} + \left(23 + \frac{0.00155}{1/4000}\right)}{1 + \left(23 + \frac{0.00155}{1/4000}\right)\frac{0.023}{\sqrt{2.17}}}\right]\sqrt{2.17 \times \frac{1}{4000}}$$

or $V = 1.16\ ms^{-1} < 1.22\ ms^{-1}\ (V_0)$ or $V < V_0$

So reduce the depth ($\because V_0 \propto y$)

3rd Trial Use 2.5m depth :

$V_0 = 0.605 \times (2.5)^{0.64} = 0.605 \times 1.797 = 1.087\ ms^{-1}$

$A = \frac{Q}{V_0} = \frac{50}{1.087} = 46\ m^2$

$A = y(b + y/2) = 2.5(b + \frac{1}{2} \times 2.5) = 46$

or $b = 17.15\ m$

$P = b + \sqrt{5}\, y = 17.15 + \sqrt{5} \times 2.5 = 17.15 + 5.58 = 22.73\ m$

$R = \frac{A}{P} = \frac{46}{22.73} = 2.02$ or $R = 2.02\ m$

Using Kutter's formula to find mean velocity V (for 3rd Trial)

$$V = \left[\frac{\frac{1}{0.023} + \left(23 + \frac{0.00155}{1/4000}\right)}{1 + \left(23 + \frac{0.00155}{1/4000}\right)\frac{0.023}{\sqrt{2.02}}}\right] = 1.1\ ms^{-1}$$

$V = 1.1\ ms^{-1} > 1.087\ ms^{-1}\ (V_0)$; or $V > V_0$

So increase the depth

4th Trial Use 2.7m depth

$V_0 = 0.605 \times (2.7)^{0.64} = 0.605 \times 1.89 = 1.147\ ms^{-1}$

$A = \frac{Q}{V_0} = \frac{50}{1.147} = 43.5\ m^2$

$43.5 = y(b + y/2) = 2.7(b + \frac{1}{2} \times 2.7)$

or $43.5 = 2.7(b + \frac{1}{2} \times 2.7)$ or $b = 14.76\ m$

$P = b + \sqrt{5}\, y = 14.76 + (\sqrt{5} \times 2.7) = 20.8\ m$

$R = \frac{A}{P} = \frac{43.5}{20.8} = 2.091$ or $R = 2.091\ m$

or $\sqrt{R} = 1.446\ m$

$$V = \left[\frac{\frac{1}{0.023} + \left(23 + \frac{0.00155}{1/4000}\right)}{1 + \left(23 + \frac{0.00155}{1/4000}\right)\frac{0.023}{1.446}}\right]\sqrt{2.09 \times \frac{1}{4000}}$$

or $V = 1.134\ ms^{-1}$

$(1.134)\ V \approx V_0\ (1.147)$

Hence, use the depth equal to 2.7m & base width 14.76m with side slopes 1/2 : 1 of a trapezoidal section

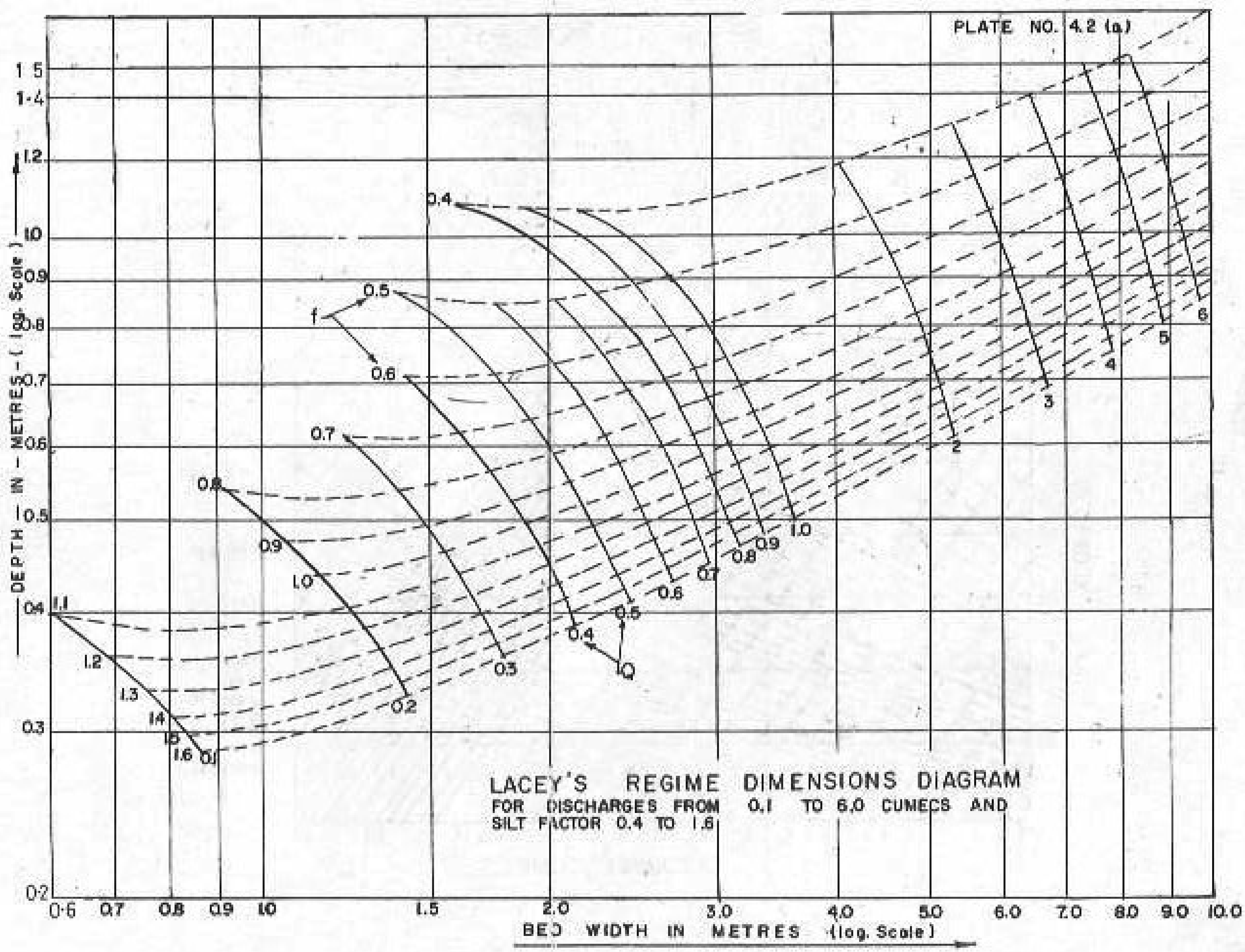
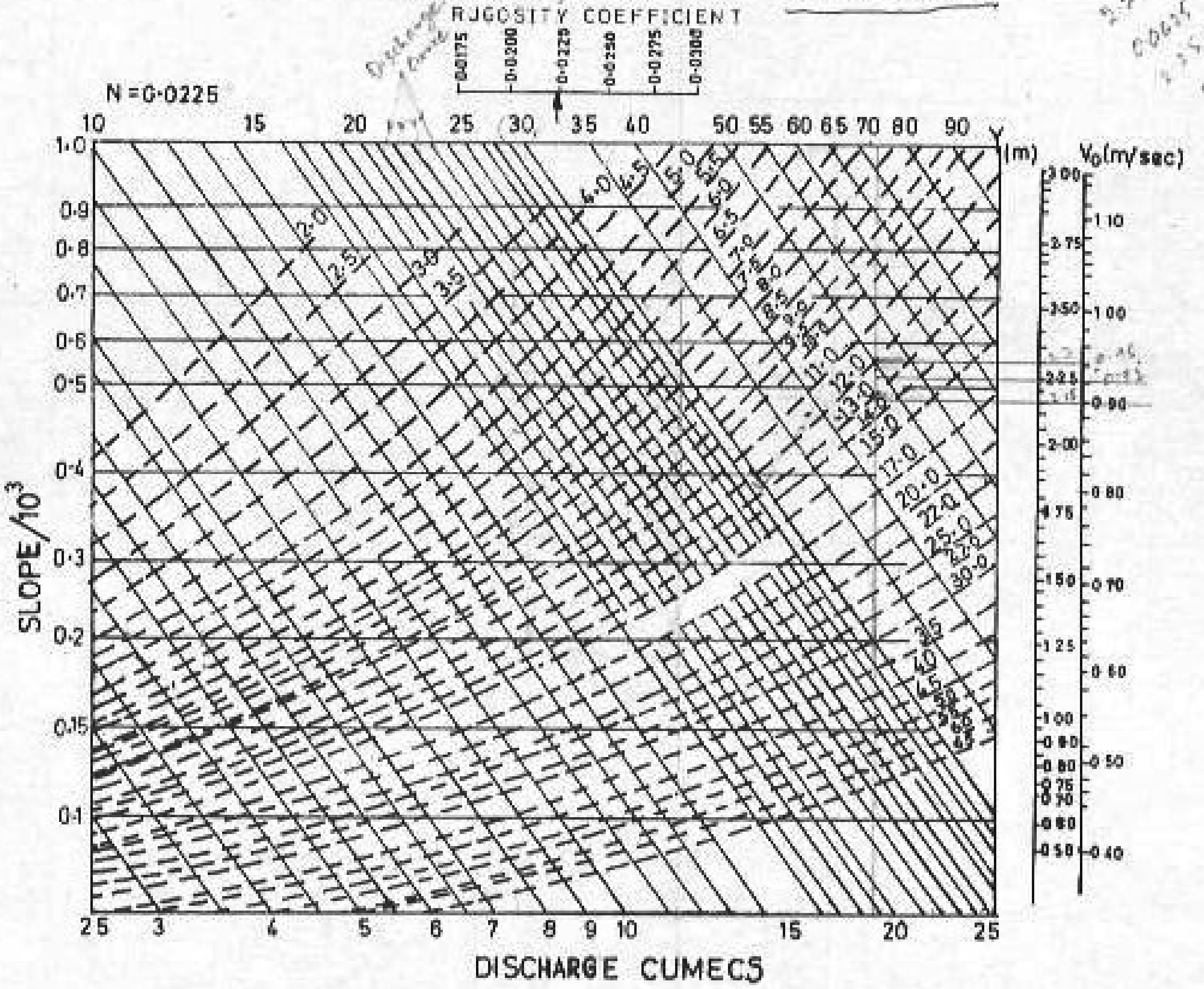


PLATE NO. 4.2 (a)
LACEY'S REGIME DIMENSIONS DIAGRAM
FOR DISCHARGES FROM 0.1 TO 6.0 CUMECS AND
SILT FACTOR 0.4 TO 1.6
DEPTH – IN – METRES – (log. Scale)
BED WIDTH IN METRES (log. Scale)
f
Q
0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 1.2 1.4 1.5
0.6 0.7 0.8 0.9 1.0 1.5 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0

GARRENT'S DIAGRAM

PLATE NO:4·1(c)



Use of Garret's Diagrams for Applying Kennedy's Theory.

- A lot of mathematical calculations are required in designing irrigation channels by the use of Kennedy's method.
- To save mathematical calculations, graphical solution of Kennedy's and Kutter's equations, was evolved by Garret.

The procedure adopted for design of irrigation channels using Garret's diagrams is explained below:

1) The discharge (Q), bed slope (S_0), rugosity coefficient (n), value of CVR (m) are given for the channel to be designed.

2) Find out the point of intersection of the given slope line and discharge curve. At this point of intersection, draw a vertical line intersecting the various bed width curves.

3) For different bed widths (B), the corresponding values of water depth (y) and critical velocity (V_0) can be read out on the RH ordinate. Each such pair of bed width (B) & depth (y) will satisfy Kutter's equation, and is capable of carrying the required discharge at the given S_0 & n.
Choose one such pair & determine actual velocity of flow (V)

4) Determine the critical velocity Ratio (V/V_0) taking V as calculated & V_0 as read.

5) If the value of C.V.R is not same as given in question, repeat the procedure with other pairs of B & y.

The diagrams have been drawn for a trapezoidal channel with side slopes as $\frac{1}{2}H : 1V$ ($\frac{1}{2} : 1$) on the assumption that irrigation channels adopt approximately this slope or shape, even though they were constructed on diff. side slopes.

Problem Design an irrigation channel to carry 30 cumec of discharge. The channel is to be laid at a slope of 1 in 5000. The critical velocity ratio for the soil is 1.1. Use Kutter's rugosity coefficient as 0.0225.

Solution: Q = 30 cumecs $\quad S = \frac{1}{5000} = 0.2 \times 10^{-3} = 0.2\,m/Km = 0.2/10^3$

$m = 1.1 \quad n = 0.0225$

$\frac{V}{V_0} = m = 1.1$ or. $V = 1.1 V_0$

Using Plate 4.1(c), find out the pt. of intersection of the given slope line (0.2 m/Km) and given discharge curve (30 cumecs). Draw a Vertical line through this point. Choose a approximate bed width = 12 m, as first approximation. Calculate m as shown in the last column of the table.

Sl. No.	B (m)	y (m)	V_0 (m/s)	$A = \frac{y}{2}(2B+y)$ (m²)	$V = \frac{Q}{A}$	$\frac{V}{V_0} = m$
1	12.0	2.3	0.95	$\frac{2.3}{2}(24+2.3) = 30.25$	0.99	1.04
2	12.5	2.25	0.92	30.66	0.98	1.07
3.	13.0	2.15	0.90	30.26	0.99	1.1

The first approximation of B = 12.0; yields the value of m = 1.04, but m = 1.1 (as given data). Inorder to increase m, we have to reduce V_0 & ultimately reduce the depth & increase the bed width (12.5m)

Hence choose the final values of B = 13.0 m, y = 2.15 m for the channel of bed slope 1 in 5000 & side slopes $\frac{1}{2}H : 1V$.

LACEY'S THEORY (1939)

He found many drawbacks in Kennedy's Theory (1895) & he put forward his new theory. The essential points which he argued, and the design procedure which he suggested, is briefly discussed here.

Lacey's regime channels.
Lacey came out with the statement that even a channel showing no silting no scouring may actually not be in regime.
He differentiated between three regime conditions:
(i) True Regime (ii) Initial Regime (iii) Final Regime.

True Regime

- For this condition to be satisfied, the silt load entering the channel must be carried through by the channel section.
- Moreover, there can be only one channel section & one bed slope at which a channel carrying a given discharge & a particular quantum & type of soil, would be in regime.
- Hence, an artificially constructed channel having a certain fixed section & a certain fixed slope can behave in regime only if the following conditions are satisfied:

(i) Discharge is constant;
(ii) Flow is uniform;
(iii) Silt charge is constant; i.e. amount of silt is constant;
(iv) Silt grade is constant; type & size of silt is always same.
(v) Channel is flowing through a material which can be scoured as easily as it can be deposited (incoherent alluvium), & is of the same grade as is transported.

[But in practice, all these conditions can never be satisfied. And, therefore, artificial channels can never be in 'true regime'; they can either be in initial regime or final regime.]

Initial Regime

[Sides of the channel that has formed its cross-section but not formed its longitudinal slope. Such channel appears to be in regime outwardly as there may be no scouring or no silting.]

- When only the bed slope of a channel varies due to dropping of silt, and its cross-section or wetted perimeter remains unaffected, even then the channel can exhibit 'no silting no scouring' properties, called Initial Regime.

→ **How Pseudo non-silting equilibrium is established in Initial Regime?**

When water flows through an excavated channel with somewhat narrower dimensions & defective slopes, the silt carried by the water may get dropped in the upper reaches, thereby increasing the channel bed slope. Consequently, the velocity is increased & a non-silting equilibrium is established, called 'Initial Regime'.

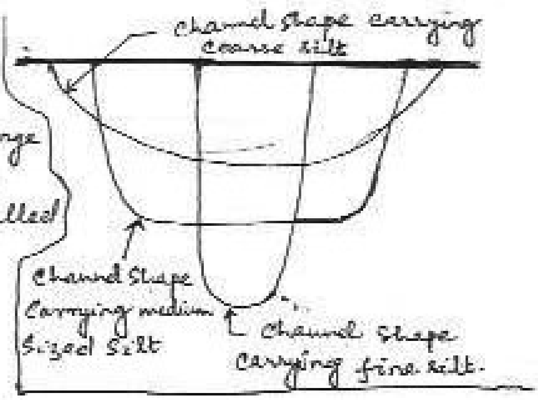
→ **How Pseudo no-scouring is established in Initial Regime?**

Sides of such channels are subjected to a lateral restraint & could have scoured if the bank soil would have been a true alluvium. But in practice, they may either be grassed or be of clayey soil, and therefore, they may not get eroded at all.

- They have achieved only a working stability due to the rigidity of their banks. Their slopes & velocities are higher & cross-sections narrower than what would have been if the sides were not rigid. Such channels are defined as channels in initial regime, & regime theory is not applicable to them, as they are not the channels in alluvium.

Final Regime

But,
- If there is no resistance from the sides, and all the variables such as perimeter, depth, slope etc. are equally free to vary & finally get adjusted according to discharge & silt grade, then the channel is said to have achieved permanent stability, called "Final Regime".



- Regime theory is applicable to such channels only, and not to all regime channels, as was envisaged by Kennedy.
- Channel in final regime has a tendency to assume a semi-elliptical section.
  - (i) The coarser the silt, the flatter is the semi-ellipse, i.e. greater is the width of the water surface.
  - (ii) The finer the silt, the more nearly the section attains a semi-circle.

Points Argued by Lacey on Kennedy's Theory.

1) Kennedy had neglected the eddies that are generated on the sides of the channel, by presuming that such eddies has horizontal movement for greater part, and, therefore, donot have sediment supporting power. Lacey thus, argued that the silt supporting power of a channel is proportional to the wetted perimeter of the channel and not to its width, as was presumed by Kennedy.

2) Lacey argued that the grain size of the material forming the channel is an important factor. He, therefore, introduced a term called silt factor (f) in his equations, and connected it to the average particle size.

Design Procedure for Lacey's Theory.

(1) Calculate the velocity from equation

$$\boxed{V = \left[\frac{Qf^2}{140}\right]^{1/6} ms^{-1}}\ ; \quad \text{where}$$

Q is in cumec
V is in ms^{-1}
f = silt factor
or $f = 1.76\sqrt{d_{mm}}$
where d_{mm} = Average particle size in mm.

(2) Work out the hydraulic mean depth (R) from the equation

$$R = \frac{5}{2}\left(\frac{V^2}{f}\right)$$

where V is in ms^{-1}
R is in m.

(3) Compute area of channel section, $A = \dfrac{Q}{V}$.

(4) Compute wetted Perimeter, $P = 4.75\sqrt{Q}$.
where P is in m; Q is in cumecs.

(5) Knowing these values, the channel section is known; and finally the bed slope S is determined by the equation:

$$S = \left[\frac{f^{5/3}}{3340\,Q^{1/6}}\right]$$ where f is the silt factor, Q is the discharge (in cumec).

<u>Example</u>

Design a regime channel for a discharge of 50 cumecs & silt factor 1.1, using Lacey's Theory.

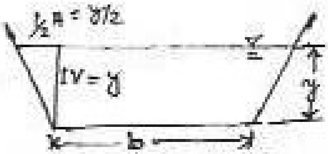
<u>Solution</u> :- $Q = 50$ cumecs ; $f = 1.1$

$$V = \left[\frac{Qf^2}{140}\right]^{1/6} = \left[\frac{50\times(1.1)^2}{140}\right]^{1/6} = 0.869$$

$$A = \frac{Q}{V} = \frac{50}{0.869} = 56.3\ m^2.$$

$$R = \frac{5}{2}\left(\frac{V^2}{f}\right) = \frac{5}{2}\left[\frac{(0.869)^2}{1.1}\right] = 1.675\ m$$

$$P = 4.75\sqrt{Q} = 4.75\cdot\sqrt{50} = 33.56\ m.$$



for a trapezoidal channel with $\frac{1}{2}H : 1V$ slopes,

Wetted Perimeter, $P = b + 2\times\frac{\sqrt{5}}{2}y$

$$\sqrt{(1)^2+(1/2)^2} = \sqrt{1+\frac{1}{4}} = \frac{\sqrt{5}}{2}$$

$$P = b+\sqrt{5}\,y \;;\quad 33.56 = b+\sqrt{5}\,y \quad\text{---(1)}$$

$$\text{Area} = \frac{1}{2}\times y\,(b+b+y)$$

or $A = \frac{1}{2}\,y\,(2b+y)$

or $A = y\left(b+\frac{y}{2}\right)$

$$56.3 = y\left(b+\frac{y}{2}\right)\quad\text{---(2)}$$

from eq^n (1) we get ; $b = 33.56 - 2.24y$

Putting this value of b in eq^n (2) we get;

$$56.3 = [33.56 - 2.24y]\,y + \frac{y^2}{2}.$$

or $56.3 = 33.56y - 2.24y^2 + 0.5y^2$

or $56.3 = 33.56y - 1.74y^2$

or $1.74y^2 - 33.56y + 56.3 = 0.$

or $y^2 - 19.3y + 32.4 = 0$

or $y = \dfrac{-b\pm\sqrt{b^2-4ac}}{2a} = \dfrac{19.3\pm\sqrt{372-129.6}}{2}$

or $y = \dfrac{19.3\pm\sqrt{242.4}}{2} = \dfrac{19.3\pm 15.6}{2}$

Neglecting unfeasible +ve sign we get

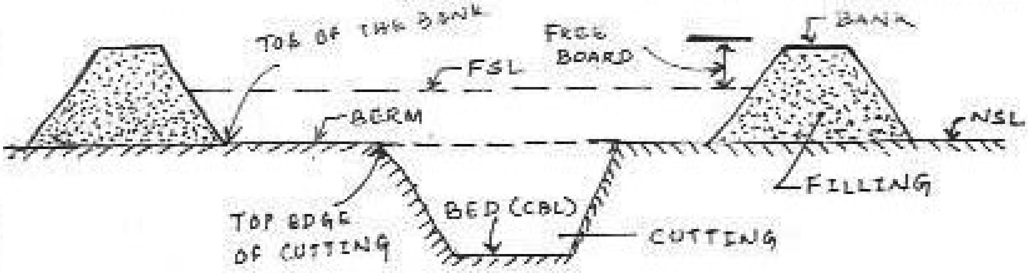
$$y = \frac{19.3-15.6}{2} = 1.65\ m \quad\text{or}\quad y = 1.65\ m \quad \underline{\text{Ans.}}$$

$b = 33.56 - 2.24y = 33.56 - 2.24\times1.65 = 29.77$ or $b = 29.77\ m$ <u>Ans.</u>

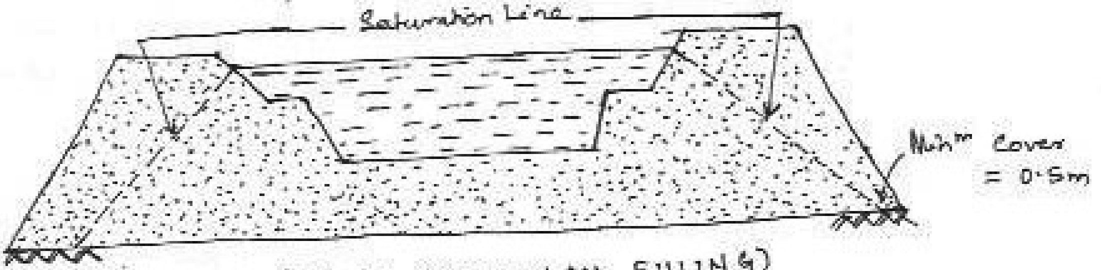
$S = \left[f^{5/3}/3340\,Q^{1/6}\right] = \left[\frac{(1.1)^{5/3}}{3340(50)^{1/6}}\right] = \frac{1}{\ldots}$; Use a bed slope of 1 in 5420 <u>Ans.</u>

CROSS-SECTION OF AN IRRIGATION CANAL

This section is 'partly in cutting and partly in filling', and aims at balancing the quantity of earthwork in 'excavation' with that in 'filling'.



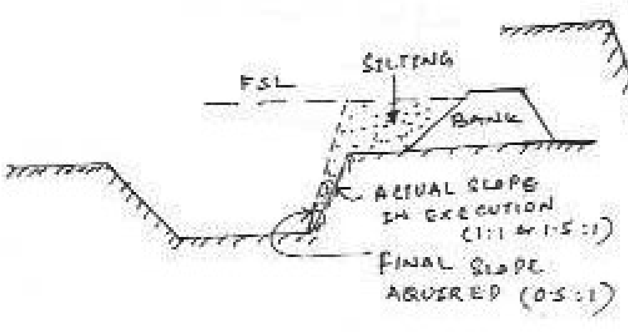
- Sometimes, when the natural surface level (i.e; NSL) is above the top of the bank, the entire canal section will have to be in cutting, and it shall be called 'Canal in cutting'.
- When NSL is lower than the Bed level of the canal, the entire Canal section will have to be built in filling, and it is called 'Canal in filling' or 'Canal in banking'.

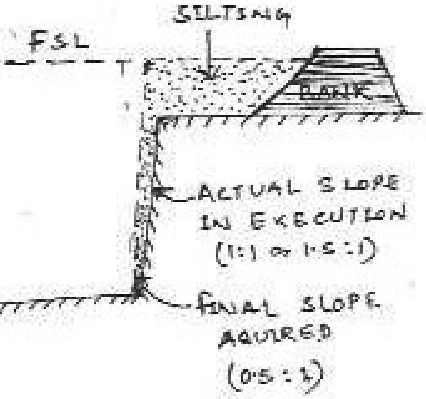


(CANAL SECTION IN FILLING)

Components of Canal c/s.

1. **Side Slopes.**





- Side slopes should be such that they are stable, depending upon the type of soil.
- Comparatively steeper slopes can be provided in cutting rather than in filling, as the soil in the former case shall be more stable.

  Slopes in cutting → 1H:1V to 1½H:1V; **Steeper Slopes**

  Slopes in filling → 1½H:1V to 2H:1V

NOTE:- In case of channels with silt laden waters, the actual capacity of the channel is worked out with ½:1 side slopes, even though flatter slopes such as 1:1 or 1½:1 may be constructed at the time of execution. This is because of the fact that the sides of such a channel gets silted upto a slope ½:1 with the passage of time.