Chapter 6: Design of Irrigation Channel

6.1 Introduction
The success of the irrigation system depends on the design of the network of canals. The canals may be excavated through the different types of soils such as alluvial soil, non-alluvial soil etc. The design consideration naturally varies according to the type of soil.

Based on the water requirements of the crops on the area to be irrigated the entire system of main canal, secondary canal, tertiary canal and field distributaries/main canal, branch canal, distributaries and water courses should be designed properly for a certain realistic value of peak discharge that must pass through them, so as to provide sufficient irrigation to the commands. Again, the design of unlined and lined canals involves different practical and economical consideration.

The design of a channel involves the selection of channel alignment, shape, size, and bottom slope and whether the channel should be lined to reduce seepage and/or to prevent the erosion of channel sides and bottom. The design of the capacity of an irrigation channel consists of the determination of the cross-sectional areas, depth, width, side slopes and the longitudinal slope, etc. for the given boundary surface. Once these parameters have been determined, the longitudinal & cross-section of the channels may be drawn.

Procedures are not presently available for selecting optimum channel parameters directly. Each site has unique features that require special considerations. Typically, the design of a channel is done by trial and error. Channel parameters are selected and an analysis is done to verify that the operational requirements are met with these parameters. A number of alternatives are considered, and their costs are compared. Then, the most economical alternative that gives satisfactory performance is selected.

The design discharge of an irrigation channel depends up on the irrigated area of crops in different seasons and the water requirement of crops. The design of the canal is mainly governed by the quantity of silt in the water and the type of boundary surface of the channel. Depending up on these factors, the irrigation channels can be broadly classified in to the following types.

1. Non-alluvial channels
2. Rigid boundary channels
3. Alluvial channels

1. The non-alluvial channels are excavated in non-alluvial soils such as loam, clay, boulder etc. Generally there is no silt problem in these channels and they are relatively stable.

2. In rigid boundary channels, the surface of the channel is lined. The quantity of silt transported by such channels remains more or less the same as that has entered the channel at the head. In such channels, relatively high velocity of flow is usually permitted which does not allow the silt to get deposited. Therefore for these channels, the problem of silt usually does not exist.

3. The alluvial channels are excavated in alluvial soils, such as silt. In the case of alluvial channels the quantity of silt may vary form section to section along reach. The silt content may increase due to scouring of bed & sides of the channel. The silt content may due to silting at some sections. If the velocity is high, scouring will occur, but when the velocity is low, silting occurs. Both these phenomena result in modifying the cross-section of the channel.
   If the bed and sides are scoured, the cross-section increases and the full supply level falls, which results in the decrease of command. On the other hand, if so silting occurs, the cross-section decreases and the discharge capacity decreases, which also results in a decrease of command, such channels should be designed for a non-scouring and non-silting velocity, called critical velocity.

6.2 Design of Non-Alluvial Channels

General
In non-alluvial channels the water is clear and therefore, no silting problem occurs. Non-alluvial soils relatively are clay, hard loam or soils formed as a result of disintegration of rocks.
Non-alluvial channels are considered stable if there is no silt problem in such channels. These channels are usually designed based on the *maximum permissible velocity* which the channel boundary surface can resist without scouring.

In the permissible velocity method, the channel size is selected such that the mean flow velocity for the design discharge under uniform flow conditions is less than the permissible flow velocity. The *permissible velocity* is defined as the mean velocity at or below which the channel bottom and sides are not eroded. This velocity depends primarily upon the type of soil and the size of particles even though it has been recognized that it should depend upon the flow depth as well as whether the channel is straight or not. Table 1 below gives the typical values of the maximum permissible velocity in different type of materials.

<table>
<thead>
<tr>
<th>S.No</th>
<th>Type of material</th>
<th>Permissible velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Lean clay soil or loam</td>
<td>0.38 to 1.37</td>
</tr>
<tr>
<td>2</td>
<td>Clay</td>
<td>0.41 to 1.67</td>
</tr>
<tr>
<td>3</td>
<td>Heavy clay</td>
<td>0.45 to 1.70</td>
</tr>
<tr>
<td>4</td>
<td>Sandy clay</td>
<td>0.52 to 1.83</td>
</tr>
<tr>
<td>5</td>
<td>Gravel</td>
<td>1.20</td>
</tr>
<tr>
<td>6</td>
<td>Boulders</td>
<td>1.5 to 1.8</td>
</tr>
<tr>
<td>7</td>
<td>Soft rock</td>
<td>1.8 to 2.4</td>
</tr>
<tr>
<td>8</td>
<td>Hard rock</td>
<td>&gt; 3.0</td>
</tr>
</tbody>
</table>

The side slopes of the channel

- For channels excavated in clay: 1:1 in cutting & 1:1.5 in filling
- For channels in gis, soft rock & hard rock taken respectively as 1:0.5, 1:0.25 and 1:0.125.
- In hard rock, the sides may be kept vertical.

The design of non-alluvial channels is usually done by using Manning's formula or Chezy's equation.

1. **Manning’s Formula.**

   \[
   V = \frac{1}{n} \left( \frac{R}{S} \right)^{1/2}
   \]

   Where
   
   - \( V \) = Velocity of flow in the channel, \( \text{m/s} \)
   - \( n \) = Mannings roughness Coefficient
   - \( R \) = Hydraulic radius of the channel
   - \( S \) = Longitudinal or hydraulic slope of a channel

   The value of \( n \) depends up on the type of surface. Table 2 below gives \( n \) values.

<table>
<thead>
<tr>
<th>S.No</th>
<th>Type of surface</th>
<th>Manning's ( n )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Earth channels, clean straight &amp; uniform</td>
<td>0.016 to 0.020</td>
</tr>
<tr>
<td>2</td>
<td>Earth channels, clean but weathered</td>
<td>0.018 to 0.025</td>
</tr>
<tr>
<td>3</td>
<td>Earth channels, with grass &amp; weeds</td>
<td>0.022 to 0.033</td>
</tr>
<tr>
<td>4</td>
<td>Channels in gravels with stones</td>
<td>0.03 to 0.035</td>
</tr>
<tr>
<td>5</td>
<td>Channels in rock, smooth &amp; uniform</td>
<td>0.025 to 0.040</td>
</tr>
<tr>
<td>6</td>
<td>Channels in rock, rough</td>
<td>0.035 to 0.050</td>
</tr>
</tbody>
</table>

2. **Chezy's Equation**

   \[ V = C \sqrt{RS} \]

   Where \( C \) is Chezy's Coefficient. The value of Chezy's Coefficient is usually determined from Bazin's equation

   \[
   C = \frac{87}{1 + \frac{K}{\sqrt{R}}}
   \]

   Where \( k \) is Basin's Coefficient, which depends up on the surface of the channel. Table 3 below shows the basins coefficient.
Table 3: Basin’s Coefficient (K)

<table>
<thead>
<tr>
<th>S.No</th>
<th>Type of surface</th>
<th>Basin's K</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Smooth Plaster</td>
<td>0.06</td>
</tr>
<tr>
<td>2</td>
<td>Ashlar, neat brick work</td>
<td>0.16</td>
</tr>
<tr>
<td>3</td>
<td>Rubble masonry</td>
<td>0.46</td>
</tr>
<tr>
<td>4</td>
<td>Plaster, very smooth earth</td>
<td>0.85</td>
</tr>
<tr>
<td>5</td>
<td>Earth channels in good condition</td>
<td>1.30</td>
</tr>
<tr>
<td>6</td>
<td>Earth channels in poor condition</td>
<td>1.75</td>
</tr>
</tbody>
</table>

Channels in which silting problems are anticipated should be designed to have some minimum permissible velocity or non-silting velocity. A minimum velocity of 0.5m/s is usually taken.

### Procedure

The following procedure is used for the design of non-alluvial channels by Manning’s formula. Similar procedure applies for the design by Chezy’s equation.

Given: The discharge \( Q \), the maximum permissible velocity \( V \), Manning’s Coefficient \( n \), Bed slope \( S \), and the side slope \( Z:1 \) are given or have been assumed.

Steps

1. Determine the area of the cross-sections from the continuity equation
   \[
   Q = AV \quad \text{or} \quad A = \frac{Q}{V}
   \]
2. Determine the hydraulic radius \( R \) from the Manning formula
   \[
   V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \Rightarrow R = \left( \frac{V n}{S^{\frac{1}{2}}} \right)^{\frac{3}{2}}
   \]
3. Determine the wetted perimeter from the relation \( P = \frac{A}{R} \)
4. Determine the depth, \( D \) and bed width \( B \) from the values of \( A \) and \( P \) obtained above by solving the equations given below. See Fig below.

\[
A = (B + ZD)D \\
P = B + 2D\sqrt{1 + Z^2}
\]

**Fig 1: Cross-section of a trapezoidal non-alluvial canal**

### Example 1

Design an irrigation channel in a non-erodible material to carry a discharge of 15cumecs when the maximum permissible velocity is 0.8m/s. Assume the bed slope of 1 in 4000, side slope 1:1 and Manning’s \( n=0.025 \).

**Given**
- \( Q=15 \text{m}^3/\text{s} \)
- \( V=0.8 \text{m/s} \)
- \( S=1 \) in 4000
- \( SS=1:1 \)
- \( n=0.025 \)

**Solution**

Step 1: Determine the cross-sectional area, \( A \)
\[
A = \frac{Q}{V} = \frac{15}{0.8} = 18.75 \text{m}^2
\]

Step 2: Compute the hydraulic radius, \( R \)
\[
V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \Rightarrow R = \left( \frac{V n}{S^{\frac{1}{2}}} \right)^{\frac{3}{2}}
\]

\[
R = \left( \frac{0.8 \times 0.025}{\left(\frac{1}{4000}\right)^{\frac{1}{2}}} \right)^{\frac{3}{2}} = 1.42 \text{m}
\]

Step 3: Determine the wetted perimeter, \( P \)
\[
P = \frac{A}{R} = \frac{18.75 \text{m}^2}{1.42 \text{m}} = 13.20 \text{m}
\]

Step 4: Determine the depth, \( D \) and bed width \( B \)
As the side slopes are given the section is trapezoidal.
6.3 Design of Lined Canals /Rigid Boundary Channel /

A lined canal is a rigid boundary channel. It can withstand much higher velocity as compared to an unlined, non-alluvial channel or alluvial channel. The design is similar to the design of non-alluvial channels discussed in the preceding section. However, the maximum permissible velocity is relatively high. See Table 4.

<table>
<thead>
<tr>
<th>S.No</th>
<th>Type of lining</th>
<th>Maximum permissible velocity, m/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Boulder lining</td>
<td>1.5</td>
</tr>
<tr>
<td>2</td>
<td>Brick tile lining</td>
<td>1.8</td>
</tr>
<tr>
<td>3</td>
<td>Cement concrete lining</td>
<td>2.7</td>
</tr>
</tbody>
</table>

The design of lined canal is usually done by Manning's formula. The value of Manning's Coefficient \( n \) depends on the type of lining. The higher values are for relatively rough surface and the lower, for smooth surface. See Table 5 below. For detailed description of \( n \) values please refer to Table 6 of chapter 5.

<table>
<thead>
<tr>
<th>S.No</th>
<th>Type of lining</th>
<th>Manning's ( n )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Cement concrete lining</td>
<td>0.013 - 0.022</td>
</tr>
<tr>
<td>2</td>
<td>Brick lining</td>
<td>0.014 - 0.017</td>
</tr>
<tr>
<td>3</td>
<td>Asphalt lining</td>
<td>0.013 - 0.016</td>
</tr>
<tr>
<td>4</td>
<td>Wood planed clean</td>
<td>0.011 - 0.013</td>
</tr>
<tr>
<td>5</td>
<td>Concrete lined excavated rock</td>
<td>0.017 - 0.027</td>
</tr>
</tbody>
</table>
Cross-section of lined canals
In most cases lined canals are designed as most economical sections. For most economical section, the hydraulic radius (R) should be maximum. Theoretically, a semi-circular section is the best section for an open channel. However, it is not practicable to adopt this section. For practical consideration a channel of trapezoidal or triangular section is usually selected. The corners of these sections are rounded to increase the hydraulic radius.

Example 2
Design the most efficient cross-section of a lined trapezoidal canal to carry a discharge of 15 cumecs when the maximum permissible velocity is 2 m/s. Assume the side slope= 1:1. Also, determine the bed slope for the canal if the Chezy coefficient, C= 60.

Given
Q=15 m³/s
V=2 m/s
SS=1:1
C= 60

Solution
Step 1: Determine the cross-sectional area, A

\[ A = \frac{Q}{V} = \frac{15}{2} = 7.5 \text{ m}^2 \]

Step 2: Compute the hydraulic radius, R

\[ R = \frac{A}{P} \]

For trapezoidal section the following holds true.

\[ A = DB + D^2 \]

Substitute the value of \( A = 7.5 \text{ m}^2 \) and \( B = 1.828D \) into the above equation

\[ 7.5 = D(0.828D) + D^2 \]

\[ = 0.828D^2 + D^2 = 1.828D^2 \]

\[ D^2 = \frac{7.5}{1.828} = 4.1026 \]

\[ D = \sqrt{4.1026} = 2.03 \text{ m} \approx 2.00 \text{ m} \]

\[ B = 0.828 \times 2 = 1.66 \text{ m} \]

Determine the bed slope, S

Chezy's formula \( V = C\sqrt{RS} \)

\[ \sqrt{S} = \frac{V}{C\sqrt{R}} \Rightarrow S = \left( \frac{V^2}{C^2 R} \right)^2, \quad R = \frac{D}{2} = 1 \text{ m} \]

\[ \therefore S = \frac{2^2}{60^2 \times 1} = \frac{1}{900} \]

Given:
\( D = 2.0 \text{ m} \)

This diagram illustrates the cross-section of a trapezoidal canal with the calculated dimensions and bed slope.
6.4 Design of Alluvial Channels

An alluvial channel is defined as a channel in which the flow transports sediment having the same characteristics as that of the material in the channel bottom. In the case of alluvial channels, the channel surface consists of alluvial soil which can be easily scoured. Moreover, the velocity is low which encourages silting. Therefore, in an alluvial channel, both scouring and silting may occur if the channel is not properly designed. The quantity of silt transported by water in alluvial channel varies from section to section due to scouring of bed & sides as well as due to silting (or deposition). If the velocity is too high, scouring may occur. On the other hand, if the velocity is too low, silting may occur.

The command of an irrigation channel decreases if the scouring occurs because the full supply level falls. The discharge capacity is decreased if the silting occurs because the reduction in cross-sectional area. Therefore, the alluvial channel should be designed such that neither scouring nor silting occurs. The velocity at which this condition occurs is called the critical velocity. Such an alluvial channel is called a stable channel. A stable channel is one in which banks and bed are not scoured and also in which no silting occurs. Even if there is some minor scouring and silting, the bed and banks of a stable channel remain more or less unaltered over a long period of time.

Two approaches have been used for the design of stable alluvial channels:
(1) Regime theory
(2) Tractive force method

The tractive force approach is more rational, since it utilizes the laws governing sediment transport and resistance to flow. The regime theory is purely empirical in nature.

1. Regime Theory
The definition of regime channel varies according to the investigating authors. Lacey [1930] defined a regime channel as a channel carrying a constant discharge under uniform flow in an unlimited incoherent alluvium having the same characteristics as that transported without changing the bottom slope, shape, or size of the cross section over a period of time. The regime theory is purely empirical in nature and was developed based on observations on a number of irrigation canals in the Indo-Pakistan subcontinent. Since the sediment concentration in these canals is usually less than 500 ppm by weight, the regime theory should be assumed to be applicable to channels carrying similar concentration of sediment load.

Generally speaking, the design of alluvial canals using regime theory depends on the investigations made on sediment/silt load of channels. Several investigators have studied the problem and suggested various theories. These are known as silt theories. The following two theories are extensively used for the design of canals in alluvial soils.

1. Kennedy's silt theory
2. Lacey's silt theory.

6.4.1 Kennedy's Theory
Gerard Kennedy, from his observation concluded that the silt supporting power in a channel cross-section is mainly dependent on the generation of eddies from the bottom width of the channel section, rising to the surface. See Fig 2. These eddies are generated due to the friction of the flowing water with the channel bottom surface. The vertical component of the eddies try to move the sediment up, while the weight of the sediment tries to bring it down, thus keeping the sediment in suspension. Thus he added if the velocity is sufficient to generate these eddies, so as to keep the sediment just in suspension, silting will be avoided. Then Kennedy stated that a channel is said to be in a state of "regime" if there is neither silting nor scouring in the channel.

![Fig 2: Generation of eddies from bottom of the channel](image-url)
Based on that, he defined the critical velocity ($V_0$) in channel as the mean velocity, which will keep the channel free from silting or scouring & related it the depth of flow by equation.

\[ V_0 = C_1 y^{C_2} \]

$C_1$ & $C_2$ are constant depending on silt discharge

\[ C_1 = 0.55 \quad \text{and} \quad C_2 = 0.64 \quad \text{in M.K.S units} \]

\[ V_0 = 0.55 y^{0.64} \quad \text{m = critical velocity ratio} \]

Later on he introduced a factor $m$ to account for soil type through which the canal passes. This factor which depends on silt grade is named as critical velocity ratio (C.V.R) denoted by $m$. Table 6 below shows the value of $m$ for different types of silt. The equation for the critical velocity was then modified as:

\[ V_0 = 0.55 my^{0.64} \]

- $V_o = \text{Critical velocity in channel in m/sec}$
- $y = \text{depth of water in meters}$
- $m = \text{critical velocity ratio (C.V.R)}$

<table>
<thead>
<tr>
<th>No</th>
<th>Type of silt</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Silt</td>
<td>0.7</td>
</tr>
<tr>
<td>2</td>
<td>Light sandy silt</td>
<td>1.0</td>
</tr>
<tr>
<td>3</td>
<td>Light sandy silt a little coarser</td>
<td>1.1</td>
</tr>
<tr>
<td>4</td>
<td>Sandy, Loamy silt</td>
<td>1.2</td>
</tr>
<tr>
<td>5</td>
<td>Debris of hard soil</td>
<td>1.3</td>
</tr>
</tbody>
</table>

**Design of Procedure**

- The critical velocity $V_o$ is to be determined by the equation $V_0 = 0.55 my^{0.64}$ by assuming a trial depth.
- Then determine the area of the channel by dividing the discharge by the critical velocity, $V_o$.
- Next determine the channel dimensions.
- Finally compute the actual mean velocity ($V$) that will prevail in the channel of this cross-section by Kutter's formula.
- Compare the mean velocity, $V$ and critical velocity, $V_o$. If the two velocities $V_o$ and $V$ works out to be the same, then the assumed depth is all right, other wise change it & repeat the procedure till $V_o$ and $V$ becomes equal.

Kutter's Formula

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

Please refer to Example 3.

**Example**

Design an irrigation channel to carry 50 m$^3$/sec 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.

<table>
<thead>
<tr>
<th>Given</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Q = 50, m^3/sec$</td>
</tr>
<tr>
<td>$S = \frac{1}{1000}$</td>
</tr>
<tr>
<td>$m = 1.1$</td>
</tr>
<tr>
<td>$n = 0.023$</td>
</tr>
</tbody>
</table>

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Solution

1) Determine the critical velocity, \( V_o \)

Assume a depth, \( y = 2m \)

\[ V_o = 0.55my^{0.64} \]

\[ V_o = 0.55 \times 1.1 \times 2^{0.64} = 0.942 \text{ m/sec} \]

2) Determine the cross-sectional area

\[ A = \frac{Q}{V_o} = \frac{50 \text{ m}^3/\text{sec}}{0.942 \text{ m/sec}} \]

\[ = 53.1 \text{ m}^2 \]

3) Determination of channel dimensions

Assume side slope of \( 1: \sqrt{2} \) in which \( Z = \frac{1}{2} \)

[Figure 3: Channel cross-section]

Cross-sectional area, \( A \)

\[ A = y(b+Zy) \]

\[ 53.1 = 2(b+0.5 \times 2) \]

\[ = 2b+2 \]

\[ b = \frac{53.1-2}{2} = 25.55 \text{ m} \]

Wetted Perimeter, \( P \)

\[ P = b + 2y\sqrt{1+Z^2} \]

\[ = 25.55 + 2 \sqrt{1+0.5^2} \]

\[ = 30.03 \text{ m} \]

Hydraulic Radius, \( R \)

\[ R = \frac{A}{P} = \frac{53.1}{30.03} = 1.77 \text{ m} \]

4) Compute the mean velocity flow, \( V \)

\[ V = \frac{1}{0.023 + \left[ 23 + \frac{0.00155}{1/4000} \right] \sqrt{1.77 \times 4000}} \times 1.33 \times \frac{1}{63.3} \]

\[ = 72.7 \times \frac{1}{1.505} \times 1.33 \times \frac{1}{63.3} \]

\[ = 1.016 \text{ m/sec} \]

5) Compare the mean velocity, \( V \) and critical velocity, \( V_o \)

\[ V = 1.016 \text{ m/sec} > V_o = 0.942 \text{ m/sec} \]

Hence we have to repeat the computation.

In order to increase the critical velocity, \( V_o \) we have to increase the depth, so increase the depth.

Assume, depth \( y = 3 \text{ m} \)

Then repeat the above procedure from 1 to 5.

\[ V_o = 0.55 \times 1.18(3)^{0.64} = 1.22 \text{ m/sec} \]

\[ A = \frac{50}{1.22} = 40.8 \text{ m}^2 \]

\[ 40.8 = 3\left[ b + \frac{1}{2} \times 3 \right] \]

\[ b = \frac{40.5 - 4.5}{3} = 12.1 \text{ m} \]

\[ p = b + 2y\sqrt{1+k^2} = 12.1 + 2(3)\sqrt{1+(0.5)^2} = 18.81 \text{ m} \]

\[ R = \frac{A}{18.81} = 2.17 \]

\[ V = \frac{1}{0.023 + 23 + \frac{0.00155}{1/4000} \sqrt{2.17 \times 4000}} \times 1.33 \times \frac{1}{63.3} \]

\[ = 1.16 \text{ m/sec} \]

\[ V = 1.16 \text{ m/sec} < V_o = 1.22 \text{ m/sec} \]

Hence we have to try again

Second trial; reduce the depth.

Depth, \( y = 2.5 \text{ m} \)

\[ V_o = 0.605 \times (2.5)^{0.64} = 0.605 \times 1.797 = 1.087 \text{ m/sec} \]
$A = \frac{50}{1.087} = 46$

$46 = 2.5 \left( b + \frac{1}{2} \cdot 2.5 \right)$

$18.4 - 1.25 = b = 17.15 \text{ m}$

$P = 17.15 + \sqrt{5 \times 2.5} = 17.15 + 5.58 = 22.73$

$R = \frac{A}{P} = \frac{4}{22.73} = 2.02$

$V = \left[ \frac{1}{0.023} + 23 + \frac{0.00155}{\frac{1}{4000}} \right] \sqrt{2.02 \times \frac{1}{4000}}$

$= 1.10 \text{ m/ sec}$

Therefore, $V = 1.1 \text{ m/ sec} > V_o = 1.087 \text{ m/ sec}$

Hence we need to try again.

Third trial, increase the depth.

**Use 2.7 m depth**

$V_0 = 0.605 \times 1.189 = 1.147$

$A = \frac{50}{1.147} = 43.5$

$43.5 = 2.8 \left( b + \frac{1}{2} \cdot 2.8 \right)$

$15.54 - 1.4 = b = 14.14 \text{ m}$

$P = 14.14 + \sqrt{5 \times 2.8} = 14.14 + 6.26 = 20.40$

$R = \frac{43.5}{20.4} = 2.13$

$V = \left[ \frac{1}{0.023} + 23 + \frac{0.00155}{\frac{1}{4000}} \right] \sqrt{2.13 \times \frac{1}{4000}}$

$= 1.148 \text{ m/ sec}$

Comparison

$V = 1.148 \text{ m/ sec} = V_o = 1.147 \text{ m/ sec}$

Accordingly, the actual mean velocity tallies with the critical velocity, $V_o$.

Hence, Use a trapezoidal canal cross-section with depth, $y = 2.7 \text{ m}$, bottom depth, $b = 14.14 \text{ m}$ or rounded to $14.20 \text{ m}$ and side slope $1:1/2$.

### 6.4.2 Lacey's Theory

Lacey carried out an extensive investigation on the design of stable channels in alluviums. On the basis of this research work, he found many drawbacks in Kennedy's Theory and he put forward his new theory.

**Lacey's regime Channels**

It is stated by Kennedy that a channel is said to be in a state of "regime" if there is neither silting nor scouring in the channel. But Lacey came out with the statement that even a channel showing no silting and no scouring may actually not be in regime.

He, therefore, differentiated between three regime conditions

(i) True regime (ii) Initial regime (iii) Final regime

According to him, a channel which is under initial regime is not a channel in regime (through outwardly it appears to be in regime as there is no silting or scouring) and hence, regime theory is not applicable to such channels. His theory is therefore applicable only to those channels which are either in true regime or in final regime.

**True Regime**

A Channel shall be in regime, if there is neither silting nor scouring. 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 and one bed slope at which a channel carrying a given discharge and a particular quantum and type of silt 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.

An artificial channel section having a certain fixed section and bed slope can behave in regime only if
1. Discharge is constant
2. Flow is uniform
3. Silt charge is constant, i.e., the amount of silt is constant
4. Silt grade is constant, i.e., the size & type of silt is always the same
5. Channel is flowing through a material which can be scoured as easily as it can be deposited (such soil known as incoherent alluvium) and is of the same grade as is transported.

Hence the designed channel shall be in "true regime". If the above conditions are satisfied. But in practice all these conditions can never be satisfied. Therefore, artificial channels can never be in "true regime", they can either be in initial regime or final regime.

**Initial regime and Final regime**

- When only the bed slope of a channel varies & its cross-section or wetted perimeter remains unaffected.
  even then the channel can exhibit "no silting & no scouring". Properties called Initial regime.

- When water flows through an excavated channel with somewhat narrower dimensions & defective slope,
  the silt carried by the water gets dropped in the upper reaches, there by increasing the channel bed slope.
  Consequently, the velocity is increased & a non-silting equilibrium is established called 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 & therefore they
  may not get eroded at all. Hence such channels will exhibit "non-silting - non-scouring" properties &
  they will appear to be in regime, but in fact they are not. 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 termed as channel in initial regime, and regime
  theory is not applicable to them as they in fact, not the channels in alluvium.

- But if there is no resistance from the sides, and all the variables such as perimeter, depth, slope are
  equally free to vary & finally get adjusted according to discharge and 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 including initial regime
  as envisaged by Kennedy.

- Final regime channels in which all variables are equally free to vary and has a tendency to assume a semi-
  elliptical section.

- The coarser the silt, the flatter is the semi-ellipse, i.e., the greater is the width of the water surface.

- The finer the silt, the more nearly the section attains a semi-circle. See Fig 4.

**Fig 4: Channel section according to silt content**

The second point which Lacey argued was that the silt supporting eddies are generated from the bottom as well
as from the sides of the channel. See Fig 5. Based on this he argued that the sediment is kept in suspension not
only by the vertical component of eddies which are generated on the channel bed, but also by the eddies
generated on the sides of the channel.

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, do not 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.

Thirdly, Lacey argued that grain size of material forming the channel is an important factor and should need
much more rational attention than what was given to it by Kennedy different values of critical velocity ration, m
for different types of soils). He, therefore introduced a term called *silt factor* \((f)\) in his equation and connected it to the average particle size as per the following equation.

\[
f = 1.75 \sqrt{d_{\text{min}}}
\]

Where, \(d_{\text{min}}\) = Average particle size in mm.

The various equations put forward by Lacey for the design of stable channels are given below in the topic design procedure for lacey theory.

**Design Procedures for Lacey’s Theory**

1. Compute the silt factor, \(f\)

\[
f = 1.75 \sqrt{d_{\text{min}}}
\]

Where, \(d_{\text{min}}\) = Average particle size in mm. See Table 7 for values of average particle size

2. Calculate the velocity of flow from the following equation

\[
V = \left[ \frac{Q f^2}{140} \right]^{1/6} \text{ m/sec}
\]

\(Q\) = Discharge is cumecs

3. Compute cross-sectional area of channel section

\[
A = \frac{Q}{V}
\]

4. Compute wetted perimeter of channel section

\[
P = 4.75 \sqrt{Q}, \text{ Where } P \text{ is in m and } Q \text{ in m}^3/\text{sec}
\]

---

**Table 7: Values of average particle size, \(d_{\text{min}}\) for various types of alluvial materials**

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Type of material (soil)</th>
<th>(d_{\text{min}}) in mm</th>
<th>Avg. grain size in mm ((d_{\text{mean}}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Silt,</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Fine</td>
<td>0.05 to 0.08</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Medium</td>
<td>0.12</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Standard</td>
<td>0.16</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Very fine</td>
<td>0.32 (f = 1.0)</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Sand,</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Medium</td>
<td>0.51</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Coarse</td>
<td>0.73</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Bajri and Sand,</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Fine</td>
<td>0.89</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>Medium</td>
<td>1.29</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>Coarse</td>
<td>2.42</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>Gravel,</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>medium</td>
<td>7.28</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>heavy</td>
<td>26.10</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>Boulders,</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>small</td>
<td>30.10</td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>medium</td>
<td>72.50</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>large</td>
<td>188.80</td>
<td></td>
</tr>
</tbody>
</table>
5. Work out the hydraulic radius/mean depth (R) from the following equation

\[ R = \frac{5}{2} \left( \frac{v^2}{f} \right) \]

Compare the value so obtained with the value of hydraulic mean depth computed by using the following formula.

\[ R = \frac{A}{P} \text{ in m} \]

6. Knowing the values of A and P above, the channel sections (the depth, y and bottom width, b) is then determined.

7. Finally, determine the bed slope S by the following equation.

\[ S = \frac{5}{\frac{f^3}{3340Q^{1/6}}} \]

See example 4.

**Example 4**

Design a regime channel of or a discharge of 50 cumecs and silt factor 1:1 using Lacey's Theory.

**Given**

Q = 50 cumecs  \( f = 1.1 \)

**Solution**

1. Compute silt factor, f

   It is already given as \( f = 1.1 \)

2. Compute the mean Velocity of flow

\[ V = \left( \frac{Qf^2}{140} \right)^{1/6} = \left( \frac{50 \times (1.1)^2}{140} \right)^{1/6} = \left( \frac{50 \times 1.21}{140} \right)^{1/6} = (0.432)^{0.167} = 0.869\text{ m/sec} \]

3. Compute cross-sectional area of channel section

\[ A = \frac{Q}{V} = \frac{50}{0.869} = 56.3\text{ m}^2 \]

4. Compute wetted perimeter of channel section

\[ P = 4.75\sqrt{Q} = 4.75\sqrt{50} = 33.56\text{ m} \]

5. Compute the hydraulic mean depth, R

i) With Lacey formula

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

ii) With conventional formula

\[ R = \frac{A}{P} = \frac{56.3\text{ m}^2}{33.56\text{ m}} = 1.677 \]

The two values are very close and may be taken as correct and ok.

6. Computation of the dimension of the channel section

For a trapezoidal channel, choose side slope.

Side slope, \( IV : \frac{1}{2}H, \text{ thus } Z = \frac{1}{2} = 0.5 \)

For a trapezoidal cross section

The cross-sectional area, \( A = y(b + Zy) \)

The wetted perimeter, \( P = b + 2y\sqrt{1 + z^2} \)

Substitute the known values from above
Substituting \( y=1.85 \text{m} \) into equ. 1 above we can get the value of \( b \).

\[ b = 33.56 - 2.24y \]
\[ = 33.56 - 2.24(1.85) \]
\[ = 29.42 \text{m} \]

### 6.5 Fixing the longitudinal section of the canal and other design consideration

If a channel is designed according to Lacey’s theory, it shall have a fixed slope and fixed section for a given discharge & silt factor. But on the other hand, if the channel is designed on Kennedy’s theory it can have different sections for different slopes.

In practice, it has been found out that Lacey’s slope equation gives excessive slopes. The slope of the channel is how ever fixed on the available country slope consistent with economy. A steeper slope governed by maximum permissible velocity, will be economical, but will lower the FSL, causing less irrigation area. Hence, the maximum possible irrigation coverage would indicate flatter slopes governed by minimum permissible velocity. A via media between these two limits must be adopted for selecting a suitable bed slope of the channel.

If the chosen designed slope is found to be flatter than the natural available slope, the difference can be adjusted by providing suitably designed drops (Chapter 7). But if the designed slope is steeper than that available, then adjustments are made to change the design slope, so as to make it near the available slope as possible.

Since a change in depth causes non-uniform flow, it is desirable to change the depth as less as possible. For this reason, the channels in the upper reaches are generally designed with large bed width to depth ratio. Suitable adjustments in bed slope, depth etc. can be made for fixing FSL on various considerations. After fixing L-section of the channel, the cross-section can be fixed on the basis of various canal standards.
Design Procedure

(1) The longitudinal section of the existing ground along the proposed canal alignment is plotted on a suitable scale.
(2) A suitable channel slope is assumed.
(3) A slope line is marked for drawing FSL line, keeping in view the guide lines already given.
(4) The channel is designed from its tail reach to its head reach, km to km.
(5) The discharge required in the channel in the given reach, for required irrigation, potential is worked out and losses are added so as to calculate the required discharge.
(6) The bed slope, FSL, falls/drops, etc. are adjusted using intelligence, judgment and knowledge. The bed levels, water depths, etc. are drawn on L-section. The X-section at every km is drawn on L-section.

Example 3

A distribution canal takes off from a branch canal having canal bed level at 204m and FSL at 205.8m. The gross command area at the head of the distributary is 30,000 hectares and after each km it is reduced by 5000 hectares. Out of this command, the culturable area is only 75%.

The intensity of irrigation for season 1 and season 2 is 32% and 15% respectively.

Design suitable channel section for the first 3km of this distributary, assuming the following data.

(i) Total losses below km 3 = 0.44m³/sec
(ii) Channel losses occurs @ 2m³/sec/million square m. of wetter perimeter
(iii) Crop period for wheat (season 1) = 4 weeks = B
(iv) Kor depth for wheat = 14cm = (Δ)_{wheat}
(v) Kor period for rice (season 2) = 2.5 weeks = B
(vi) Kor depth for rice (season 2) = 20cm = (Δ)_{rice}
(vii) Manning's, n = 0.0225
(viii) Critical velocity ratio (C.V.R) = 0.95
(ix) Silt factor = f = 1

The ground levels at every 200 meters, along the line of proposed alignment have been obtained and are tabulated in the table below.

<table>
<thead>
<tr>
<th>Distance from Head</th>
<th>Reduced G.L</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>205.20</td>
</tr>
<tr>
<td>200</td>
<td>205.30</td>
</tr>
<tr>
<td>400</td>
<td>205.25</td>
</tr>
<tr>
<td>600</td>
<td>205.00</td>
</tr>
<tr>
<td>800</td>
<td>204.90</td>
</tr>
<tr>
<td>1000</td>
<td>204.30</td>
</tr>
<tr>
<td>1200</td>
<td>204.30</td>
</tr>
<tr>
<td>1400</td>
<td>204.20</td>
</tr>
<tr>
<td>1600</td>
<td>204.20</td>
</tr>
<tr>
<td>1800</td>
<td>204.10</td>
</tr>
<tr>
<td>2000</td>
<td>204.05</td>
</tr>
<tr>
<td>2200</td>
<td>204.00</td>
</tr>
<tr>
<td>2400</td>
<td>203.95</td>
</tr>
<tr>
<td>2600</td>
<td>203.95</td>
</tr>
<tr>
<td>2800</td>
<td>203.90</td>
</tr>
<tr>
<td>3000</td>
<td>203.80</td>
</tr>
</tbody>
</table>

Solution

➤ The channel is to be designed from its tail (where the losses are known) towards its head km by km.
➤ The gross command areas and culturable commanded areas at various kms are, first of all worked out in table 9 below.
Table 9: Gross and culturable command areas

<table>
<thead>
<tr>
<th>Below km</th>
<th>Gross command area, ha</th>
<th>Gross culturable area (G.C.A) (ha)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 (i.e., head)</td>
<td>30,000</td>
<td>22,500 (30,000*0.75)</td>
</tr>
<tr>
<td>1</td>
<td>25,000</td>
<td>18,750 (30,000*0.75)</td>
</tr>
<tr>
<td>2</td>
<td>20,000</td>
<td>15,000</td>
</tr>
<tr>
<td>3</td>
<td>15,000</td>
<td>11,250</td>
</tr>
</tbody>
</table>

Outlet discharges for the two crop seasons are determined as given below.
(i) For season 1

\[ D = \frac{8.64B}{\Delta} = \frac{8.64 \times 28}{0.14} = 1728 \text{ ha/m}^3/\text{s} \]

\[ B = 4 \text{ weeks} = 28 \text{ days} \quad \Delta = 14 \text{ cm} = 0.14 \text{m} \]

(ii) For season 2

\[ D = \frac{8.64B}{\Delta} = \frac{8.64 \times 17.5}{0.20} = 756 \text{ ha/m}^3/\text{s} \]

\[ B = 2.5 \times 7 = 17.5 \text{ days} \quad \Delta = 20 \text{cm} = 0.20 \text{m} \]

Intensity of irrigation for season 1 = 32%
Intensity of irrigation for season 2 = 15%

If G is the gross culturable area at any point, then 0.32 G is the area for season 1 and 0.15 G is for season 2.

\[ \text{Outlet discharge} = \frac{\text{Area}}{\text{Duty}} \]

\[ \therefore \text{Discharge required for season 1 (outlet discharge)} = \frac{0.32G}{\text{Outlet factor for season 1}(D)} = \frac{0.32G}{1728} = \frac{G}{5400} \]

\[ \text{Discharge required for season 2 (Outlet discharge)} = \frac{0.15G}{756} = \frac{G}{5040} \]

Thus \[ \frac{G}{5040} > \frac{G}{5400} \]

Since the discharge required for season 2 is greater than that required for season 1, the outlet factor for season 2 becomes a controlling factor.

Discharges required (needed) at various kilometers for the given command are worked out below.

Table 10: Computation of discharge

<table>
<thead>
<tr>
<th>Below km</th>
<th>Culturable Area G</th>
<th>Discharge required Col.2, in m$^3$/sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>0</td>
<td>22,500</td>
<td>4.46</td>
</tr>
<tr>
<td>1</td>
<td>18,700</td>
<td>3.72</td>
</tr>
<tr>
<td>2</td>
<td>15,000</td>
<td>2.98</td>
</tr>
<tr>
<td>3</td>
<td>11,250</td>
<td>2.23</td>
</tr>
</tbody>
</table>
Design at 3 km

- Losses below km 3 = 0.44 m³/sec (given)
- Discharge required for crop at this point = 2.23 m³/sec
- Total discharge required = 2.23 + 0.44 = 2.67 m³/sec
- Design discharge = 10% more than required
  \[ Q_d = 1.1 \times 2.67 = 2.94 \text{ m}^3/\text{sec} \]

Critical velocity ratio \(\frac{V}{V_c} = 1, n = 0.0225\)

- Lacey’s regime slope for this discharge & silt factor 1 is approximately 22 cm per km
- Determine S from \(S = \frac{5}{340Q^{1/6}}\) and is found to be, \(S = \frac{1}{4550}\)
- Assuming side slope of \(1: \frac{1}{2}\), the channel section is designed and the cross-sectional dimensions are found to be as indicated below.
  \((b = 4.5 \text{ m})\)
  \((y = 1.06 \text{ m})\)

<table>
<thead>
<tr>
<th>Discharge</th>
<th>S.</th>
<th>B m</th>
<th>y m</th>
<th>(A = \left(\frac{B + \frac{y}{2}}{2}\right)) \text{ m}^2</th>
<th>(V = \frac{Q}{A}) m/sec</th>
<th>(V_0) m/sec</th>
<th>(\frac{V}{V_0})</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.94 cumecas</td>
<td>1</td>
<td>4.444</td>
<td>0.98</td>
<td>5.38</td>
<td>0.55</td>
<td>0.53</td>
<td>1.04</td>
<td>much larger than 0.95</td>
</tr>
<tr>
<td>4.5</td>
<td>1.05</td>
<td>5.28</td>
<td>0.56</td>
<td>0.58</td>
<td>0.96</td>
<td>O.K.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

ii) Design at km 2

- Outlet discharge required below km 2 = 2.98 m³/sec
- Losses below km 3 = 0.44 m³/sec
- Losses in channel b/n km 3 & 1 cm 2
  For the calculation of this loss, the perimeter of the section at km - 3 shall be taken
  \(Wetted\ \text{perimeter} = b + \sqrt{5} \times y = 4.5 + \sqrt{5} \times 1.08 = 6.92 \text{ m}\)

\[
\text{Loss} @ \frac{2 \text{ m}^3}{\text{sec}}/10^6 \text{ sqm} = 2 \times \left[\frac{6.92 \times 1000}{10^6}\right] = 0.014 \text{ m}^3/\text{sec}
\]

Total loss below km -2
  = Losses below km 3 + losses b/n km 3 & km 2
  = 0.44 + 0.014 = 0.454 m³/sec

Total discharge required at km 2 = 2.98 + 0.454 = 3.434 m³/sec

\[ \text{Use the same side slope } S = \frac{1}{4500} \text{ and } \left(\frac{b = 6.0 \text{ m}}{Y = 1.08}\right) \]

<table>
<thead>
<tr>
<th>Q</th>
<th>S.</th>
<th>B m</th>
<th>y m</th>
<th>(A = \left(\frac{B + \frac{y}{2}}{2}\right)) m²</th>
<th>(\frac{Q}{A}) = m/sec</th>
<th>(V_0) m/sec</th>
<th>(\frac{V}{V_0})</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.79 cumecas</td>
<td>1</td>
<td>4.444</td>
<td>1.05</td>
<td>6.85</td>
<td>0.55</td>
<td>0.58</td>
<td>0.95</td>
<td>O.K.</td>
</tr>
</tbody>
</table>
(iii) Design at km 1

- Outlet discharge required below km 1 = 3.72 m³/sec
- Losses below km 2 worked out earlier = 0.454 m³/sec
- Losses between km 2 and km 1

To work out these losses, the perimeter of the section at km 2 shall be taken, as the section at km 1 is not known so far.

\[
\text{Wetted perimeter at section km 2, } p = b + 2y\sqrt{1 + k^2} = 6 + 2 \times 1.08\sqrt{1 + (0.5)^2} = 8.415 m
\]

\[
\text{Losses @ } 2 m^3/\text{sec/ million sq.m} \quad (i.e., \text{in the length of 1km, i.e., } 1000m) = 2 \times \left[ \frac{8.42 \times 1000}{10^6} \right] = 0.017 m^3/\text{sec}
\]

Total losses below km 1 = 0.454 + 0.017 = 0.471 m³/sec

Total discharge required at km 1 = 3.72 + 0.471 = 4.191 m³/sec

Design discharge = 1.1 \times 4.191 = 4.61 m³/sec

Let us adopt a slope of 20 cm in 1 km i.e.

\[
S = \frac{0.20}{1000} = \frac{1}{1000} = \frac{1}{5000}
\]

<table>
<thead>
<tr>
<th>Q</th>
<th>S</th>
<th>B</th>
<th>Y</th>
<th>A</th>
<th>V_A</th>
<th>V_0</th>
<th>V/V_0</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.61</td>
<td>1/5000</td>
<td>6.0</td>
<td>1.2</td>
<td>7.92</td>
<td>0.582</td>
<td>0.615</td>
<td>0.947</td>
<td>OK</td>
</tr>
</tbody>
</table>

iv) Design at 0 km

- Outlet discharge required at 0 km = 4.46 m³/sec
- Losses below km 1 as found above = 0.471 m³/sec
- Losses b/n km 0 to 1:

To work out these losses, the perimeter of the section at km 1 shall be taken, as the section at km 0 is not known so far.

\[
\text{Wetted Perimeter } P = b + \sqrt{5.12} = 6 + \sqrt{5.12} = 6 + 2.68 = 8.68 m
\]

\[
\text{Losses @ } 2 m^3/\text{sec/ million sq.m in a length of 1km} = 2 \times \left[ \frac{8.68 \times 1000}{10^6} \right] = 0.01736 \approx 0.017 m^3/\text{sec}
\]

Total losses below km 0 = 0.471 + 0.017 = 0.488 m³/sec

Total discharge required at 0 km = 4.46 + 0.488 = 4.948 m³/sec

Design discharge = 1.1 \times 4.948 = 5.44 m³/sec

Let us adopt a slope of 20 cm in 1 km i.e., \( S = \frac{1}{5000} \)

<table>
<thead>
<tr>
<th>Q</th>
<th>S</th>
<th>B</th>
<th>Y</th>
<th>A</th>
<th>V/A</th>
<th>V_0</th>
<th>V/V_0</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.44 m³/sec</td>
<td>1/5000</td>
<td>7.2</td>
<td>1.2</td>
<td>9.36 m²</td>
<td>0.58 m/sec</td>
<td>0.615</td>
<td>0.944</td>
<td>OK</td>
</tr>
</tbody>
</table>

\[
B = b = 7.2 m
\]

Adopt \( y = 1.2 m \)

\[
S = \frac{1}{5000}
\]
**Detailed Design at km 3: Presented as example**

- Losses below km 3 = 0.44 m³/sec
- Discharge required at this point = 2.23 m³/sec
- Total discharge required = 0.44 + 2.23 = 2.67 m³/sec
- Design Discharge = 1.1 * 2.67 = 2.94 m³/sec

\[
\frac{V}{V_o} = CVR = 1\\
n = 0.0225\\n = 1\\n\frac{S}{3340Q^{1/6}} = \frac{5/3}{f^{1/3}}\\n = \frac{1}{4000}
\]

- \( y = 0.5\sqrt{B} \)
- \( B = 5 \)
- \( y = 0.5\sqrt{5} \)
- \( = 1.12 m \)

\[
V_0 = 0.55 * 1 * (1.12)^{0.64} = 0.59 \frac{m}{scc}\\nA = 1.12[5 + 0.5 * 1.12] = 6.23 m^2\\n\frac{V}{A} = 0.47 = \frac{0.47 V}{6.23} = 0.47 n/s\\n\frac{V}{V_o} = 0.59 = 0.792\\nB = 4.5\\ny = 0.5\sqrt{4.5} = 1.06
\]

\[
A = 1.06[4.5 + 0.5 * 1.06] = 5.332 m^2\\nV = \frac{2.94}{5.332} = 0.55\\nV_o = 0.55 * 1 * (1.06)^{0.64} = 0.57\\n\frac{V}{V_o} = 0.55 = \frac{0.47}{0.57} = 0.6565\\nB = 4.5 m\\ny = 1.06 m
\]

- Hence Adopt
- \( S = \frac{1}{4000} \)
- \( = 25 cm per km \)

All the data worked out above, has been entered at their proper places in ‘Schedule of area statistics and channel dimensions’. See Table 11 below. The table has been completed with the help of Canal Standards by making some adjustments. Then the L-section of the distributary canal is drawn as shown in Figure Starting from the (i.e., 0km) by keeping its FSL at the head at 0.2 m below the FSL of the branch channel.
### Table 11: Schedule of area statistics and channel dimensions

<table>
<thead>
<tr>
<th>Below km</th>
<th>Gross commanded area (hectares)</th>
<th>Cultivable commanded area (hectares)</th>
<th>Controlling area to be irrigated (15%) in hectares</th>
<th>Controlling outlet discharge factor</th>
<th>Losses in reach (cusecs)</th>
<th>Total losses (cusecs)</th>
<th>Design discharge (cusecs)</th>
<th>Bed slope (cm/km)</th>
<th>Bed width (m)</th>
<th>Water depth (metres)</th>
<th>Freeboard from Table 14.11 in metres</th>
<th>Ht. of bank above ground decided after drawing L-section</th>
<th>Width of bank from Table 14.11</th>
<th>Velocity in m/sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>30,000</td>
<td>22,500</td>
<td>3,775</td>
<td>756</td>
<td>0.017</td>
<td>0.488</td>
<td>5.44</td>
<td>20</td>
<td>7.2</td>
<td>1.2</td>
<td>0.6</td>
<td>1.00</td>
<td>2.25</td>
<td>0.58</td>
</tr>
<tr>
<td>1</td>
<td>25,000</td>
<td>18,750</td>
<td>2,812.5</td>
<td>756</td>
<td>0.010</td>
<td>0.471</td>
<td>4.61</td>
<td>20</td>
<td>6.0</td>
<td>1.2</td>
<td>0.5</td>
<td>1.00</td>
<td>2.0</td>
<td>0.582</td>
</tr>
<tr>
<td>2</td>
<td>20,000</td>
<td>15,000</td>
<td>2,250.0</td>
<td>756</td>
<td>0.014</td>
<td>0.454</td>
<td>3.79</td>
<td>22.5</td>
<td>6.0</td>
<td>1.05</td>
<td>0.5</td>
<td>0.90</td>
<td>2.0</td>
<td>0.55</td>
</tr>
<tr>
<td>3</td>
<td>15,000</td>
<td>11,250</td>
<td>1,687.5</td>
<td>756</td>
<td>—</td>
<td>0.440</td>
<td>2.94</td>
<td>22.5</td>
<td>4.2</td>
<td>1.05</td>
<td>0.5</td>
<td>0.925</td>
<td>2.0</td>
<td>0.56</td>
</tr>
</tbody>
</table>

Note: $\frac{V}{V_0} = m$
### Figure 6: Longitudinal section of the distributary canal

| Cutting depth | 0.50 | 0.94 | 0.95 | 0.72 | 0.66 | 1.26 | 0.70 | 0.74 | 0.68 | 0.72 | 0.76 | 0.66 | 0.644 | 0.638 | 0.782 | 0.678 | 0.62 |
|---------------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|
| Designed discharge | 5.44 m³/s | 4.61 m³/s | 3.79 m³/s | 2.94 m³/s |
| Bed slope | 20 cm/km | 20 cm/km | 22.5 cm/km | 22.5 cm/km |
| Depth | 1.20 m | 1.20 m | 1.05 m | 1.05 m |
| Bed width | 7.2 m | 6.0 m | 6.0 m | 4.2 m |
| Designed FSL (m) | 205.60 | 205.44 | 204.84 | 204.80 |
| Ground level available (m) | 204.40 | 205.30 | 205.25 | 205.20 |
| Designed bed level (m) | 204.40 | 204.36 | 204.32 | 204.28 |
| Chainage in metres | 0 | 300 | 600 | 900 | 1200 | 1500 | 1800 | 2100 | 2400 | 2700 | 3000 | 3300 | 3600 | 3900 | 4200 | 4500 | 4800 |

0 km | (1 km) | (2 km) | (3 km)