scour below top of the upstream pucca pavement. Its thickness is fixed on the same principles as those for the downstream talus.

![Diagram showing Inverted filter, Position of settled stone, Level of deepest scour, D/S Talus, 2S y, 3y, D/S H.F.L, and downstream loose stone talus.]

Fig. 61

There are some theories for the design of impervious aprons on permeable foundations. The two common theories are:

(a) Bligh’s creep theory.

(b) A. N. Khosla’s theory.

4. Bligh’s creep theory for weirs on permeable foundation: According to Bligh, subsoil water passing through permeable foundation creeps along the bottom contour of the weir and its pucca aprons. It starts creeping from upstream end of the upstream pucca pavement and, while creeping towards the downstream side, it goes on losing its energy and comes out at downstream end of the downstream pucca apron with a velocity known as exit velocity. In order that this velocity may be safe, the minimum length of creep may be found from the following empirical formula (see fig. 62):

\[ l = CH \text{ feet} \]

where, \( l = \text{minimum length of creep of the subsoil water.} \)

\( H = \text{maximum cutoff across the weir i.e. the head from top of the weir wall or crest shutter (if provided) to the top of downstream pucca apron.} \)

\( C = \text{coefficient of creep (or classification number) of the foundation soil.} \)
Mr. Bligh suggested the following values of $C$:

(i) $C = 5$ to $9$ for boulder, shale, shingle, gravel.
(ii) $C = 12$ for coarse sand, as found in Madras and Central India river beds.
(iii) $C = 15$ for fine sand, as found in Punjab river beds in mountainous region.

From the above equation we get, $\frac{H}{l} = \frac{1}{C}$; hence $\frac{1}{C}$ is called hydraulic gradient of the subsoil flow under the structure.

*Note:* In Metric system, if $H$ be in metres, $l$ shall be in metres.

Length of creep is inclusive of cutoffs if any, the value of creep-length along each cutoff being equal to twice its depth. The aggregate creep length provided by the upstream pucca apron, the downstream pucca apron and the cutoffs should be greater than its minimum value $l$ or $CH$ so that the foundation soil may not be disturbed by the excessive velocity of creep or percolation. Bligh has also given some empirical formula for the lengths of downstream pucca apron and the total downstream protection consisting of downstream pucca apron and downstream pervious protection in the form of talus. These formula are:

$$l_1 = 4C \sqrt{\frac{H_5}{13}} \text{ feet}$$  \hspace{1cm} (i)

where, $l_1 =$ length of downstream pucca apron.
\[ H_s = \text{head from the top of crest shutter to the top of downstream pucca apron. For weir without crest shutters, } H_b \text{ (shown in fig. 62) will be used instead of } H_s \text{ in the above formula.}
\]

Knowing the values of length of creep, cutoffs and \( l_1 \), we can find length \( l_2 \) of the upstream pucca apron. Also,
\[
l_3 = 10C \sqrt{\frac{H_s}{13}} \times \frac{q}{75} \text{ feet} \quad \ldots \ldots \ldots \ldots \quad (ii)
\]

where, \( l_3 = \text{downstream pucca apron + downstream talus; hence, the downstream talus length will be } (l_3 - l_1) \text{ feet; } \)

\( q = \text{intensity of high flood discharge in cusecs i.e. discharge in cusecs per foot run of the length of weir; } H_s \text{ has usual significance.}
\]

Length of upstream talus, not shown in figure, may be kept as \( \left( \frac{l_3 - l_1}{2} \right) \) feet.

Thickness of downstream pucca apron is found from the formula,
\[
t = \frac{4}{3} \cdot \frac{H_r}{s - 1} \text{ feet} \quad \ldots \ldots \ldots \ldots \quad (iii)
\]

where, \( t = \text{thickness at any point of pucca apron, in feet, using a factor of safety of } 4/3. \)

\( H_r = \text{residual head or net uplift at that point, in feet } = (H - h_f), \text{ in which } H \text{ is the maximum cutoff and } h_f \text{ is the head lost by water while creeping, from upstream, upto that point.}
\)

\( s = \text{specific gravity of the masonry of apron.}
\]

To know the net uplift at any point of the downstream pucca apron, the hydraulic gradient of the subsoil flow is first plotted. Net uplift at a point will be the ordinate from the hydraulic gradient to the bottom of apron at that point. Greater the length of creep, flatter will be the hydraulic gradient and less will be the uplift and consequently, the thickness of downstream pucca apron required will also be less. As already said, the hydraulic gradient should be flat enough to ensure safe exit velocity.

\text{Note: In the above formula, if } H_r \text{ be in metres, } t \text{ will be in metres.}
5. **Khosla's theory for the design of weirs on permeable foundation:** This theory is given, at length, in technical paper No. 12 published by Central Board of Irrigation and Power. The main principles of this theory, in brief, are as given below:

(a) The flow of subsoil water through the permeable foundation soil is in stream lines and, when deep vertical cutoffs are provided, the creeping of water along the bottom contour of pucca floor is impossible.

(b) Subsoil water, as it travels from the upstream end of upstream pucca floor to the downstream end of downstream pucca floor, has certain force at each point along its path of flow; this force \( F \) acts in the direction of flow, tangential to the path of flow as shown in fig. 63. Vertical component of this force tries to disturb the particles of foundation soil; this force has the maximum disturbing tendency at exit point of the subsoil water i.e. at the downstream end of downstream pucca pavement where the subsoil water comes up at the end of its travel through the subsoil. At this point, therefore, the submerged weight \( W_s \) of the soil particles should be more than the upward disturbing force. The disturbing force at any point (and therefore at exit) is propor-
tional to the gradient of the pressure of water at that point; the gradient of pressure of water at exit is called the exit gradient. In order that the soil particles at exit may be stable, the upward pressure at exit should be safe or in other words, the exit gradient should be safe since the force is proportional to the pressure gradient. The exit gradient is said to be critical when the upward disturbing force at exit is just equal to the submerged weight of soil particles at exit. In practice, a factor of safety of 4 to 5 is kept and the exit gradient, in such case, is called the safe exit gradient. Safe exit gradient is thus \( \frac{1}{4} \) to \( \frac{1}{6} \) of the critical exit gradient. Values of safe exit gradients for some of the subsoils are as given in the following table:

<table>
<thead>
<tr>
<th>No.</th>
<th>Kind of subsoil</th>
<th>Safe exit gradient (denoted as ( G_e ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>(i)</td>
<td>Shingle</td>
<td>0.25 to 0.2 ( (or, \frac{1}{4} \text{ to } \frac{1}{6}) )</td>
</tr>
<tr>
<td>(ii)</td>
<td>Coarse sand</td>
<td>0.2 to 0.17</td>
</tr>
<tr>
<td>(iii)</td>
<td>Fine sand</td>
<td>0.17 to 0.14</td>
</tr>
</tbody>
</table>

(c) Undermining of the floor starts from downstream end of the downstream pucca floor and it travels upstream towards the weir wall, if left unchecked. This undermining starts only when the exit gradient is unsafe for the subsoil on which the weir is founded. It is therefore absolutely necessary to have a reasonably deep vertical cutoff at the downstream end of downstream pucca floor to prevent undermining. Depth of this cutoff is governed by the maximum depth of scour and the safe exist gradient. The exit gradient actually provided depends on the maximum head across the weir, total length of upstream and downstream pucca floors and the depth of cutoff at the downstream end of downstream pucca floor.

(d) Failure of weir on permeable foundation is due to:
   (i) Surface flow and,
   (ii) Subsurface flow.
Surface flow causes scour and dynamic action; in addition, it causes uplift on the downstream pucca floor if hydraulic jump forms on the downstream side of weir. Uplift due to this cause has to be investigated for various stages of riverflow and the maximum due to this cause should be picked out.

Subsurface flow causes uplift on pucca floor, net uplift being only on the downstream pucca floor. Uplift due to subsurface flow is maximum when there is maximum cutoff across the weir. The downstream floor should be designed for the worst uplift due to either surface flow or subsurface flow. Subsurface flow may also cause undermining if the exit gradient is unsafe.

To dissipate the surplus energy of water on the downstream side of weir, hydraulic jump is very effective. In addition, the friction blocks (projecting up from top of the downstream pucca floor) are provided as shown in fig. 60. These friction blocks consist of:

(i) Impact blocks provided slightly on the downstream side of toe of the downstream glacis in the case of sloping masonry weir.

(ii) Deflector blocks at the end of downstream pucca floor.

(c) It is advisable to provide vertical cutoff at the upstream end of the upstream pucca floor and a few intermediate cutoffs between this cutoff and one at downstream end of the downstream pucca floor.

6. Finding the thickness of downstream pucca gravity floor according to Khosla’s theory: Uplift pressures at key points $E$, $D$ and $C$ (see fig. 60) located round each cutoff or toe wall as the case may be, are first read from the plates (prepared by Khosla) corresponding to given data about the length of pucca floor and the position and depth of cutoffs. These plates are given in technical-paper No. 12 published by C. B. I. & Power. From these plates, the pressures are read as percentages of the maximum head across the weir and they are denoted by $\phi_E$, $\phi_D$ and $\phi_C$ for points $E$, $D$ and $C$ respectively; $\phi_E$, $\phi_D$ and $\phi_C$ are then corrected for the following corrections:
Weir scouring sluices (or undersluices) and head regulator of main canal

Fig. 64(a)
(a) Correction for the thickness of floor.

(b) Correction for the mutual interference of neighbouring sheet piles with each other.

(c) Correction for the slope of floor, if any.

The corrected pressures $\phi_E'$, $\phi_D'$ and $\phi_C'$ at points $E$, $D$ and $C$ are then converted into feet of water to give the residual heads (in feet of water) at points $E$, $D$ and $C$. The residual heads (i.e. the uplift pressures in feet of water) at points of floor between any two cutoffs are found on the assumption that the residual head varies by a straight line law between these two neighbouring cutoffs. The thickness of floor is then found from the residual heads as in Bligh's theory, if the floor be a gravity floor.

In some cases, the floor is designed as a R.C.C. raft to resist uplift pressure. The raft is assumed to be fixed at the positions of piers which are provided, in such case, along the width of river. Such a floor is called a non-gravity floor and it is often provided in barrages.

7. Undersluices or Weir scouring sluices: Obstruction due to weir wall causes deposit of silt on the upstream side of weir; it is objectionable if such silt deposit takes place on the immediate upstream side of the head regulator of main canal as it will interfere with the proper working of the main canal. Hence, weir scouring sluices are provided at the flank of weir, near the head regulator (i.e. H.R.) of main canal as shown in figs. 64(a) and 64(b). In order that these sluices may be effective, a divide wall is provided (on the upstream and downstream sides of weir and normal to weir) at that end of sluices which is away from the head regulator. The downstream divide wall is carried upto the end of downstream talus of undersluices. The upstream divide wall, the sill of scouring sluices and the crest wall of H.R. form a pocket (or silt trap) on the upstream side of the sluices and just in front of the head regulator. Silt gets deposited in this pocket and is periodically scoured by weir scouring sluices and taken out through them to the downstream side of weir. Thus, the pocket and undersluices help in minimis-
ing the chances of entry of silt into the main canal through the head regulator of main canal

If \( x \) metres be the length of head regulator face, the length of upstream divide wall should be \( x \) metres or slightly more. The total length of undersluiices between the divide wall and head regulator face may be about \( 1.5x \); actually, this length should be such that the discharging capacity of undersluiices should be sufficient to perform their main function of scouring the pocket efficiently; some authorities suggest that for the efficient working of undersluiices, the discharging capacity of undersluiices should be about twice the full supply discharge of main canal at its head. Other subsidiary consideration in fixing the discharging capacity of undersluiices is that they should be able to dispose off 10 to 15% of the high flood discharge during severe floods. The spans of undersluiices should be wide enough (usually 9 m to 18 m i.e. 30' to 60') in order to be efficient in scouring action. Stoney gates are usually used for controlling the flow through undersluiices. In closed position, the top of these gates should be above pond level in the pocket. Pond level is the river water level in the pocket which is maintained, as far as practicable, to feed the main canal with its full supply discharge.

Sill of undersluiices is kept at or slightly above the deepest river bed and about 0.9 to 1.8 m (3' to 6') below the sill of head regulator (i.e. top of the crest wall of head regulator). The floor of pocket is kept at the deepest river bed. Undersluiices are designed like a regulator; the thickness of pucca floors is designed as in case of the main weir portion; but the downstream aprons (impervious as well as pervious) should be longer and stronger due to the greater intensity of discharge passing through undersluiices and due to the consequent more velocity of retreat and more likelihood of erosion of the downstream river bed. In the case of undersluiices, according to Bligh,

\[ i_1 = 7C \sqrt{\frac{H_s}{13}} \text{ feet} \]  

\[ l_3 = 15C \sqrt{\frac{H_s}{13}} \times \frac{q}{75} \text{ feet} \]
Here, $l_1$, $l_3$, $C$ and $q$ have the usual significance;

and, $H =$ head from top of sluice shutters (or the pond level in pocket) to the downstream low water level (i.e. top of downstream pucca apron).

Ordinarily, for working the undersluices, still pond system of regulation is resorted to. According to this system of regulation, the undersluices remain closed while water is flowing through the head regulator of main canal. Due to this, water will enter pocket with a very low velocity and will bring less silt with it in the first instance. Also, the main canal will be fed from the comparatively still pond of water in pocket so that the chances of silt getting into the main canal are less and, of the silt getting deposited in the pocket are more. This deposited silt is taken out by first closing the head regulator spans and then opening the weir scouring sluices; this is usually done during low stages of river flow and when no water is required in the main canal. After the scouring of silt from the pocket is over, scouring sluices are again closed and the head regulator spans are then opened. Thus, the weir scouring sluices remain ordinarily closed and are opened once or twice in a year during scouring operations only. When the silt excluders are provided in the pocket, the semi-open system of pond regulation is resorted to.

8. Fish ladder: In large rivers carrying plenty of fish, a fish ladder is provided at one end of weir, next to (i.e. adjoining) the weir scouring sluices as shown in fig. 64(a). The difference of water levels on the upstream and downstream sides of weir is split up into water steps by means of the baffle walls constructed across the inclined chute of fish ladder, as shown in fig. 65. The design of fish ladder should be such that the velocity of current in the chute should not be more than 3 m/sec (10 ft/sec) so that the fish can conveniently travel upstream against this current. There is control in the form of gate at the extreme upstream and downstream end walls of the fish ladder.
9. **Movable weir crests:** If the whole height of obstruction caused by the weir be solid (i.e. of masonry or concrete), it will cause heavy deposit of coarse silt (in form of sand islands) on its upstream side; this interferes with the proper working of main canal and further, these sand islands cause cross currents which may undermine the weir foundation; also, during floods, there will be much afflux (i.e. rise in upstream water level in river) which will cause too much flooding of the upstream land and, sometimes, the damage to head works. Hence, the usual practice is to make the whole height of obstruction partly of solid weir and partly of falling (i.e. drop) crest shutters resting on the top of solid weir. Such collapsible crest shutters are also known as *movable weir crests* as distinguished from the crest of solid weir portion which is called the *permanent* weir crest or masonry weir crest. Such arrangement ensures less silting on the upstream side, lower upstream H.F.L. and higher upstream fair weather level; also, due to less afflux, comparatively low and short marginal embankments are required to confine the upstream flood water. These shutters will be in erect position on the permanent weir crest during low stages of river flow but they will lie flat on the permanent crest during floods, thus allowing the flood water to pass down without causing much afflux. The top of shutters in closed position will be at such level as to ensure full supply discharge in
main canal during low stages of river flow, when water level on upstream side is kept at the top of shutters. The types of movable weir crests, common in India, are:

(a) Flash boards or wooden falling shutters: they are usually about 1.2 m (4') high and up to 3 m (10') long. They fall automatically when water level on the upstream side goes slightly above the tops of these shutters. They can be put again in the vertical position, either by manual labour or by crane, from the portion of permanent weir crest on their downstream side, when there may even be water on the permanent crest up to a depth of 0.3 m (1 foot). They are not so suitable for diversion weir as for storage weir.

(b) Wooden or steel shutters: they are widely used for diversion weirs and may be automatic [see fig. 66(a)] or non-automatic [see fig. 66(b)]. They are usually 1.2 to 2.4 m (4' to 8') high and 1.8 to 3 m (6' to 10') long. Non-automatic shutters give better service than the automatic ones and are therefore preferred. Non-automatic shutters are lowered by manual labour before the flood water overtops them and are lifted in the erect position by crane even when there may be 0.6 to 0.9 m (2' to 3') depth of water passing over the permanent weir crest.

10. Head regulator of main canal: It is a structure at the head of main canal to control the flow of water into the main canal, as shown in fig. 64(a). A regulator essentially consists of a number of sluices (between piers) controlled by gates which are operated from the over-head gate bridge on the top of piers. The span of sluices may be from 6 to 18 m (20' to 60'); the modern tendency is towards using bigger spans controlled by Stoney gates or by sector gates. The main objects of this head regulator are:

(a) To regulate the supply of irrigation water into main canal (i.e. M.C.) according to requirements.

(b) To control the entry of coarse bed silt into the main canal.

(c) To shut out or prevent the unwanted flood water from entering the main canal, as it is heavily charged with bed silt.
The face of head regulator is kept practically in line with river bank so that the silt deposited (in front of it) in the

![Shutter diagram](image1)

*Shutter*

*Hinge*

*Tie bar*

*Weir crest*

*Shutter in fallen condition*

*Fig. 66(a)*

![Non-automatic falling shutter diagram](image2)

*Groove*

*Pin*

*Strut*

*Shutter in fallen condition Hinge*

*Hinge Weir crest*

*Fig. 66(b)*

Pocket can be easily scoured out. Crest wall is constructed along the face line of head regulator to keep out bed silt from
entering the main canal as shown in fig. 67; top of this wall is called the sill of head regulator and is fixed about 1.2 to 2.4 m (4' to 8') above the floor of pocket; also, sill level should be such that the full supply discharge of main canal can be

Head regulator of main canal in alluvial soil

**Fig. 67**

passed through head regulator into the main canal during low stages of river flow. Head wall is constructed from the top of sluice openings, upwards; top of this head wall (or breast wall) of head regulator should be about 1.2 m (4 feet) above
the upstream H.F.L. in pocket. The main factors governing
the design of head regulator of main canal are:

(i) Maximum head across the head regulator: this
is equal to the upstream H.F.L. minus the canal
bed level on the downstream side of head regulator.

(ii) Full supply discharge of main canal and the
pond level.

(iii) Nature and quantity of silt carried by river water
at the site of weir.

Waterway of head regulator should be ample so as to
ensure small velocity of entry through head regulator spans
into the main canal. This velocity of water through head
regulator is kept about 0.6 to 0.9 m/sec (2 to 3 ft/sec); such low
velocity also ensures that the bed silt in pocket will not get
stirred up and therefore will not find entry into the main canal.
As a further precaution against the entry of bed silt into the
main canal, head regulator gates are usually provided in 2 or
3 tiers where silt trouble is acute; such gates in tiers are also
called rising sill gates because the total height of each gate is
split up in 2 or 3 parts and therefore, the water from the
pocket can be fed into the main canal from various temporary
sill levels according to necessity, by proper manipulation
of these parts (i.e. by changing the temporary sill level of head
regulator). The 2 or 3 parts of a gate placed one behind
the other make up the whole height of gate and close the span
when necessary. By means of a rising sill gate, the surface
layer of water which is comparatively free from coarse silt
can be admitted into the main canal. The protective aprons
of head regulator are designed like those of main weir but the
uplift pressures are more in case of head regulator aprons
than in case of weir aprons; however, the lengths of head regu-
lator aprons will be less than those of weir aprons because
the velocity of retreat is comparatively small in case of head
regulator. To be on safe side, the formulae for their lengths
may be same as those for the lengths of weir aprons. The
maximum available feeding head across the head regulator
will be equal to pond level minus the full supply level of
main canal on downstream of the head regulator. Usually,
half of this available head is utilised for designing the waterway
of head regulator; such utilised head is called the working
head of head regulator. Some authorities give the following approximate rules for the design of aprons of head regulator:

Total length of upstream and downstream pucca floors is 12 and, 4H to 5H when C is 15.

Length of downstream talus = 4 to 5 times the full supply depth of main canal.

11. Silt regulation at head works: As the silt trouble is acute in alluvial soils, silt regulation works are provided at head works of irrigation scheme in alluvial area to regulate the entry of coarse bed-silt into the main canal. These methods of silt regulation consist of:

(a) Silt regulator which may be,
   (i) Silt excluder (see fig. 68) and/or
   (ii) Silt ejectors or silt extractors (see fig. 69).

(b) Raised crest wall at the face of head regulator.

(c) Head regulator gate in tiers.

(d) Still pond system of regulation.
(e) Low velocity of entry through the head regulator spans.

A silt regulator is a regulator provided with under-tunnels so arranged as to secure the escapage (or disposal) of the bottom layers of water charged with coarse silt. Before water passes through under-tunnels of silt regulator, the velocity of water is made to decrease and thus the lower layers of this water become concentrated with silt load.

Silt ejector (without roof slab)

Fig. 69

Such silt regulator provided in the pocket in front of head regulator is called silt excluder and those provided in the first few miles of main canal, one below the other, are called silt ejectors or silt extractors.

Silt excluder consists of a number of equidistant piers (of required height) parallel to head regulator face, the
pier being covered over by a R.C.C. slab to form under-tunnels; the top of slab is kept at the sill level of head regulator. The bottom water (charged with heavy and coarse bed silt) is made to pass out of the pocket through the weir scouring sluices via these under-tunnels. The water above the slab is comparatively free from coarse silt and is fed into the main canal. Where silt excluder is provided, the weir scouring sluices should be kept partially open so that the bottom water charged with bed silt may pass out through them into the river on downstream side.

Silt ejector consists of a number of equidistant piers, of required height, along the width of main canal; these piers curve through 90° and pass through one bank of main canal; they are covered over by a R.C.C. slab to form under-tunnels. If any bed silt finds entry through head regulator into the main canal, it is taken out of main canal via these under-tunnels into an artificially excavated lead-channel, leading from the ends of tunnels (under the bank) to the river on downstream of weir. The tunnels are controlled by gates at their exit. The comparatively clear water passes over slab to the canal on downstream side of ejector. Two to three such ejectors are arranged at about 300 m (1000 feet) centre to centre, one below the other in the head reach of main canal, the first silt ejector being 300 m from head regulator; a cross regulator is provided below the last silt ejector to ensure proper working of the ejectors. Ejectors are found to be more efficient than silt excluder and are also better than canal scouring sluices or silt traps which, in old days, used to be provided in the head reach of main canal to scour out unwanted coarse silt deposit from this reach. The canal section immediately upstream of silt ejector is made bigger than normal by lowering the bed level of canal; this reduces the velocity of water, resulting in the concentration of heavy silt in the bottom portion of canal. This heavy silt is then taken out of canal by the under-tunnels.

Items (b), (c) and (e) have been already treated in article 10 and item (d) has been treated in article 7. Still pond system of regulation is possible where there is no silt excluder in the pocket.
CHAPTER XI

IRRIGATION CHANNELS IN ALLUVIAL SOIL

1. Introductory: This chapter gives the description of irrigation channels in alluvial soil; main points treated about them are: the alignment, design principles and the silt problem.

2. Irrigation channels (or canal system): They consist of:
   (a) Main canal
   (b) Branch canal and/or distributary
   (c) Water course or Irrigator’s channel or Field channel.

The above classification of irrigation channels is according to their function; in any irrigation scheme, the discharge and size of irrigation channel decrease progressively from main canal to water course. Main canal or canals take off directly from the upstream side of weir headworks. There may be a number of branch canals taking off from each main canal and a number of distributaries from each branch canal; alternatively, there may be only distributaries taking off directly from each main canal or there may be a few branch canals and a few distributaries taking off from main canal. Main canals, branch canals and distributaries are called the government channels because they are owned, constructed, controlled and maintained by government. From distributaries, a number of water courses take off to carry water to the fields for irrigation purposes; ordinarily, no irrigation is practised directly from the government channels. The water courses are called the non-government channels because they are owned, constructed, controlled and maintained by the irrigators. The authority of government officials ends at the heads of water courses. Now-a-days, water courses are being constructed by government on behalf of the irrigators but the maintenance is entirely left to the irrigators.

3. Alignment or location or layout of irrigation channels: That layout of irrigation channels is the best
which ensures economy and, equitable and easy distribution of water by flow. The following main factors are therefore considered while fixing the alignment of irrigation channels:

(a) Channels should be taken by direct route to the land to be irrigated because this ensures less loss of head, less transit losses and over-all economy.

(b) Channels should be aligned on ridges wherever practicable because this ensures better command by channels and the construction of a few cross drainage works; where however the channels come across natural drainage lines, the necessary cross drainage works should be provided. A channel on ridge is called ridge channel. Main canal in alluvial area is always on main ridge for most of its length except (in some cases) its head reach which may be nearly parallel to contour of the ground. Such a channel in head reach is called contour channel; it irrigates on one side only. Main canal has, therefore, the commanded area on both of its sides. Branch canals and distributaries are aligned on minor ridges in the commanded area so that they also have their commanded area on both sides as they are mostly ridge channels. Water courses are usually aligned on the slopes of these minor ridges. Water course may be aligned on field boundary and should have straight alignment as far as possible. It may irrigate on one or both of its sides according to local contours.

(c) Channels should be central to their commanded area as far as practicable because this ensures better command by channels, less length of off-taking channels and, the resulting better distribution of water. Water courses, however, may preferably be along the upper boundary of the outlet area as far as possible.

(d) Channels should not pass through undesirable country which may consist of very hard soil, very unstable and pervious soil, alkaline soil, water-logged soil, costly inhabited land, etc. because, in such cases, the channels will be costly and/or will not work efficiently.

(e) There should be very few curves in the alignment of channels and, where they are unavoidable, they should be smooth (i.e. of large radius) because channels have the
tendency to silt and scour at sharp curves. A rough rule for radii of the curves is as follows:

'For large channels, the minimum radius of curve should be 60 times the bed width of the channel and for small channels, the minimum radius of curve should be 45 times the bed width of the channel'.

(f) Alignment should be such as to avoid deep cuttings and high embankments as far as practicable because this ensures economy.

(g) Length of water course should be about 3·2 to 4·8 km (2 to 3 miles) and hence, the distributary should stop short of the boundary of the commanded area by 2 to 3 miles.

Note: Sometimes, a channel may be aligned as a side-slope channel; such a channel is roughly at right angles to the contours but it is neither on ridge nor in valley. It is parallel to the natural drainage line.

4. Full supply discharge of irrigation channels: The maximum discharge that an irrigation channel normally carries at its head to satisfy the irrigation requirements of its commanded area, under worst conditions during any part of year, is called its designed (or authorized) full supply discharge (i.e. F.S.Q.) or capacity; the maximum level of water in the irrigation channel when this F.S.Q. is flowing in the channel is called its full supply level (i.e. F.S.L.). The depth of water from F.S.L. to canal bed level (i.e. C.B.L.) is called its full supply depth (i.e. F.S.D.). After the final layout of all channels is fixed on the contour map, the area proposed to be irrigated by each channel is marked on the plan; the final design of F.S.Q. at head of each channel is worked out, beginning from tail of channel upwards. Thus, first the F.S.Q. at the head of each water course is worked out knowing the area to be actually irrigated by each water course, the crops to be raised on this area, the (net) duties of water on the field for these crops and the losses of water while the water travels from the head of water course to the fields; instead of net duty on field and the losses, the duty at the head of water course, if given, can be utilized for working out F.S.Q. at the head of water course. Next, the design of F.S.Q. at the head of distributary (from which the above-
mentioned water courses take off) is taken in hand, again beginning from tail of the distributary upwards. Then, the design of F.S.Q. at the head of branch canal (if any), from which above-mentioned distributaries take off, is taken in hand, again beginning from the tail of the branch canal upwards. And finally, the design of F.S.Q. at the head of main canal is worked out on the same principles i.e. beginning from tail upwards. While finding the F.S.Q. of distributary, branch canal and main canal, the transit losses (i.e. the losses due to evaporation and absorption while the water travels in these channels) are allowed for. The rough idea of F.S.Q. at the head of a distributary can be had by knowing the area to be irrigated by the distributary water, the crops to be raised on this area and the duties of these crops at the head of distributary. Similarly, the rough idea of F.S.Q. at the head of a branch canal or main canal can be formed but the final design is always worked out as already shown above. Irrigation water, however, does not usually flow in irrigation channels throughout the crop period of crops but it is allowed on the intermittent system i.e. there is water in a channel for some days and then the channel is dry for a few days; this becomes necessary when the supply of water is short but it also ensures the economy of water even otherwise. The ratio of the total number of days for which a channel is in flow to the days of crop period is called the time factor and this is taken into account in addition to the irrigated area, crops, their duties and the transit losses, to find out the F.S.Q. at the head of each channel. Evidently,

\[
\text{F.S.Q. on intermittent system} = \frac{\text{F.S.Q. on continuous system}}{\text{time factor}}
\]

Usually the value of time factor is from 0.5 to 0.7 i.e. \(\frac{5}{10}\) to \(\frac{7}{10}\). The F.S.Q. of a channel (main canal, branch canal or distributary) will go on progressively decreasing from its head to its tail because the offtaking channels (e.g. branch canals, distributaries or water courses) will be taking off from it at various points along its length; hence its size (i.e. width and depth) will also go on progressively decreasing from its head to its tail. At tail of each channel, there should be a F.S.Q. of about 57 to 85 litres/sec (2 to 3 cusecs)
from practical considerations of maintenance etc. of the channel. In case of water course, however, the size should be uniform from its head to its tail as a water course has to carry its F.S.Q. from its head to its tail.

The F.S.L. of a channel at any point along its length should be such that the channel can command the area settled on it by flow as far as practicable; also, it should be able to feed its offtaking channels with their respective full supply discharges. Further, if practicable, during construction it may give the balanced cut and fill for the channel as this will ensure economy. The F.S.L. may be near the ground level for safety and may be slightly (say 0·3 m to 0·6 m or 1' to 2') above it, if possible.

It may be noted that each channel has certain area in its charge which may be cultivated: such area is called the gross commanded area (i.e. G.C.A.); gross commanded area in the charge of water course is called outlet area, or chak of outlet. A portion of this area is occupied by villages, roads, buildings and other unculturable land like alkaline, water-logged or barren lands. The difference between G.C.A. and this unculturable land is called the culturable commanded area i.e. that area which can be cultivated. A portion of this is occupied by high patches of land; the balance portion requires irrigation water and such balance portion is called the culturable irrigable area (i.e. C.I.A.). Usually a certain percentage of the C.I.A. is actually irrigated annually and the remaining land lies at fallow; the ratio of the actually irrigated area to the C.I.A. is called the intensity of irrigation; intensity of irrigation is different on different canal systems and is different on the same canal in different seasons. The annual intensity of irrigation on a canal is the sum of the seasonal intensities of irrigation. Intensity of irrigation is less where there is the danger of soil becoming water-logged.

Note: A certain portion of annually irrigated area is under Kharif crops in Kharif season and a particular portion is under Rabi crops in Rabi season. The ratio of the areas under crops in different seasons is known as crop ratio. This crop ratio should be so fixed in case of perennial canals that the discharge of the canal is uniform in all seasons.

5. Losses by evaporation and seepage in canals: While irrigation water flows in irrigation channels, there
are losses of water due to evaporation from water surface and absorption (or seepage) through the bed and sides of the channel; such losses are called **transit** losses or canal losses; in case of canals, the loss due to absorption is more than that due to evaporation. In alluvial canals these combined losses are allowed at the rate of 2.4 to 3 cusecs per million sq. metres (8 to 10 cusecs per $10^6$ sq. feet) of wetted surface of the channel. Loss of water in water courses is more significant. Loss can be reduced to some extent by proper alignment, design, construction and maintenance of the channels. Between the head of main canal and the field, the losses may be 10 to 40% of the discharge at the head of main canal.

6. **Design of irrigation channels**: Design of irrigation channel consists in finding its various elements e.g., bed width $B$, full supply depth $D$, bed slope $i$ (or $S$) and the mean velocity $v$ of water in it; also, the side slopes of cuttings and embankments, top width of embankments, width of inside berms and the extent of free board have to be settled.

Velocity of water should be non-silting and non-scouring i.e. channel should neither silt nor scour. Non-silting velocity in alluvial soil is *practically* the same as the non-scouring velocity; channels in alluvial soil should, at any rate, be non-silting. Minimum non-silting velocity usually determines the bed slope of a channel. The approximate non-silting and non-scouring velocity for light sand is 0.45 to 0.6 m/sec (1.5 to 2 ft/sec), for sand-loam 0.75 m/sec (2.5 ft/sec) and for soft clay about 0.9 m/sec (3 ft/sec). Velocity in alluvial soils depends on the hydraulic mean depth (i.e. H.M.D.) of a channel and the soil through which the channel passes. Since the velocity depends on hydraulic mean depth of channel, more velocity will be given in big size channel and less in small size channel. Thus, velocity in water course may be 0.15 to 0.3 m/sec ($\frac{1}{2}$ to 1 ft/sec) and that in main canal may be 0.9 to 1.1 m/sec (3 to 3.5 ft/sec). In branches and distributaries, velocity may be within these two limits. When the velocity is excessive, it causes erosion and the consequent lowering of the channel bed. The lowering of bed levels of a channel due to bed erosion is called **retrogression of levels**; the retrogression of levels causes fall in
water levels (i.e. specific levels), resulting in the loss of command by the channel.

Bed slope is inter-connected with the non-silting and non-scouring velocity. It depends on the capacity or discharge of a channel and the nature of soil through which the channel passes. Greater the discharge, flatter the slope; harder the soil, steeper the slope. A slope flatter than one actually required, causes silting and conversely the steeper slope causes erosion of the bed and sides of channel.

The suitable $B$ and $D$ of channel are calculated, knowing the F.S.Q., the nature of soil through which channel passes and the silt characteristics of water in the channel. The channels in alluvial soil are as a rule shallower i.e. their $B/D$ ratio has greater value; $B/D$ ratio provided in a channel depends mainly on F.S.Q. and the silt characteristics of the water; greater the discharge, greater the $B/D$ ratio. Design of the elements of a channel also proceeds from the tail upwards; thus $B$ and $D$ of each water course are first calculated; next, the distributary design is taken up. Whole distributary is divided into suitable reaches of the same cross section area. In each reach, the design elements are the same because discharge is same in the whole reach; the design starts with the tail reach and is carried progressively up, towards the head of the distributary. The change in the F.S.D. of the neighbouring reaches is effected (by a rise in bed at the junction of two reaches) to the extent of 3 cm (0.1 foot) only; more change than 3 cm between the two neighbouring reaches can be effected, with safety, only when there is some irrigation structure like cross regulator or canal fall etc. between the two reaches; at such structures, usually, a drop in F.S.L. and bed level is effected. Change in the bed width of the two reaches may be 0.15 to 0.6 m ($\frac{1}{4}$' to 2') or so; greater change than this can be effected at irrigation structures. Distributary bed levels at various points along its length are worked out by subtracting the F.S.D. from the F.S.L. at those points. Similarly the $B$ and $D$ of branch canal are worked out (from tail upwards) in the various reaches of the branch canal; and finally, same process is repeated for the main canal. Every channel should be of such design that it should be able to carry its due share of water and
silt-load without showing signs of silting or scouring at any point along its length. Side slopes of a channel should be such as can stand in the presence of water; harder the soil, steeper the side slopes. Suitable side slopes are as follows:

- Light sand ........ 2:1 to 3:1
- Sandy-loam or, soft clay . 1½:1
- Hard clay ............... 1:1

*Free board* is the vertical distance between the F.S.L. of a channel and the top of the channel banks. Free board is about 15 cm (6") for water course and 0.45 m to 1.2 m (1½' to 4') or more for other bigger channel; greater the size of a channel, greater the free board. For government channels free board should be at least equal to \( \left( \frac{F.S.D.}{10} + 1 \right) \) feet.

A strip of land between the upper edges of the government channel section and the water-side toe of the channel bank is kept on each side of the channel; such a strip is called *inside berm*. It is kept so that the channel can be widened in future, if necessary, without removing the banks; also, the material of the banks, when it slips, will not fall in the channel but will come down on the inside berm from where it can be removed. When the F.S.L. is above the ground level, these inside berms are formed by inducing fine silt deposit and then they are called the inside silted berms. Width of inside berm may be 2 to 4 times the F.S.D. of channel and the top of berm may be 0.15 m to 0.3 m (½' to 1') above the F.S.L. of channel.

Top of one bank of a government channel is made wide enough to carry vehicular traffic and it is called *inspection path*; inspection path may also be on the inside berm. The other bank called *non-inspection path* should be of such cross section area as to be just stable. Banks should cover the line of saturation. High bank is made more stable and also economical by providing back berms on its land-side slope at one or more places along its height. The slope of bank on water side is made flatter than the slope on land side; the slopes given to bank depend on the nature of material of bank; more stable the material, steeper the slope and vice versa.
The material realized from the excavation of a channel is utilized in making the channel banks; where, however, the excavation is less than the bankwork or the channel is wholly in bank, the extra earth required to form the banks is taken from the inside *borrow pits* excavated in channel bed or in the inside berms. These pits get silted up after some time when the channel begins flowing. In some cases, the outside borrow pits are excavated on land, slightly away from the land-side toe of the channel bank. Where excavation is more than bankwork, the extra earth is dumped in the form of a bank parallel to the canal bank and, of height equal to that of the canal bank; such a bank is called *spoil bank*.

![Diagram of irrigation channel](image)

*Fig. 70(a)*

Channel cross section, wholly in cutting

*Fig. 70(b)*

The width of land occupied by channel section including its inside berms, banks, outside borrow pits if any, spoil banks if any, is called *canal land-width*. This width is demarcated by canal *boundary stones* fixed on both sides of channel, at intervals, in the longitudinal direction of the channel. If canal passes through private land, so much land-width shall have to be purchased or acquired by the government from the owner of this land. The width of land to be acquired is rounded so as to be the exact multiple of 3.3 m (11 feet); by so doing, the area of land can be *easily* calculated in acres.

After fixing the elements of a channel as shown above, the longitudinal section and cross sections of the channel
are drawn and the estimates are prepared. The typical L-section of a distributary is shown in fig. 70(a).

The channel cross section may be:

(a) Wholly in cutting.
(b) Partly in cutting and partly in bank.
(c) Wholly in bank.

Channel cross section partly in cutting and partly in bank
Fig. 70(c)

Channel cross section wholly in bank
Fig. 70(d)

Typical cross sections of the above-mentioned three types of channels in alluvial soil are shown in figs. 70(b), 70(c) and 70(d).

7. Kennedy’s and Lacey’s silt theories: These two theories are used for the design of channels in alluvial soil. The authors of these theories claim that a hannel designed according to their theories will be stable. A stable channel may be defined as a channel which is more or less steady with regard to its cross section and course. These theories, however, have not fully solved the problem of the design of channels in alluvial soil and give only nearest approach to this problem.

R. G. Kennedy was Executive Engineer in Punjab P.W.D. and while taking the statistics on Bari Doab canal system (in Punjab), he found this canal system to be in regime i.e. the channels of this system were, on the whole, neither silting nor scouring. On studying these regime channels, he found that the silt transporting power of a
channel depends on the velocity of water and the depth of flow in the channel and he gave the following empirical formula for the desirable velocity of water in any alluvial channel:

\[ v_c = C D^n \text{ ft/sec} \]

where, \( v_c \) = critical velocity or non-silting and non-scouring velocity i.e. that velocity which ensures no net silting or net scouring in a channel.

\( C \) := constant which depends on the nature and the charge of silt, held in suspension, in a channel; value of \( C \) is greater for coarser silt; thus, \( C \) is 0.84 for some Punjab canals and 0.63 for some Sind canals.

\( n := \) a constant; its approximate value is 0.64.

\( D = \) F.S.D. of the channel.

Mean velocity \( v \), given to water in a channel, should be equal to or greater than \( v_c \). The ratio \( \frac{v}{v_c} \) is known as critical velocity ratio. Its value depends on the soil in the bed and sides of the channel. Kennedy claimed that if alluvial canals be designed according to his theory, silt will always be kept in suspension in the government canals and it will get deposited on the fields or in the field channels. Kennedy's theory has not been found to give much satisfaction.

G. Lacey, who was Superintending Engineer in Uttar Pradesh P.W.D., studied a number of regime canals (in alluvial soil) in the world and gave his own silt theory. According to him, the velocity of water in a channel does not depend on \( D \) but on the hydraulic mean depth \( R \) of the channel. He claimed that a channel designed on Kennedy's silt theory is in initial (and not final) regime and, it simply transfers the trouble of silt-deposit and scour from one point of a channel to the other. Lacey showed that for a channel in an incoherent alluvium (like pure sand), \( B, D, S \) of the channel carrying a given F.S.Q. and given silt-charge and silt-content, are uniquely fixed if the channel is to be in final regime. He also showed that \( v, R, P_w \) (i.e. wetted perimeter), F.S.Q., \( S \) and \( N \) (i.e. rugosity factor) of a regime channel are closely related to one another.
The fundamental equations given by him for the design of regime channel in pure sand or gravel are:

\[ v_m = 1.17 \cdot f^{1/2} R^{1/2} \text{ ft/sec} \]  \hspace{1cm} (i)

\[ A f^2 = 4.0 \cdot v_m^5 \]  \hspace{1cm} (ii)

\[ R^{1/2} S = 0.0009759 f^{3/2} \]  \hspace{1cm} (iii)

where, \( v_m \) = mean velocity of water in a channel, in feet.

\( R \) = hydraulic mean depth of the channel.

\( A \) = waterway of the channel, in sq. ft.

\( f \) = a constant, known as Lacey's number or Lacey's silt-factor and, it depends on the nature, grade, charge and other attributes of silt held in suspension by water in the channel.

\( S \) = bed slope of the channel.

From these fundamental equations, he derived the following equations:

\[ P_w = 2.67 Q^{1/2} \text{ feet} \]  \hspace{1cm} (iv)

\[ S = \frac{f^{5/3}}{1844 Q^{1/6}} \]  \hspace{1cm} (v)

\[ A = 1.26 \left( \frac{Q}{f^{2/5}} \right)^{5/6} \text{ sq. feet} \]  \hspace{1cm} (vi)

\[ R = 0.47247 \left( \frac{Q}{f} \right)^{1/3} \text{ feet} \]  \hspace{1cm} (vii)

\[ v_m = 0.7937 Q^{1/6} f^{1/3} \text{ ft/sec} \]  \hspace{1cm} (viii)

where, \( P_w \) = wetted perimeter of the channel, in feet.

\( Q \) = F.S.Q. of the channel, in cusecs.

In actual practice, in alluvial area, purely incoherent alluvium does not exist; hence, Lacey's silt theory is used with some modifications as required by the attributes of the local soil and the silt-in-suspension.

When \( S \) found by formula (v) is not naturally available on the site, heavy bed silt is excluded by silt regulators to change the value of \( f \) to the required figure.

It may be noted that according to Lacey's theory, \( B/D \) ratio affects the silt bearing capacity of a channel.
8. Silting and Scour: Water of a given velocity and depth of flow in a channel can carry, in suspension, only a certain amount of silt of certain nature. Silt charge and its coarseness increase from water surface to the bed of a channel. 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 and sides of channel till it is fully charged with silt; also the increase of velocity or, the change of direction of stream causes scour; if on the other hand, the velocity of water is reduced, a certain portion of silt formerly carried in suspension will settle down and will deposit on the bed and sides of the channel. Silting and scour in channel are undesirable and should be avoided by proper design of the channel. Scour causes the loss of command (due to the fall of F.S.L.), the breaching of canal banks and the failure of foundations of irrigation structures. Silting interferes with the proper working of a channel as the channel section gets reduced by silting.

Silt problem is serious in alluvial soils. Silt consists of:

(a) *Light fine silt* like clay, fine sand and silt particles; this is usually held in suspension.

(b) *Heavy coarse silt* like coarse sand and gravel; this moves near the bed of a channel and is undesirable in a channel as it gives trouble by being deposited in the channel.

Silt in a channel may be:

(i) That received from the source or parent channel; it is called original silt.

(ii) That which is the result of scour of bed and sides of the channel; it is called *local silt* or derived silt.

Precautions taken against silting or sedimentation in canals consist of:

(a) Silt regulation at the head works across the river. This has been already described in chapter x.

(b) Keeping out the unwanted heavy bed-silt by suitable design of the head regulator of branch canal or distributary. The design of head regulators and irrigation outlets (at heads of water courses) should be such that they
pass the due share of only silt-in-suspension into the channels controlled by them.

(c) Designing the channel-section suitably so that it neither silts nor scours. Ideal condition in this respect is that whatever silt-in-suspension has entered the off-taking channel from its parent channel, should be kept in suspension so that it does not settle and deposit at any point of the channel; also, the velocity of water should be such that it does not produce local silt by erosion of the bed and sides of channel.

9. Canal Lining: Ordinarily out of every 100 cumeecs released at the head of main canal in alluvial area, about 25 cumeecs are lost in main canal, branch canal and distributaries; about 20 cumeecs are lost in water courses and only 55 cumeecs reach the agricultural fields. Taking about 15 cumeecs loss on the fields, only 40 cumeecs are available to be used actually by the crops. If the loss of water in channels due to absorption and percolation is still more serious, it may cause much loss of valuable irrigation water and also produce water-logging; irrigation channel may be lined to stop this loss of water if the lining be found economical, looking to the results of lining. In India, where the channels are of great magnitude and the irrigation water rates are low, lining with superior and costly material is generally not an economically feasible proposition; however in some cases, where it pays to line a channel, the channel is lined. In India, the usual way to stop excessive percolation (i.e. secure staunchness of channel bed and the sides of channel banks) is to excavate the channel of a section greater than the required one and then get it artificially silted with light and fine silt to the required section; such silt deposit on the bed and sides makes the channel staunch. The lining of water courses in this manner will pay more dividends than the lining of main canal, branch canal and distributaries.

Lining is simply a protective covering of impermeable material over the wetted perimeter of a channel and slightly above F.S.L., as shown in fig. 71; it is mainly used to prevent the seepage but, in some cases, it is also used to improve the command of a channel in flat country or to cut down the cost
of excavation through hard rocky strata in uneven country or to increase the discharge capacity of channel. Lining may be of:

(a) 6 cm to 15 cm (2½" to 6") thick layer of plain concrete 1:2:4 or 1:3:6, as required. Reinforcement is necessary in bad soils. Reinforced concrete lining may be 3½" to 4½" thick. Precast cement concrete tiles are also used as canal lining; the tiles may be 1½" to 2½" thick.

(b) 5 cm (2") thick layer of cement mortar.

(c) Oiled paper.

(d) 9 cm (3½") thick layer of puddled clay.

(e) Stone masonry of suitable thickness.

(f) Brick masonry of suitable thickness.

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**Fig. 71**

Cross section of lined canal

Lining should be strong, durable and watertight. Lining of concrete or cement mortar is more strong and durable and it is therefore more common.

To find the velocity of water in a lined channel either Chezy or Manning formula may be used. The rugosity factor \( N \) to be used in Manning formula is as shown below when the channel is in average condition:

<table>
<thead>
<tr>
<th>Material of lining</th>
<th>Value of ( N )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete in situ</td>
<td>0.016</td>
</tr>
<tr>
<td>Precast cement concrete tiles</td>
<td>0.011</td>
</tr>
<tr>
<td>Cement plaster</td>
<td>0.013</td>
</tr>
<tr>
<td>Brick masonry in cement mortar</td>
<td>0.015</td>
</tr>
</tbody>
</table>

**Note:**

(i) Value of \( N \) for earthen channel in average condition is taken as 0.025.

(ii) Higher velocities can be given to the water flowing in lined channels. Thus, for a given discharge, cross section area of a lined channel will be smaller as compared to that of an unlined channel. A velocity of about 8 ft/sec (i.e. 2.5 m/sec) in a concrete-lined channel is permissible.
10. **Strengthening of the weak canal banks by inducing fine silt deposit:** This becomes necessary when a big channel is wholly in high bank and the material of banks is permeable and soft so that the banks are liable to give way by breaching. There are three systems of strengthening the canal banks by silting; they are:

(a) In and out system: see fig. 72.
(b) Long reach system:
(c) Internal silting system: see fig. 73.

![Diagram of canal bank strengthening](image)

The first two methods are useful for existing canals and the last one for newly constructed canals. Silting process can be continued throughout the year.

In the 'in and out' system, a secondary bank is constructed parallel to the canal bank (called primary bank) and at certain distance away from it. The two banks (primary and secondary) are connected by cross banks at about 150 m (500') centre to centre from one another to form the compartments.
An inlet-cut through primary bank is given to allow water from canal into the compartment; the fine silt brought in suspension by this water gets deposited in the compartment and comparatively silt-free water leaves the compartment by another outlet-cut (in the primary bank) located at the end of the compartment. A number (usually four) of inlet-cuts one after the other becomes necessary as the previous cut gets silted up after some time and therefore proves useless. When the compartment gets silted up, the secondary bank may be destroyed and its material used to staunch the canal bank further, at its back (i.e. on its land side).

![Internal silting system](Fig. 73)

In 'long reach' system, all the water of the canal is diverted into the compartment, at a point of canal near the beginning of the compartment; this water flows through the compartment and is taken back into the canal at the tail of the compartment; thus no water flows in a reach of canal parallel to the compartment and of length equal to the length of the compartment. This system is not so efficient as the first one.

'Internal silting' system is for making the banks staunch (on water side) with silted berms formed by inducing the deposit of fine silt. Brushwood groynes are constructed (projecting from the canal banks to the water side) of length equal to the width of the inside silted-berm to be formed; the groynes are constructed at certain distance centre to centre along the length of canal; this distance is usually equal to 2-3 times the length of groyne. The top of groynes will be upto F.S.L. of the canal. Due to obstruction to the flowing water caused by these groynes and the subsequent reduction
in velocity of water, the fine silt gets deposited in space between the two adjoining groynes; after some time, inside silted berms (with top at F.S.L. of the canal) are obtained. This method is cheap and hence, very common.

11. **Aquatic weeds**: Vegetal growth that thrives even when it is constantly under water is called aquatic weed or water-loving plant; thus, the vegetation on bed and sides of an irrigation channel is called ‘weed’. Where weed growth is abundant, the canal section gets choked and the canal does not function properly. These weeds, like other plants, require sunshine for their growth; because, the irrigation water in alluvial soil is generally so turbid due to silt charge in suspension that the sunshine cannot penetrate through it and reach the weeds, the nuisance of weed-growth is not so acute in alluvial canals; where, however, the weeds get sunshine, they grow luxuriantly. Where they grow, they have to be removed periodically.

As already said in the paragraph above, the weed-growth chokes the canal section. It must therefore be removed as there is no good point or virtue about a weed. Emerson was correct when he defined weed as ‘a plant the virtue of which has not been discovered’.
CHAPTER XII

CANAL MASONRY WORKS IN ALLUVIAL SOIL

1. Introductory: This chapter deals with the canal masonry works on an irrigation channel (i.e. main canal, branch canal or, distributary). The canal masonry works treated here will mainly consist of:

(a) Cross drainage works

(b) Regulation works.

It is not proposed to describe communication works here as they could be found in any standard book on bridge engineering. The navigation works have been briefly described in chapter xxi.

2. Cross drainage works: Where an irrigation channel comes across a natural drainage line (e.g. stream or river), a masonry work known as cross drainage work is provided for the safe disposal of drainage water across the channel. Following are the types of cross drainage works:

- **Aqueduct**
- **Super-passage**
- **Level crossing**

![Diagram](image)

**Fig. 74(a)**  **Fig. 74(b)**  **Fig. 74(c)**

(a) Aqueduct: see line plan, fig. 74(a).

(b) Superpassage: see line plan, fig. 74(b).

(c) Level crossing: see line plan, fig. 74(c)

(d) Inlet and outlet.
The type of cross drainage work to be adopted depends on:

(i) Relative bed levels of irrigation channel and natural drainage.

(ii) Relative volumes of water of both channels.

(iii) Overall economy of construction.

The crossing of two waters should be normal to each other and preferably at a point where the banks of drainage are high and stable and where good foundations for the cross drainage work can be had.

3. **Aqueduct**: When drainage water at the point of crossing goes under irrigation water, the cross drainage work is called an aqueduct. Following are the forms or sub-types of aqueduct:

(i) Masonry aqueduct: (fig. 75).

(ii) Irrigation culvert or Culvert under irrigation channel: (fig. 76).

(iii) Irrigation slab drain or Irrigation box culvert: (fig. 77).

(iv) Irrigation pipe-aqueduct: (fig. 78).

(v) Syphon aqueduct or Masonry syphon or Drainage syphon.

(vi) Irrigation syphon culvert (fig. 79) or Syphon slab drain.

In case of masonry aqueduct, the drainage water passes through bridge-like spans (formed by a number of piers and their roof arches) constructed along the width of river; the roof of these spans carries on its top a rectangular masonry trough or chute, from one bank of river to the other, through which the irrigation water of canal flows at the point of crossing; there should be sufficient clear headway between the H.F.L. of drainage and the underside of roof-arches. Usually a roadway is provided parallel to the chute and in continuation of the inspection path of the canal. The wing walls are provided for smooth entry and exit of drainage water and irrigation water at the point of crossing. The foundations of piers and abutments etc. of the structure portion,
through which the drainage water passes, should be below
the depth of maximum scour as given by Lacey because the
rocky natural foundation is rare in alluvial soil; usually, only
impervious floor is provided between piers and also slightly
on the upstream and downstream sides of the piers and, no
other extensive protective aprons on upstream and down-
stream sides of this floor are necessary as there is no appre-
ciable cutoff across the structure. Masonry aqueduct is

![Diagram of masonry aqueduct]

constructed in preference to other sub-types when the width
of drainage is more, say above 15 m (50') or so. For drainage
water to pass, the width between abutments should be
practically equal to that of the natural drainage so that the
velocity of water through the structure may be same as that
in the natural drainage; the span between piers is usually
3.6 m to 9 m (12' to 30'). When span is greater than 15 m (50'),
arch type is replaced by R.C.C. slab and beam type aqueduct.
The width of trough may, however, be made less than the bed

**Fig. 75**

**Masonry aqueduct**
width of irrigation channel if such a procedure ensures economy; in such case, we say that the irrigation water has been flumed (i.e. width has been made narrow) at the crossing. Pitching of canal bed and sides is ordinarily provided on the upstream and downstream sides of the canal trough; but where the trough is flumed, greater protection of downstream canal bed, against scour, is necessary because the velocity of water coming out of flume will be more than that of water in the irrigation channel on its downstream side.

Irrigation culvert

Fig. 76

When the width of natural drainage is from 2.4 m to 15 m (8' to 50'), irrigation culvert may be constructed with spans of drainage water about 1.8 m to 3 m (6' to 10') each. The piers and roof arches for drainage water will be as in a masonry aqueduct; however, the canal section over the roof of drainage will be the same as it is on upstream and downstream sides of crossing i.e. the canal banks are continued wholly or partially, from upstream side, over the drainage roof and the irrigation water flows between these banks instead of the masonry trough walls.

When the width of natural drainage is less than 2.4 m (8'), slab drain is constructed. Slab drain consists of one or two
spans with R.C.C. (or stone) slab-roof instead of arch-roof. Like irrigation culvert, the canal banks from upstream side are continued, over the slab drain spans, from one bank of drainage to the other.

When irrigation channel is very small, irrigation water may be taken in a pipe over the drainage, as shown in fig. 78.

In syphon aqueduct, the drainage is taken under irrigation channel by means of inverted syphon masonry barrels consisting of piers and roof arches. It is usually used for thus syphoning the small drainages. The barrels (between piers) of inverted syphon through which the drainage water passes, should have slopes at their ends to join the horizontal bottom of barrels with the drainage bed on each side; this is necessary to avoid the likelihood of heavy silt deposit in the barrels of syphon. To avoid the likely choking of barrels with the
floating debris, a minimum of 0.9 m (3') headway is kept between the bed of drainage and the intrados of the barrel-arch. As barrels run full, the barrel-roof should be strong enough to take up bursting pressures caused by the difference of water levels of drainage on upstream and downstream sides of the barrels; the modern practice is to have barrel-roof of R.C.C. slab as it is economical. Velocity of water through the barrels should be self-cleansing to avoid silt deposit in the barrels. The drainage bed and sides on the downstream side of barrels should be pitched with stone. A roadway is provided parallel to the canal trough.

In syphon culvert or syphon drain, the drainage is syphoned as in syphon-aqueduct but the drainage must be much smaller; as drainage water level is above the roof of barrels, canal banks over the barrels are partly replaced by masonry retaining walls at their outer toes. Thus the water on upstream and downstream sides of the barrels is in contact with the retaining walls and not with the earthen canal banks. The general principles of design of barrels etc. are the same as in case of syphon aqueduct. Syphon drain may be slab drain or pipe drain; in case of pipe syphon drain, the pipes should be strong enough to stand the likely bursting pressure.

Syphoned waterway is undesirable and should be avoided where it can be safely and economically done so.

4. Superpassage: When drainage water at the point of crossing goes over an irrigation channel, the cross drainage work is called a superpassage. Following are the sub-types or forms of superpassage:
(i) Superpassage: (see fig. 80).
(ii) Syphon superpassage or Canal syphon or Inverted syphon carrying irrigation water.

Superpassage is like masonry aqueduct, with this difference that the irrigation water passes below and, the drainage water passes through the masonry trough above. In many cases, a roadway parallel to the trough is, however, not provided. Also, the trough is never flumed i.e. its
width is kept equal to the bed width of the drainage on its upstream side. The lower waterway (i.e. canal waterway) may be flumed, if desired. There should be sufficient headway between the F.S.L. of canal and the underside of canal roof. It may be noted that, like pipe aqueduct, there may be pipe superpassage when the drainage to be taken over the canal is very small.

Syphon superpassage is like syphon aqueduct, with this difference that an irrigation channel is syphoned instead of a drainage. Here also, a roadway parallel to trough is not usually provided and the drainage trough is never flumed. For passing the canal water, R.C.C. pipes (instead of masonry barrels) may be used as they can stand the bursting pressures better.

Compared to aqueduct, superpassage as a cross drainage work is inferior and should be avoided where practicable; to choose between the superpassage and canal syphon, the former is preferable.

5. Level crossing: When the bed levels of two streams (viz., natural drainage and irrigation water) are practically at the same level, both waters are practically of

![Diagram of level crossing](image)

Level crossing
Fig. 81

the same magnitude and the flood duration of drainage is short, the drainage water is taken through irrigation channel, in some cases, for economy; such a cross drainage work is
called a level crossing because the waters of the two streams are inter-mixed in the first instance. A regulator is provided across canal just on the downstream side of crossing and another regulator is provided across the drainage just on the downstream side of the crossing, as shown in fig. 81. Necessary protection of bed, against scour, is provided on the downstream side of each regulator; also canal bed and sides, just upstream of the crossing, are pitched for some distance. Ordinarily, when there are no floods, the regulator across the drainage remains closed; during floods, however, the drainage water is taken out of this regulator and during that time the regulator across the canal is closed to prevent the unwanted water and silt from getting into the canal section downstream of the crossing. Instead of having regulator spans with lift gates across the drainage, there may be falling shutters with their top at F.S.L. of the irrigation channel.

6. Inlets and outlets: Inlet (fig. 82) is a cross drainage work consisting of an open cut in a canal bank, suitably protected by pitching, to admit the upland drainage

![Surface inlet](Fig. 82(a))

water into the canal. Canal bed and sides, for some distance on upstream and downstream sides of the inlet, are also pitched. Inlet is feasible when the drainage is small, its water is comparatively silt-free, the bed of drainage is at or slightly below the F.S.L. of irrigation channel and it does
not warrant the construction of any other superior type of cross drainage work. The drainage water allowed in the canal is taken out of the canal through a surface-outlet located in the canal bank opposite the inlet or, through existing surplus escape located on the canal, slightly on the downstream side of the inlet. When the bed level of drainage is at F.S.L. of canal, the cross drainage work is called a flush inlet; when bed level is slightly lower than F.S.L., a low weir-wall is constructed across the open cut, with top of weir-wall at F.S.L. of canal; drainage water will spill over this wall into the canal.

Surface outlet is another open cut in the bank of canal, with bed and sides of the cut properly pitched. It may be flush escape type or, low weir type with top of weir-wall at F.S.L. of canal; in some cases, it may also be regulator-type with the sill of regulator sluices below the F.S.L. of canal. Water from surface outlet is taken away, by lead channel, to near-by drainage on the downstream side of the surface outlet.

When drainage water is allowed to enter an irrigation channel, the discharge carried by the irrigation channel then is called its maximum supply discharge.

Level crossing, inlet, and surface outlet are inferior to superpassage and aqueduct but they are cheaper. Generally, aqueduct and superpassage are constructed when H.F.Q. is large and continues for some time; level crossing is constructed when H.F.Q. is large but short-lived and the floods are intermittent and not continuous. Inlet and outlet are constructed when H.F.Q. is small.

7. Regulation works: These are irrigation structures provided on an irrigation channel and are necessary for the efficient working and safety of the irrigation channel. They may mainly be:

(a) Head regulator or Head sluice.

(b) Cross regulator.

(c) Canal fall or Canal drop.
(d) Canal escape or Canal waste-way; this may be:
   (i) Canal scouring escape or Silt escape.
   (ii) Surplus escape.
   (iii) Tail escape.

(c) Irrigation outlet.

(f) Venturi meter.

(g) Venturi flume.

8. **Head regulator**: An irrigation structure at the head of an irrigation channel (viz. main canal, branch canal or distributary) is called a head regulator. The head regulator of main canal has already been described in chapter X. Head regulator at the head of branch canal or distributary essentially consists of a number of piers (along bed width of channel) with roof-arches to carry a road bridge for traffic and also, in some cases, a gate bridge for the manipulation of gates which move between piers as shown in fig. 83(a).

![Head regulator of a distributary](image)

**Fig. 83(a)**

From the top of regulator sluices, starts a head wall with its top above the F.S.L. of parent channel (i.e. main canal or branch canal). The function of this regulator is to pass into the channel the required quantity of water, at required level, as and when necessary. Its design should be such as to allow into the channel its due share of silt-in-suspension and prevent the entry of bed-silt into the channel. **General** principles of design remain the same as for head regulator of main canal; however, the spans will be comparatively smaller. Regulator sill is usually fixed at the bed level of
parent channel on its upstream side. The spans are controlled by vertical lift gates or radial gates and, in the case of some unimportant regulators, by needles or baulks; baulk type control is, however, undesirable as it interferes with the entry of due share of silt into the channel. The design of ventway (i.e. waterway) of regulator should be such that with head (across the regulator) less by 15-30 cm (6"-12") than the available one, the F.S.Q. of channel can be passed through the ventway into the channel; usual velocity of entry of water through head regulator spans is about 0.9 m/sec (3 ft/sec) or less.

The modern practice is to construct the head regulator of branch canal or distributary as a flumed head regulator because it is usually economical. The flumed structure however entails greater loss of head which should be available at the site. A flumed regulator [see fig. 83(b), plate I] consists of piers and arch-roof as in case of a full-width (i.e. unflumed) regulator but the ventway is constricted or narrowed; this portion of narrow width (of regulator) is called throat; the throat has got converging masonry channel on its upstream side and a diverging masonry channel on its downstream side. From the end of divergent channel, the width of irrigation channel starts. In case of flumed regulators, sufficient protective aprons are provided to protect the downstream canal bed from scour. The piers (when there are more than one span) are constructed along the width of throat and the gates move between these piers; in closed position, gates rest on the horizontal top or crest of the throat which is slightly raised (in form of a hump) above the floors of the convergent and divergent sections. The top of throat is joined to these floors by suitable slopes. The design is such that, invariably, a standing wave forms on the downstream slope (or glacis) of the hump to dissipate the surplus energy of water; also due to formation of wave, the regulator can be used as a device for measuring the quantity of water passing through it; the formula for the discharge passing through a flumed regulator is,

\[ Q = C_d \times 3.09 \times L \times H^{3/2} \text{ c.ft/sec.}, \]

neglecting the velocity of approach;

here, \( C_d = \) coefficient of discharge for flumed structure.
\[ L = \text{net lineal width of waterway in the throat portion, in feet.} \]

\[ H = \text{head of water on throat, measured from upstream water level to the horizontal crest of throat, in feet.} \]

This regulator is, in essence, a standing wave flume (described later in this chapter) fitted with gates in the throat portion.

*Note:* In Metric system, discharge formula shall be, \( Q = C_d \times 1.7 \times L^3 H^{3/2} \) cumec, where \( L \) and \( H \) are in metres.

**9. Cross regulator:** A regulator constructed *across* an irrigation channel (i.e. main canal, branch canal or distributary) is called a cross regulator and is like head regulator of a branch canal or a distributary. It is *ordinarily* constructed at distances of 9.6-12.8 km (6-8 miles) along main canal and at distances of 6.4-9.6 km (4-6 miles) along branch canal. It is used to control the quantity and level of water on its upstream as well as downstream sides and is constructed,

(i) across a parent channel just on the downstream side of the point of offtake of the offtaking channel;

(ii) across a channel just on the downstream side of the site of canal scouring escape or surplus escape.

The sill of regulator is kept a little higher than the upstream bed level of canal across which it is constructed; the regulator spans are controlled by gates, needles or baulks. This regulator is useful for,

(a) efficient regulation of water on its upstream and downstream sides;

(b) efficient working of canal scouring escape and surplus escape near it;

(c) easy closing of a breach in the canal bank on its downstream side.

Cross regulator is often combined with a road bridge to carry over it the existing road which may cross irrigation channel near the site of cross regulator. It is also usually combined with a fall (if required at the site of cross regulator), when it is called a fall-regulator; a fall regulator is
designed as a fall but it is further fitted with gates etc. to work as regulator. The general principles for design of a cross regulator are the same as for head regulator of a branch canal or a distributary.

Cross regulator may also, with advantage, be flumed like head regulator of a branch canal or a distributary; a flumed cross regulator is shown in fig 83(b), plate I.

10. Canal fall: In some situations, it becomes necessary to give a sudden fall or drop to the bed of an irrigation channel so that, at the site of drop, the canal bed level is high on upstream side of drop and low on downstream side of the drop. Such a drop in natural canal bed will not be stable and therefore, to retain this drop, a masonry structure is constructed; such a structure is called a canal fall or canal drop. Canal falls are avoided, if possible, specially in big canals. They however, in some cases, become unavoidable, e.g. at the tail end of branch canal or distributary. As at the site of canal fall there is a drop in the bed level, there will be drop in F.S.L. also, i.e. F.S.L. on upstream side will be higher than that on the downstream side of the canal fall; the difference of upstream and downstream F.S.Ls., measures the extent of fall; thus if this difference is 1.8m (6 feet), the structure is called a 1.8 m (6') fall. For economy, a canal fall may be combined with regulator or, road bridge if required at the site of fall; in such cases, the structure is called a fall-regulator or, a fall-and-bridge. When combined with regulator and bridge, it is called a fall regulator with road bridge. The common types of fall are:

(a) Vertical fall; this may be,
   (i) simple rectangular weir type fall: (see fig. 84).
   (ii) trapezoidal notch fall: (see fig. 85).

(b) Flumed fall: (see fig. 86, plate II).

In vertical fall, the difference of F.S.Ls on the two sides of fall should preferably not be more than 1.2 m (4').

Simple rectangular-weir type fall is commonly known as simple fall and it essentially consists of a drop wall across
the full width of canal and just at the drop in the bed, with its top at upstream canal bed; the drop wall is usually fitted with gates, needles or, baulks and then the structure is called a fall regulator. There is a water cushion or cistern, just on the downstream side of the drop wall and depressed below the downstream canal bed level, to dissipate the surplus energy of falling water; in addition, there are provided the necessary protective aprons on upstream and downstream sides to protect the canal bed from scour.

Simple vertical fall

Fig. 84

When a simple fall is not provided with the controlling gates, it has been found that the upstream water level suddenly falls down in a convex parabolic curve as the water approaches the drop wall; this effect is called wire-drawing and is undesir-
able as it induces excessive velocity in the water spilling over the wall and causes subsequent scour on downstream side. To do away with this wire-drawing, gates are necessary; where however, gates are not required (as, when there is no offtake channel taking off from the upstream side of

Trapezoidal notch fall
Fig. 85

fall), another type of vertical fall known as notch fall is constructed. It essentially consists of a low wall constructed across the full width of channel, with its crest or top at the upstream F.S.L. of channel; in this wall, are constructed a number of trapezoidal notches with their sills at the up-
stream canal bed level; this low wall, with notches in it, rests on the top of a masonry wall (similar to drop wall) which serves as its foundation. The masonry between adjacent notches is called notch pier. Each notch is curved in plan, as shown in the figure, and has projecting lip (at its downstream end) over which the water spills on the downstream side in the water cushion; the discharging capacity of each notch may be from 2·8 to 5·6 cumecs (100 to 200 cusecs). When the F.S.Q. of canal is less than 1·4 cumecs (50 cusecs), there may be one notch; otherwise, always more than one notch are provided. Other necessary protective aprons are provided as in case of a simple fall.

For economy, a fall may be flumed when it is called a flumed fall. It is always provided with gates and is known as flumed fall-regulator; if further, it carries a road bridge, it is called a flumed fall regulator with road bridge. The type of flumed fall commonly used is that given by Mr. Montague. A flumed fall is economical than a vertical fall and is therefore common now-a-days. It is like a flumed regulator with this difference that the extensive protective measures are adopted on the downstream side of the throat. Thus, a baffle wall is provided slightly away from the toe of the downstream glacis to destory further the surplus energy of water by sheer impact with it; on the downstream side of baffle wall, a water cushion or cistern is provided with deflector wall at the downstream end of the cistern. The cistern further helps in dissipating the surplus energy of water; the deflector wall gives an upward turn to the water currents striking against it and thus helps to check the canal bed scour on its downstream side. Beyond the deflector wall, pitching on the canal bed and sides is provided for some length. On the upstream side of the throat, similar pitching is provided beyond the upstream end of pucca floor. Discharge formula for Montague type fall is,

\[ Q = 2.8 \times L H^{3/2} \text{ c.ft/sec.} \]

Note: In Metric system, the discharge formula will be, \[ Q = 1.5 \times L H^{3/2} \text{ cumecs} \]

11. Canal escape: Any structure, on irrigation channel, through which some of the canal water is wasted
(of course with some purpose), is called a canal escape. The types of canal escape are:

(a) Canal scouring escape: (see fig. 87).
(b) Surplus (water) escape.
(c) Tail escape: (see fig. 88).

Canal scouring escape

Fig. 87

Tail escape

Fig. 88

Canal scouring escape is sometimes provided in the head reach of main canal to scour out bed-silt deposited in the head reach. It is, in essence, a regulator in one bank of main canal, with sill of regulator about 0.3 m (1') below C.B.L. at the site of escape. The gates are of the under-shot type.
The discharging capacity of this escape should be about \( \frac{1}{2} \) to \( \frac{2}{3} \) of F.S.Q. at the head of main canal and the water-way provided should be such as to ensure high velocity of entry (of the order of 4-5-6 m/sec or 15-20 ft/sec) through the escape sluices. Canal bed and sides are pitched for some distance on the upstream and downstream sides of the site of escape. Water (with the scoured silt-deposit) from the escape is led by the shortest possible lead (or escape) channel to the nearest natural drainage. When silt deposit is to be scoured out, water in excess of canal F.S.Q. (at head) is allowed through the head regulator of main canal and the gates of escape are raised up to produce scouring velocity which scour the deposited silt. For efficient working of the escape, a cross regulator is provided across the main canal just on the downstream side of the location of escape. Now-a-days, instead of scouring escape, silt ejectors are provided in the head reach of main canal as they have been found to be better and more efficient.

Canal surplus escape is meant to take out surplus or unwanted water from an irrigation channel as this surplus water may otherwise prove harmful for the channel; it is thus the safety valve of an irrigation channel (i.e. main canal, branch canal or distributary). There is also usually a cross regulator on downstream side of its location across the channel. It is useful when,

(i) there is excessive rain on the commanded area and, as such, there is no demand for irrigation water;

(ii) a breach of canal bank occurs in a canal reach, downstream of the location of surplus escape;

(iii) there is no separate surface outlet provided opposite an inlet.

Canal surplus escapes are provided at suitable points (on upstream side of every second or third cross regulator across channel) along main canal, branch canal or distributary in the bank of channel if and when found necessary after these irrigation channels come into actual operation.

A canal surplus escape may be:
(a) Weir type, with the crest of weir wall at F.S.L. of canal at the site of escape; such type resembles tail escape in construction.

(b) Regulator (or sluice) type, with the sill of sluices at C.B.L. at the site of escape; the top of gates should be above the F.S.L. of canal at the site of escape; such type resembles canal scouring escape in construction.

The discharging capacity of canal surplus escape may be \( \frac{1}{3} - \frac{1}{2} \) of the F.S.Q. of canal at the site of surplus escape.

In some cases, an irrigation channel tails in a natural drainage; in such case, an escape is constructed across the channel, at its tail or fag end; such an escape is called a tail escape and is useful in maintaining the required F.S.L. at tail of the channel. This escape is weir type, with top of weir wall at the F.S.L. of canal at the tail. A few sluices at the centre of wall are provided, with their sills at C.B.L. near the tail; these sluices are useful in making dry the tail reach when required; they can also be used for scouring the silt deposit in tail reach of the channel.

12. Irrigation outlets: Masonry structure at the head of a water course is called an irrigation outlet. It controls the flow of water in water course. Irrigation outlet is thus useful in distributing the irrigation water equitably and is, hence, of much importance. There are a number of such irrigation outlets on the right and left banks of each distributary.

Irrigation outlet may be:

(a) Non-modular outlet.
(b) Modular outlet.

A non-modular outlet is that outlet the discharging capacity of which depends on the water level in the distributary on its upstream side and also on the water level in the water course on its downstream side. A modular outlet is that outlet the discharging capacity of which is either independent of both the above-said water levels or it depends on water level in the distributary only. The former modular outlet is called a rigid module or absolute module while the latter
modular outlet is called a *semi-module* or *flexible module.* A modular outlet is necessary for *equitable, proportionate* and *economical* distribution of irrigation water (of distributary) from head to tail of the distributary; it however entails greater loss of head than non-modular outlet and is more costly.

![Diagram of Non-modular pipe outlet](Fig. 89(a))

Non-modular barrel type outlet

![Diagram of Non-modular barrel type outlet](Fig. 89(b))

Non-modular outlet may be of the following sub-types:

(i) Orifice type.

(ii) Notch type or Trough type.

Orifice type non-omdular outlet has a head wall on the distributary side and tail wall on the water course side. A concrete pipe [fig. 89(a)] or, masonry barrel with arch or flat roof as in [fig. 89(b)] runs from head wall to tail wall, under the bank of distributary; invert of pipe or, the sill of barrel is approximately kept at bed level of distributary near the irrigation outlet. F.S.L. in distributary is above the top of pipe or barrel and hence, the pipe or barrel acts as an orifice. The pipe (or, barrel) end on the water course
side remains submerged below F.S.L. in water course and hence, the orifice is a submerged orifice having the discharge formula,

\[ Q = C_d a \sqrt{2gh} \text{ c.ft/sec.}, \]

where, \( C_d \) = coefficient of discharge of orifice.

\( a \) = cross section area of pipe or barrel, in sq. ft.

\( h \) = (F.S.L. in distributary) \(-\) (F.S.L. in water course) and is called the working head of irrigation outlet.

Note: \( Q \) will be in cumecs if \( a \) is in sq. m, \( h \) is in m and \( g = 9.81 \text{ m/sec}^2 \).

In construction, notch type outlet is similar to the orifice type outlet having masonry barrel, with this difference that the barrel does not run full i.e. the F.S.Ls in the distributary and the water course are below the top of barrel. The discharge formula for this type is the same as that for the water flowing through bridge spans of a bridge.

Non-modular outlet is controlled by a shutter on its upstream end; arrangement is provided to lock the shutter in any required position so as to have a given discharge through the outlet. As loss of head in case of non-modular outlet is less than that in case of modular outlet, the former is specially useful where much loss of head is not available e.g. in case of lift area served by a distributary.

Rigid module and flexible module work as modular outlets within certain limits of water levels in the distributary and the water course; the range over which each module works as a modular outlet, is called its working range or range of modularity; also certain minimum difference of water levels on the two sides of each module should always be there to ensure its modularity (i.e. its working as a modular outlet). This minimum difference of levels is called the minimum modular head or minimum loss of head of the modular outlet. Requirements of a good modular outlet are as follows:

(i) It should be simple in design, cheap, water-tight, durable, easy of regulation and without moving parts.

(ii) It should pass, through it, its due share of silt into the water course and should not get deranged by the silt or weeds passing through it.
(iii) It should be fool-proof against outside tampering by the cultivators.

(iv) It should ensure proportionate distribution of water, with a minimum loss of head (or working head).

Semi-modules satisfy most of these requirements and are, therefore, preferred to rigid modules. The latter are, however, used under certain conditions. The only disadvantage of semi-module is that it entails comparatively greater loss of head; some of this lost head can however be recovered,

(a) by providing a divergent masonry channel on the downstream side of semi-module opening;

(b) by designing the semi-module in such a manner that a standing wave forms on the downstream side of the semi-module opening; such a semi-module is called a standing wave semi-module;

(c) by providing a divergent section and also creating a standing wave in it.

When a certain change in the discharge of a distributary causes a proportionate change in the discharge of a semi-module, the semi-module is said to be proportional and is very useful for the proportionate distribution of water when the discharge passing through a distributary cannot be kept constant. Every semi-module can work as a proportional semi-module if its sill is fixed at a particular level with respect to bed level of distributary. Fixing of the sill of semi-module is called the setting of semi-module. A semi-module for a particular setting will work as proportional as long as the distributary is in regime. When, however, the regime of distributary is disturbed due to scour and silting, the semi-module at the previous setting will cease to be proportional and the setting shall have to be changed in order that the semi-module may again work as proportional. Where the setting can be changed (by changing the opening of module, without changing the sill level, and thus changing the working head), the semi-module is called the adjustable proportional semi-module. It is also said to be rateable semi-module because it can be rated or set, as desired, to give a
proportionate discharge under different regime conditions of a distributary.

Semi-modules may be:

(i) Orifice type.

(ii) Notch type or Trough type.

Examples of orifice type semi-modules are: Kennedy gauge outlet, Jamrao orifice outlet, Inglis’ standing wave pipe outlet, Crump’s standing wave orifice outlet.

Examples of notch type outlet are: Jamrao flume type outlet, Crump’s open flume outlet, etc.

Examples of rigid modules are: Gibb’s module, Foote’s module, Kent’s module, Khanna’s module, etc. Gibb’s module is cheap and common and is used where a fixed discharge has always to be drawn from distributary into water course.

In rare cases, irrigation is practised directly from main canal or branch canal by taking the water through an outlet under the bank of main canal or branch canal; such an outlet is called direct outlet and is usually a rigid module.

Gibb’s module is the only rigid module without moving parts; it is described in the next article and is shown in fig. 90.

13. Gibb’s module: The discharge of Gibb’s module may be from 0·03 to 0·45 cumec (1 to 16 cusecs); when its discharge is from 1 to 3 cusecs or 0·03 to 0·08 cumec (as is usual), it is economical to construct it in R.C.C.; for discharges greater than 80 lit/sec (3 cusecs), it may be constructed in brick masonry. It essentially consists of:

(a) An inlet pipe under the distributary bank, taking water from distributary to a rising spiral pipe. The upstream end (near distributary) of the inlet pipe is controlled by a shutter.

(b) A rising pipe, spiralling through 180° and at the same time rising up to join the eddy chamber. Water, while passing through this spiral pipe, gets a free vortex motion.
(c) A rectangular eddy chamber with horizontal floor, circular (ordinarily semi-circular) in plan and discharging into an open spout. This chamber is fitted with a number (minimum six) of vertical baffle plates placed equidistant along its circular length; the lower edge of each baffle plate slopes up from the inner wall to the outer wall of the eddy chamber.

(d) Masonry spout with a divergent masonry flume on its downstream side. Water from the divergent flume flows into water course.

In alluvial soil, Gibb's module gives a lot of trouble with regard to silt-draw from distributary into water course. Gibb's module is also costlier than semi-module.

*Note:* 1 cumec = 1000 litres/sec.

14. **Venturi meter or Rate meter:** In its principle and construction, it is same as the one used for gauging the flow through water mains; thus, it has got a converging section, a throat and a diverging section as shown in fig 91;

![Irrigation Venturi meter](image)

**Fig. 91**

its discharge formula is also similar. The only difference is that the irrigation Venturi meter is constructed in masonry. It is used for measuring the discharge passing through an irrigation channel at the point where it is constructed. Usually, there is an automatic arrangement for measuring venturi head which is required in calculating its discharge. It is costly and is not so common in irrigation practice. Now-a-days, for measuring the discharge of a channel at a point, a flume is usually constructed at that point; this is described in the next article.
15. **Flumes**: A flume means an artificially constricted (i.e. narrowed) waterway; this is *commonly* achieved by a converging masonry channel leading (from natural cross section of canal on the upstream of flume) to the narrowed throat from where a diverging masonry channel starts, leading back to the natural cross section of canal on the downstream side of flume. The converging walls on upstream side of throat may have splay of 1:1 to 1:2 and the diverging walls on the downstream side of throat may have splay of 1:2 to 1:10. More gradual the convergence and divergence, less will be the loss of head in the flume. A flume thus, in essence, consists of a convergent masonry channel, a rectangular narrow throat and a divergent masonry channel. It is also called a *control flume* or *rating flume* and is used to measure the discharge of an irrigation channel at the point where it is constructed. There are two types of *rectangular* narrow-throated flumes. They are:

![Diagram of a flume](image)

**Plan**

Venturi flume

Fig. 92

(a) Non-modular Venturi flume or Drowned Venturi flume, see fig. 92 (line plan).

(b) Standing wave flume or Modular Venturi flume or Free-flow Venturi flume: see figs. 93(a) and 93(b).
The discharge formula for non-modular Venturi flume is the same as that for a Venturi meter; in this case, no standing wave forms in the diverging channel.

When a standing wave forms on the downstream glacis, in the diverging channel, the flume is called a standing wave flume. It is superior to Venturi flume because its discharge depends only on the upstream head measured above the crest of the throat; also, for the same upstream head, its discharging capacity is more than that of a Venturi flume. In case of standing wave flume, the length of throat should be at least 2 to 3 times the maximum head \( H \) on the top of throat. Discharge formula for a standing wave flume is,

\[
Q = C_d \times 3.09 \ L \ H^{3/2} \text{ c.ft/sec., neglecting velocity of approach.}
\]

*Note:* In Metric system, \( Q = C_d \times 1.7 \ L \ H^{3/2} \text{ cumecs.} \)

![Diagram](image)

SECTION ON AB

![Plan](image)

Standing wave flume

Fig. 93 (a)

The head \( H \) is read in a stilling well which is in communication with water in the canal, immediately upstream of the flume. Its only disadvantage is that it entails greater loss of head and where this loss of head is not available, the flume will act as a Venturi flume and not as a standing wave flume.
16. Regulating gates for sluices and, the gate lifting gear: Many canal masonry works have got sluices between piers and these sluices are controlled by gates which are operated by lifting gear from overhead gate bridge. Following are the usual types of gates (or, shutters) used:

(a) Lift gate
(b) Radial gate: see fig. 94.
(c) Needle shutter: see fig. 95.
(d) Baulk shutter: see fig. 96.

Standing wave flume
Fig. 93(b)

Lift (or draw) gate may be a sliding gate (as for small spans upto 3 m or 10') or a roller gate as for larger spans. The small gate is usually of cast iron with a mild steel lifting rod at its centre; the lifting rod is threaded in its top portion. Such a gate is called screw gate.

Large gates are usually built up of horizontal rolled steel joists at certain distance centre to centre along the depth of gate, with a facing of mild steel plate fixed to the
upstream flanges of R.S.Js to retain water. The gate may be reinforced (i.e. strengthened) at its back by angle iron (or channel iron) members. Stoney gates are used for big spans. Big and heavy gate is tied at both of its ends to the lifting cables; such gate may have counter-weight attached to it so that the power required to pull it up will be less.

![Diagram of Radial Gate](image)

Radial gate

**Fig. 94**

Now-a-days, radial gate is being used for small as well as large spans. In essence, a radial gate is like a sector gate which is also used for waste weir sluices of a storage dam; the arc-face of gate closes the span between two adjacent piers; in closed position, the bottom of arc rests on sill of the sluice and its top is slightly above the top of sluice opening which is controlled by this gate.

Needle shutter consists of a number of vertical timbers or scantlings about 15 cm (6") wide by 10 cm (4") thick and of required length; these timbers are called needles and are kept (with their length vertical and the width in the direction of span) side by side, butting each other to close the entire width of span. The lower ends of needles rest on sill of the sluice and the upper ends are supported against a horizontal piece running between two adjacent piers; sometimes, another horizontal piece may support them at the middle of their (vertical) length. Such shutter is used for un-important
regulators and for depths of water less than 3 m (10'). There is leakage of water between needles and the regulation by needles is rather slow and tedious. When water is to be allowed on the downstream side, needles are lifted up and stored on the overhead bridge or near it.

![Diagram of needle shutter](image)

Needle shutter

Fig. 95

Baulk shutter consists of a number of horizontal timbers of required section and of length slightly in excess of the sluice span. There are vertical grooves in the sides of two neighbouring piers; each timber (called a baulk or wale) is made to slide down, in the vertical grooves at its ends,
one after another to make up the required height of shutter. Baulk shutter is useful for holding water upto 1.2 m (4') depth. It is inferior to needle shutter as the baulks are more difficult to handle than needles and further, the baulks cause silt-deposit on their upstream side.

The device for operating gates (lift and radial) is called gate lifting gear or gate-hoist. Usually, screw gear is used for small lift gates with lifting rods and, winch gear is used for large span gates. When there are a number of spans in a structure, there may be one winch gear for each gate fixed on the overhead bridge or, for economy, there may be only one or two travelling winch gears. A travelling winch gear consists of winch gear mounted on a travelling gantry and each such winch gear can operate a number of gates.
CHAPTER XIII

IRRIGATION CHANNELS AND CANAL MASONRY WORKS IN NON-ALLUVELIAL SOIL

1. Introductory: This chapter deals with pickup weir head works, irrigation channels and canal masonry works in non-alluvial soil. Most of the description, about them is the same as that for the similar irrigation works in alluvial soil and hence, such description will not be repeated here; however, the special points of contrast will be described here. Also, certain works that form a special feature of non-alluvial area will be treated here.

![Diagram of pickup weir head works](image)

Line plan of pickup weir head works

Fig. 97

2. Valley irrigation and pickup weir: In non-alluvial soils, valley irrigation is highly developed and there are many major and minor storage irrigation schemes. As already said in chapter iv, pickup weir forms the head works of main canals in some major storage irrigation schemes; these head works are shown in fig. 97. Water from main reservoir is allowed to come out through supply sluices, it
flows down the river, is picked up by the pickup weir and is diverted into main canals which take off from the upstream side of the pickup weir. Weir scouring sluices (i.e. undersluisces) are provided in the flank of pickup weir, just near the head regulator of main canal on each side. There is subsidiary storage at the back of pickup weir, with water level usually maintained upto the top of the pickup weir as far as possible. Main canals are fed from this subsidiary storage. During floods, surplus water flows downstream over the entire crest of the pickup weir. Necessary protection in the form of apron or water cushion is provided on the downstream side of pickup weir to protect the river bed from scour due to water spilling over the weir crest. The theoretical profile of a solid gravity pickup weir will be like that of a solid overflow dam. In the practical profile slight upstream batter is given, keeping the base width same as designed; the downstream face in practical profile should not be flatter than 0.5 in 1; the minimum crest width = $\sqrt{d + H}$ feet, subject to the roadway requirement, if any. As usual, the stability of practical profile is checked analytically and graphically. The pickup weir in valley irrigation is usually on impermeable foundation. When, however, the foundation bed is some what pervious or permeable, the uplift should be considered in the design of pickup weir. It may be constructed in stone masonry or cement concrete.

3. Testing the stability of pickup weir analytically: The stability of a pickup weir is tested for the following stages of river flow:

(a) No water passing downstream, over the crest of weir.
(b) Water passing downstream, over the weir crest; here, there are two conditions, viz.,
   (i) Weir discharging freely, as in case of greater-height weirs; this is usual.
   (ii) Weir giving a drowned discharge, as in case of low-height weirs; this case is not so common.

In case of condition (a), the weir stability is checked when water level on the upstream side is upto weir crest and there is no water on the downstream side.
In case of condition (b) (i), the stability is checked when water level on the upstream side is at the upstream H.F.L. and that on the downstream side is at the corresponding downstream H.F.L. which will be below weir crest or, in the limit, up to the weir crest.

Under condition (b) (ii), the stability is checked when water level on the upstream side is at the upstream H.F.L. and that on the downstream side is at the corresponding downstream H.F.L. which will be always above the weir crest but below the upstream H.F.L.

It is a bit difficult to state, off-hand, the condition which will prove worst for a weir; hence the stability of a weir may be checked for all the three conditions and the worst condition located and provided for.

4. **Irrigation channels**: See article 2 of chapter xi.

5. **Alignment**: See article 3 of chapter xi and note the following special points of contrast:

   Main canal is along the side-long ground, on falling contour, for practically whole of its length and as such, it forms the boundary line of the irrigated area; it thus commands the land on one side only, namely, between its alignment and the river across which its head works are constructed. Also, as it is a non-ridge channel, it comes across many natural drainage lines, necessitating the construction of costly cross drainage works. It is usually in deep cutting and high embankment. In some cases, main canal may be able to catch a ridge at its tail and, in such cases, it will be able to command land on both sides in the tail reach. Branch canals and distributaries are aligned on ridges across the side-long ground. From above, it is clear that main canal is never central to its commanded area; in fact, it is on the border line of the commanded area. A rough rule for the radii of curves in channel alignment is as follows:

   For large channels, the minimum radius of curve should be 40 times the bed width of channel and for small channels, it should be 30 times the bed width of channel.
6. **Full supply discharge**: See article 4 of chapter xi.

7. **Losses due to evaporation and seepage**: See article 5 of chapter xi and note the following change:

In non-alluvial canals, these combined losses are usually allowed at the rate of so much percentage of the F.S.Q. of channel, per mile length of the channel. This percentage is greater for less value of F.S.Q. and vice versa. Below is given a table of these percentages as used for Bombay-Deccan canals:

<table>
<thead>
<tr>
<th>F.S.Q. in cusecs</th>
<th>Allowance for transit losses</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 100</td>
<td>0.25 %</td>
</tr>
<tr>
<td>51 — 100</td>
<td>0.5 %</td>
</tr>
<tr>
<td>26 — 50</td>
<td>1.0 %</td>
</tr>
<tr>
<td>15 — 25</td>
<td>2.0 %</td>
</tr>
<tr>
<td>&lt; 15</td>
<td>3.0 %</td>
</tr>
</tbody>
</table>

*Note:* This table can be changed to Metric units by putting 1 cusec = 0.02832 cumeer, and rounding off the figures slightly.

8. **Design of irrigation channels**: See article 6 of chapter xi and note the following peculiarities of design:

Velocity of water should be non-scouring as the water is comparatively silt-free in non-alluvial soil. Following are the non-scouring velocities considered safe for the strata mentioned against them:

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Safe mean (non-scouring) velocity in ft. per sec.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hard clay or Grit</td>
<td>4 (1.2 m/sec)</td>
</tr>
<tr>
<td>Soft rock</td>
<td>5-8 (1.5-2.4 m/sec)</td>
</tr>
<tr>
<td>Hard rock</td>
<td>8-12 (2.4-3.6 m/sec)</td>
</tr>
</tbody>
</table>

The bed slope usually given is that which is available on the ground surface; it however should be such that it does not induce scouring velocity in the channel. Silt problem is not considered while fixing bed slope as the water is comparatively silt-free.
B/D ratio depends on F.S.Q., transit losses and the economical earthwork for channel. A channel of given discharge can have a number of B/D ratios; of course, one B/D ratio will be slightly more suitable than the other from the point of view of economy and efficient working of the channel. For the same discharge, B/D ratio of non-alluvial canal is comparatively less than that of alluvial canal. The rise in bed at the junction of the reaches of a channel may be slightly more than 3 cm (0·1 foot), depending on the hardness of the strata through which the channel passes. Every channel should take its due share of water without showing signs of scour. Side slopes for some of the strata are as given below:

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Side slopes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grit</td>
<td>$\frac{1}{3} : 1$ to $\frac{2}{3} : 1$</td>
</tr>
<tr>
<td>Soft rock</td>
<td>$\frac{1}{4} : 1$</td>
</tr>
<tr>
<td>Hard rock</td>
<td>$\frac{1}{8} : 1$ or, vertical sides</td>
</tr>
</tbody>
</table>

Inside-berms are kept during construction; they cannot be formed by inducing the deposit of fine silt. High banks are made staunch by constructing the inner core of watertight material if the soil of canal bank is highly pervious.

Borrow pits are located outside and not in the bed or the inside-berm of channel.

Typical cross sections of channel in non-alluvial soil are given in figs. 98(a), 98(b) and 98(c).

9. **Velocity formulae:** Mean velocity in a channel in non-alluvial soil is worked out from,

(a) Chezy formula or,

(b) Manning formula.

According to Chezy,

$$V_m = C\sqrt{m} \text{ ft/sec,}$$

where, $V_m$ = Mean velocity of flow in an open channel;

$C$ = Chezy constant and is found from the equation for $C$ given by either Kutter or Bazin;
\( m = \) Hydraulic Mean Depth of channel; it is also denoted by 'R';
\( i = \) Hydraulic slope of channel; it is also denoted by 'S'.

*Note:* In M.K.S. system, \( V_m = C \sqrt{mi} \) m/sec where \( m \) is in metres and value of \( C \) used in this formula will be the value of \( C \) used in F.P.S. units divided by 1.81.

---

According to Manning,
\[
V_m = \frac{1.486}{N} m^{2/3} i^{1/2} \text{ ft/sec.}
\]
where, $N = \text{Rugosity factor or Coefficient of roughness and}$ depends on the resistance offered by the bed and sides of channel to the flow of water; more rough the bed and sides, greater the value of $N$ and vice versa.

$V_m, m$ and $i$ have usual significance.

*Note:* In Metric units, $V_m = \frac{1}{N} m^{3/2}, i^{1/2} \text{ m/sec}.$

10. **Silting and Scour:** See article 8 of chapter xi except the silt problem as this is *practically* non-existent in non-alluvial soil.

11. **Canal lining:** Read article 9 of chapter xi and note the following difference:

Stauntness of canal bed and sides cannot be secured by inducing the deposit of fine silt; hence, where economically feasible, canal lining is resorted to.

12. **Aquatic weeds:** See article 11 of chapter xi. The weed-growth nuisance is more acute in non-alluvial channels. The water in channels is given on intermittent system and during the period of closure of channel, weed growth can be removed.

13. **Canal masonry works:** As in alluvial soil, they *mainly* consist of cross drainage works and regulation works. The function, the *general* shape and *general* principles of design of these works in alluvial and non-alluvial soils are similar.

Following are a few items of contrast:

(a) Alluvial soil is soft and of great thickness; hence, the structural foundation of an irrigation structure has no rock bed to rest upon; due to this, the depth of structural foundation is considerable or, if depth is kept small, the structure is protected by extensive protective aprons (specially on the downstream side) and deep cutoffs in the form of sheet piles.
(b) Stone is usually not available in plenty in alluvial area; hence, the irrigation structures are constructed in brick work or cement concrete; structures in non-alluvial area are usually of stone or cement concrete.

(c) Due to facilities of irrigation in alluvial area, irrigation structures in that area are usually of bigger magnitude.

(d) In designing an irrigation work in alluvial soil, silt problem has to be considered; there is no such silt problem in non-alluvial area.

14. **Cross drainage works**: See article 2 of chapter XII.

15. **Aqueduct**: See article 3 of chapter XII and note the following points:

   Foundations will not be so deep as the rocky stratum is usually available at a little depth below river bed. Floor between piers may be of inverted arch resting on mass concrete.

   The portion of downstream wall of the trough of masonry aqueduct can be used as surplus escape if the bed of river is fairly hard at the site of work. Thus, by combining surplus escape with masonry aqueduct, the cost of separate surplus escape can be saved.

   Silt deposit in the barrels of syphon aqueduct is not much to be feared and as such, there need not be flat slopes at the ends of barrels. A siphoned structure is more undesirable in alluvial soil than in non-alluvial soil because of the silt problem in alluvial soil.

16. **Superpassage**: See article 4 of chapter XII.

17. **Level crossing**: See article 5 of chapter XII.

18. **Drainage inlet and Surface outlet**: See article 6 of chapter XII.

19. **Regulation works**: See article 7 of chapter XII.

   In addition to a simple vertical fall, there may also be an inclined fall or drop (in channel bed) known as a rapid. There may also be a tail reservoir at the tail of a big channel.