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जल संसाधन, नदी विकास और गंगा संरक्षण विभाग
केन्द्रीय जल आयोग
राष्ट्रीय जल अकादमी, पुणे

केन्द्रीय जल अभियांत्रिकी सेवा के नव नियुक्त अधिकारियों का इकत्तीसवां प्रवेशन प्रशिक्षण कार्यक्रम

19 August 2019 – 07 February 2020

डिजाइन और अनुसंधान

Module IV: Design of Weirs, Barrages and Canals

पुणे
दिसंबर 2019

Government of India
Central Water Commission
National Water Academy

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Module- IV

DESIGN OF WEIRS, BARRAGES AND CANALS

09-11 December 2019

Module Co-ordinator
Shri S N Pande, Director (Designs)

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Module – IV

DESIGN OF WEIRS, BARRAGES AND CANALS

INDEX

Chapter - 1

Barrage and Weir Planning

1.0 Introduction

Diversion of water for various purposes like irrigation, drinking, flood control etc has been practiced from ancient times. In India before the British rule diversion / irrigation was done generally by Inundation canals in north India and by Tanks in south India. Many improvements were brought to this system during British period. Some of which are summarized as below:

- Western Yamuna Canal – Tajewala (1873/1943)
  – (Presently-Hathnikund (1999))
- Hansli Canal – UBDC with Madhopur Headwork (1851 / 1879)
- Grand Anicut –Cauvery (1826)
- Godavari – Cotton Anicut (1852)- 6km –Dowlwswaram, Ralli, Maddur and Vizzeshawaram- Godavari Barrage(1982)
- Krishna Anicut (1855)-Prakasham Barrage(1955)
- Sirhind Canal – Rupar Weir - Sutluj (1882) -Remodelled 1895 – Divide Wall / Still Pond
- Sidhnai Weir – Ravi (1886)
- Khanki Weir – Chenab (1902) – Hinge type Shutters/ Silt Excluder
- Hussenawala, Sulemani, Islam and Panjnad – Satluj
- Taunsha Weir – Indus (1947) – RCC Raft
- Sukkar Barrage – Indus (1932) – 7 canals
- Kotri Barrage – Indus (1947)
- Triple Canal Project – Interlinking of rivers

Many important concepts were developed by reputed institutes and exceptional researchers some of which are as follows:

- Irrigation Research Institute Lahore (1929) –Hydraulic Research Station Malikpur (Pathankot) – Dr Taylor, -Dr KanwarSain – Mr . T Blench
- R. G. Kennedy – CVR concept
- F. H. Wood, E. S. Lindley
- Gerald Lacey
- Hydraulic Gradient Theory (Cantley, Hingham)
- W. C. Bligh (19010) Creep Theory
- Khosla's Theory
- Concepts for Silt Prevention – Higher HR crest, silt vanes, Silt ejector, Silt Excluder etc.
- Divide Wall
- Collapsible wooden / steel shutters evolving to motorised Gates

A brief description of diversion structures (Barrages and Weirs) along with issues pertaining to their planning and design are discussed as follows:

1.1 Basic Planning for Diversion Structures

Diversion structures are run of the river structures and involve only incidental storage and hence provide better sediment control, very low submergence, less ecological problem. However these do not provide flood moderation and serve generally less command area than storage projects.

Many different types of diversion structures are as follows:

- **Diversion without any Structure**
- **Weirs**

Side Weir

Vertical Drop Weir

Slope Weir

(a) Solid masonry Weir

(b) Rockfill Weir

(c) Glasis Type Weir

Ogee or Parabolic type weir

Buttress typeWeir

Crib Weir

Syphon Weir

Trench Type Weir.

Kolhapur Type Weir (K T weir)

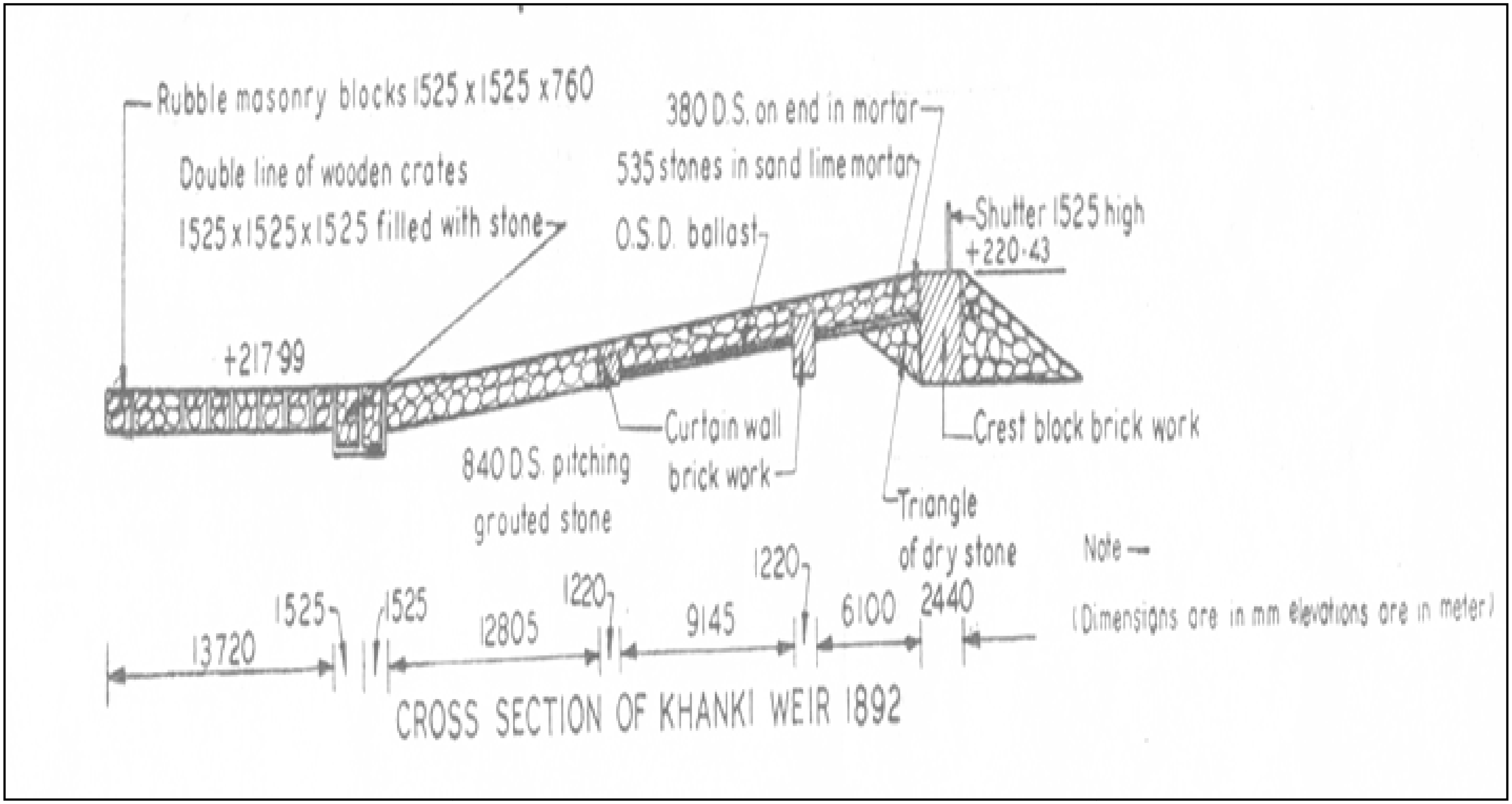
- **Barrages**

Gravity Type

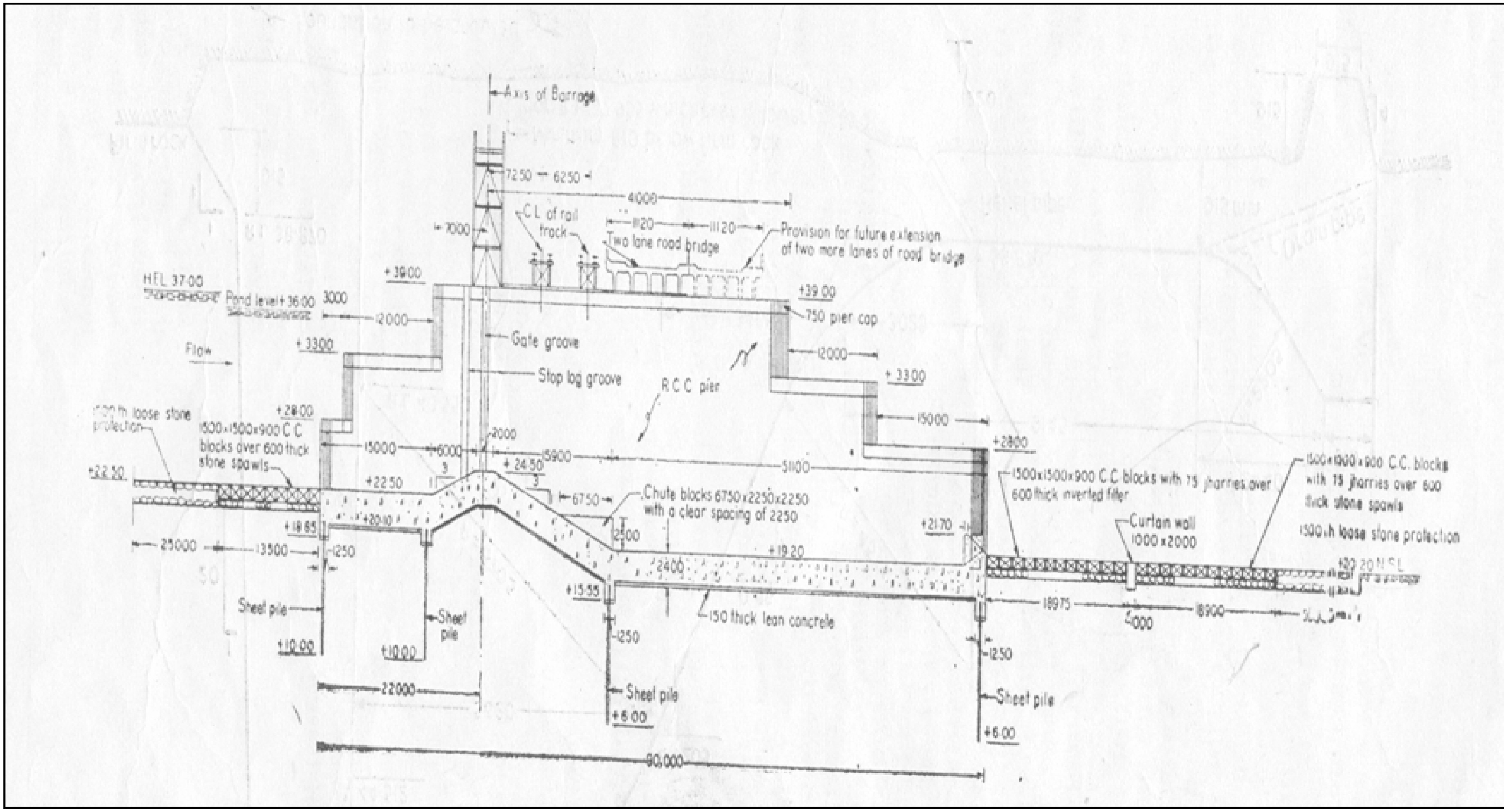
Raft Type

1.2 Basic components of Barrage

- Upstream protection arrangement
- Upstream Pucca Floor
- Crest
- Stilling Basin
- End sill
- Downstream protection arrangement
- Piers
- Divide Walls
- FishPass
- Undersluices
- Abutments
- Flare out wall
- Flank wall
- Guide Bunds
- Return Walls
- Afflux bunds
- Cutoffs
- Intake Regulator with similar arrangements



Weir (Old Pattern)


Axis of Barrage
7250
6250
4000
7000
C.L of rail track
1120
1120
Two lane road bridge
Provision for future extension of two more lanes of road bridge
HFL 37.00
Pond level +36.00
3000
+39.00
12000
+39.00
750 pier cap
Flow
+33.00
Gate groove
Stop log groove
12000
+33.00
R C C pier
1500 th loose stone protection
+28.00
1500×1500×900 C C blocks over 600 thick stone spawls
15000
6000
2000
15900
5100
15000
+28.00
+22.50
+22.50
+24.50
+20.10
6750
Chute blocks 6750×2250×2250 with a clear spacing of 2250
1500×1500×900 C.C blocks with 75 jharries over 600 thick inverted filter
1500×1500×900 C.C. blocks with 75 jharries over 600 thick stone spawls
Curtain wall 1000×2000
1500 th loose stone protection
+18.65
25000
13500
1250
2500
+21.70
+19.20
2400
+15.55
18975
2000
18900
Sheet pile
Sheet pile
150 thick lean concrete
1250
1250
+10.00
+10.00
22000
Sheet pile
+6.00
Sheet pile
+6.00
90,000

Barrage

Points to be considered while selecting a barrage vis –a – vis weir

Barrage		Weir	
Advantages			
(i)	Due to low crest of under sluices and spillway, the afflux created on the u/s by the barrage is usually small	(i)	Where the natural banks are high enough to contain the affluxed water within the banks or when only low flood protection bunds would be necessary, a weir would be economical
(ii)	With Barrage better control over the river flow can be effected by judicious operation of the gates	(ii)	Due to provision of gates only in the sluice bays, the operation would be easier and cost of structure would be less. Wherever needed, the pond level could be conveniently raised by falling shutters up to about 2 meters
Disadvantages			
(i)	Cost of barrage is higher than that of a weir generally.	(i)	Control over the river flow cannot be had fully as in the case of barrage
		(ii)	Chances of silting on U/S are more

1.3 Detailed Investigations required for DPR

- Working Group Report on Guidelines for Preparation of Detailed Project Reports of Irrigation and Multipurpose Projects; Vol II Guidelines for Detailed Project Report; Section 3 Report; **sub section 3.4 & Survey and Investigation subsection 3.5 hydrology**

(Some important criteria for the extent of topographical and geological survey to be carried out for various structures covered in DPR are given in annexure 1)

Further guidelines are given in the codes as mentioned below:

- IS 7720Criteria for Investigation Planning and Layout of Barrages and Weirs; Cl. 3 Detailed Investigation
- IS 6966 Hydraulic design of Barrages and Weir Part 1 Alluvial Reaches; Cl. 4 Data Required
- IS 11130 Criteria for Structural Design of Barrages and weirs; Cl. 3 Data required
- IS 13578 code of Practice for subsurface exploration of Barrages and weirs;
- Established design practices, past experience and sound engineering judgment

Under various sections of Working Group Report on Guidelines for Preparation of Detailed Project Reports of Irrigation and Multipurpose Projects; Vol II Guidelines for Detailed Project Report; Section 3 Report, guidelines have been given with respect to topographical, geological, hydrological, sediment survey, ecological etc aspects. These will be covered by other experts in their respective fields. However the main investigation required for planning and design of barrages and weirs for DPR preparation as per various IS codes are given below:-

From the analysis of the data obtained during preliminary investigations, the final selection of the site for locating the diversion structure with its head regulators can be judiciously made. For collecting the data necessary for design of structures at this site, detailed investigations need to be carried out. These should cover: (i) Detailed topographic survey (ii) Collection of hydrological, sediment and evaporation data(ii) Surface and sub surface investigation including laboratory tests (iv) Diversion requirements (v) construction material (vi) communication system (vii) other miscellaneous studies.

1.3.1 Topographical Investigations

For topographical investigations, following data is required:

(a) An index map of the area has to be prepared. This should show the catchment of the entire river valley upstreams of the selected site and should also show salient features like important irrigation works in the vicinity, road and railway crossing, hydrological stations, dry and wet fields, garden, temples, graveyards, villages etc, which may come under submergence, if any, due to the pond created. The map may preferably be drawn to a scale of 1 in 4000.

(b) A contour plan of the site of barrage or weir has to be neatly prepared. The contour interval should not be more than 0.5 m and should cover elevation of at least 2.5 m above the high flood level. The contour plan should extend to about 5 km on upstream and downstream of the site and upto an adequate distance on both the flanks up to which the effect of pond is likely to extend. The plan should clearly indicate salient features like tributaries and their confluence with river, firm banks, Rock outcrop, deep channels, large shoals and islands, deed pools, important landmarks, marginal erosion etc. the length of survey may depend on configuration of the river. This could be shortened or increased depending upon the straight reach or meandering nature of the river. In case of meandering river, the length of survey should cover at least two fully developed meanders on the upstream and one on the downstream of the axis of diversion structure. The plan may preferably be drawn to a scale of 1 in 4000.

(c) Cross sections of the river have to be taken at the axis of barrage or weir at the proposed site and at regular intervals, both on the upstream and downstream of the axis. The interval shall be 200 m upto atleast 600 m from the proposed site. In addition to these, cross sections of the river on the upstream side may be taken at 2 km interval up to the distance to which the back water effect of the ponding is likely to extend. The interval for taking the cross section may be reduced if the topography indicates appreciable fall in the slope of the river. The cross section levels in the river bed may be spaced at 10 to 30 m depending upon the topography of the river. The cross sections should generally be extended on both the flanks up to about 2.5 m above the high flood level. It should be such that a proper layout of guide bunds and afflux bunds wherever necessary, could be planned. In hilly streams, the cross section interval should be closer. The requirement of cross section and longitudinal section in respect of model studies are discussed in the chapter on model studies.

(d) Longitudinal section of the river along the deep current should be taken for a distance of about 5 km on the downstream side and 15 km on the upstream side. If the backwater effects are likely to extend beyond 15 km longitudinal section should be taken up to that distance. The water levels observed all along the section should be marked on the section.

1.3.2 Hydrological Investigations

1.3.2.1 Hydrological Data

The success or otherwise of the diversion scheme depends to a great extent on the accuracy and availability of hydrological data. The hydrological data (such as Flood discharge corresponding to 50, 100 and 500 year frequency and corresponding water levels, Gauge – Discharge Curve at Barrage site upto HFL, Other salient water levels i.e. Pond level, MDDL etc) forms the basis of hydrological and structural design of Barrage. For such basic parameters, it is required to assess the available weekly and monthly runoff and also for computing the flood for which the structure has to be designed. It is very important to collect the following data for further studies.

(a) Daily rainfall data of all rainfall gauging stations in the catchment area and also around the catchment area wherever available. The data should be collected for as many years as possible. Data regarding the storms in respect of successive positions of the center of the storm on the catchment should be collected. Storms causing peak discharge should be separated for unit hydrograph analysis.

(b) For working out unit hydrograph, flood hydrograph for isolated rain storms are to be collected.

(c) If adequate data is not available, synthetic hydrograph has to be developed. For this purpose catchment characteristics such as shape, slope, orientation, drainage system and infiltration capacity should be collected.

(d) For working out the design floods by frequency analysis, peak flow data for the river for as many years as possible should be collected.

(e) Daily stage and discharge data of the river at or near the site for as many years as possible for monsoon and non monsoon period should be collected. In case gauge and discharge data at the proposed site is not available, the hydrological observations at the site should be immediately started so that the calculated and observed values could be correlated with two gauges on the upstream, one at the axis and two on the downstream. All the gauges should be connected to the nearest GTS Bench Mark and observations carried out as per standard practice.

(f) For working out a gauge – discharge curve and also to estimate maximum flood by slope-area method, the flood marks in the vicinity of site should be recorded by local inquiries.

1.3.2.2 Sediment Data

For planning sediment excluding and / or ejecting devices at the headworks and in the canal system and to evolve a suitable gate regulation for satisfactory sediment exclusion, it is necessary to have the sediment load carried by the river for as such period as possible. It is required especially for the flood season when the sediment carried will be more. If no data on the sediment is available, sediment observations should be started immediately along with gauge and discharge observations at the site to assess the quality and quantity of the sediment carried in the river. The sediment sampling should be done as per standard practice. If the quantity of the sediment carried by the river is excessive, the pond levels have to be fixed carefully taking the sediment data into account. This is especially important when the pondage is proposed to be provided to meet diurnal power fluctuations also.

1.3.2..3 Evaporation Data

Evaporation is an important factor to be considered while working out the water requirements of the crops and hence the diversion requirement. The available data on evaporation by pan measurement etc. should be collected and analyzed.

1.3.3 Surface and sub surface Geotechnical Investigation including lab Tests

For Proper design of the diversion structure and its components, it is very necessary to have a complete picture of the surface and sub surface conditions. These are required both for deciding the type of structure to be provided and also its safety. The foundation may be of rock or alluvium and depending upon the same, the investigation may be different. These are discussed below:

(a) Bore holes should be driven at specifies intervals covering the entire area of the barrage or weir and its appurtenant structures, and the location of the bore holes should be indicated in the survey sheets. Such Plan should be appended with DPR. Correct log charts are to be prepared clearly. The boring should be generally carried out to hard rock level or to a depth of about 15 to 25 m below the deepest river bed level depending upon the strata and type of the structure. The bore holes should be at the rate of at least one in each bay proposed. If possible two bore holes in each bay, one at the upstream cutoff and another at downstream cutoff, should be obtained. When the presence of clay in the foundation is detected, these are however essential so that the extent, depth and location of the clay layer could be correctly assessed. A table showing the corresponding 'N' i.e. S.P.T. Values covering foundation of all the major components of barrage should be appended with DPR. The grain size distribution analysis, moisture content, voids ratio, in-situ density, submerged density, coefficient of permeability etc. may also be determined and furnished. Shear parameters (C-Φ values) of the foundation and backfilled material should also be investigated and furnished.

(b) Modulus of sub-grade reaction at the proposed foundation level of the barrage. This value to be obtained by conducting in-situ tests conforming to I.S. 1888-1982 - Method of Load Test on Soils.

(c) If clayey strata is met, undisturbed sample of clay layers from the proposed foundation level or up to 8m or more depth may be taken. These samples may be analyzed for unconfined compressive strength, swelling index, consolidation characteristics and other parameters of soil as stated above and results furnished.

(d) Values of Design Seismic Co-efficient to be adopted, in case the barrage is situated in earthquake zone III onwards.

(e) Permeability coefficient of foundation material as well as material on banks.

(f) Water table data in monsoon and spring for past few years.

(g) If loose sand is met, liquefaction potential studies may be undertaken.

(h) If rocky strata is available at shallow depths, trial pits may be excavated to determine the depth of overburden, loose deposits, depth of weathered zone and extent of joints and fissures so that the necessary function treatment for preventing excessive seepage loss etc., can be worked out.

(i) Whenever the depth of overburden is large and comprises large sized boulders and it is difficult to have ordinary boring methods, geophysical investigations needs to be carried out to locate the rock surface.

(j) Whenever impervious layers are encountered, the direction of its boundary i.e. horizontal or sloping up towards the downstream or upstream should be ascertained. An impervious boundary sloping up towards the downstream will cause constriction of seepage. This needs to be avoided by shifting the location of weir or barrage slightly upstream.

(k) If the river bed consists of boulders or is made of stiff soil, feasibility or otherwise of driving sheet piles to act as cut offs must be investigated properly.

(l) If the piers or abutments are to be designed with reinforced cement concrete pile foundation, bearing capacity of piles should be determined by driving test piles before finalizing the design of pile system, at sites where the bearing capacity of the soil is low at shallow depths.

(m) In sandy strata, standard dynamic and static penetration tests should be done below the position of each structure like abutments, piers, divide walls, center of each bay etc. for estimating bearing pressure, likely settlement and necessity of settlement joints etc. For boulder strata, plate bearing test should be done. The depth of penetration may be at least half the bay width of the barrage or weir. The interval may be decided depending upon strata encountered.

(n) Whenever the main floor of the structure is to be designed as an R.C.C. raft supported on an elastic medium, in situ tests for determining the modulus of sub grade reaction at the proposed foundation level should be conducted. If there is a wide variation in the properties of the foundation material, the length of structure should be spilt up into suitable sections isolated from each other by means of double piers and the modulus of sub-grade reaction must be determined at every section.

(o) Soil samples should be collected at suitable intervals of depths and their properties such as classification, unit weight, angle of internal friction, void ratio, specific gravity, grain size distribution etc should be determined by sieve analysis and other laboratory tests.

(p) Whenever clayey strata is encountered, undisturbed samples of clay layers should be obtained, one from the proposed foundation level and another 5 m below the foundation level for each bay. These samples should be analyzed to determine the cohesion, unconfined compressive strength, moisture content, dry density and sensitivity and consolidation characteristics.

(q) Whenever clayey strata is proposed to be treated by means of sand drains or stone columns, coefficients of permeability and consolidation should be determined in vertical as well as radial direction.

(r) Observations of water table in the vicinity of barrage or weir site should be carried out. The effect of higher pond level leading to rise in water table with consequent water logging in the adjacent areas and the attendant problems thereof should be investigated.

(s) Field permeability tests should be carried out to assess the extent of seepage loss from the pond and thereby the dewatering requirements.

1.3.4 Diversion Requirement

The quantum of water that is required to be diverted through the barrage or weir has to be determined. For this purpose, detailed study of the command area, crop pattern, drainage, water requirement at the farm outlets, any supplementary source of water, evaporation losses, percolation losses, requirements for power, drinking water supply, industrial use, riparian rights etc. have to be made. The full supply level at the head regulator has to be worked out.

1.3.5 Construction Material

The type of structure that can be economically built at the site will depend upon the availability of the construction material in the vicinity. Hence a detailed survey of the materials like stone, limestone, brick, sand gravel, suitable earth etc. has to be made with regard to both quality and quantity of the same. The lead and lifts with necessary transport arrangement will have to be found out. Laboratory and field tests should be carried out for determining the quality of aggregate and earth materials. For limestone, its hydraulicity, strength and durability have to be assessed. It is preferable to prepare a map indicated the quarries, approach roads, colonies etc., for a proper assessment of the available construction material. Comparative estimates of the type of structure will have to be made depending upon easy and uninterrupted supply of the required quantity of quality materials.

Some other investigations may also required as per site requirement

1.3.6 Communication system for access to the site

1.3.7 Other Miscellaneous Studies

1.3.8 Pond Survey

1.3.9 Fish Pass

1.3.10 Log Chute

1.3.11 Rail / RoadBridge across the Barrage

1.3.12 Data relating to Ice Problems

1.3.13 Environmental Considerations

1.3.14 Fish Culture

1.3.15 Wild Life Habitat

1.3.16 Historical and Cultural Repercussions

1.3.17 Other Ecological Factors

1.3.18 Water Logging Problem

1.4 Layout and Planning

IS 7720 code of Practice for Investigation Layout and Planning of Barrage

1.4.1 General arrangement

A barrage or a weir normally comprises a deep pocket of under sluice portion in front of the canal head regulator on one or both sides and the remaining river bays separated from the under sluice bays by divide walls. In addition, guide bunds on the upstream and downstream of the barrage or the weir and sediment excluding devices, such as excluders in the barrage and ejectors in the canal are provided. Detailed model studies should be carried out to decide the location and layout of the barrage appurtenant works.

1.4.2 Location

Location for a barrage or weir should be decided on considerations of suitability for the barrage or weir proper, the under sluices and the canal head regulator. An ideal location is that which satisfies the requirements for all the three. For irrigation purposes, the headworks shall be so planned that full command may be obtained by a barrage or weir or reasonable height. The combined cost of construction of the headworks and the canal from the barrage or weir to the point where irrigation commences should be as small as is consistent with the efficiency of the project. Sometimes a most favorable site for a barrage or a weir may have to be discarded due to large quantity of excavation involved in the construction of the works. These points need careful considerations while deciding the proper location of the barrage.

The river reach should as far as possible, be straight so that velocities may be uniform and the sectional area of the stream fairly constant. The banks should be preferably high, well defined and in-erodible. This will obviate oblique approach as well as non-uniform

distribution of flow on to the barrage. If such a site is available, it may need very small or practically no guide bunds. In case of high banks, the country will not be submerged during high floods and a considerable saving in the cost of flood protective embankments can be effected. In the case of a meandering river the barrage or weir should be located at the nodal point.

A slight curvature at site may be advantageous from the point of view of off-taking channel, which when located on the downstream end of the outer curvature will have the advantage of drawing less sediment. However, cross currents may be produced due to curvature and may endanger the foundation. Moreover, if canals take-off from both the banks, the canal taking off on the inner side of the curve will draw comparatively more sediment. Therefore, proper judgments should be exercised while deciding the location in a curved reach of the river

The under sluices should be sited in the deep channel in order to ensure adequate supply to the canal head at all times, when canals take-of from both sides, a site with deep channels on both banks and low water in the centre is the most suitable.

While deciding the layout, due consideration should be given for all possible locations of the sediment ejector and availability of levels for effective functioning of the escape channel

1.4. 3 River diversion Scheme

While deciding the location and layout of the barrages due consideration should be given to the river diversion and flood handling scheme during construction. At times the hydraulic requirement may have to be compromised to obtain a workable diversion scheme or the barrage constructed in a spill of the river and the river diverted on to it by providing suitable river training works. Such decision should be supported by adequate model studies.

1.4.4 Alignment of Barrage or Weir

The alignment of a barrage or weir should be such as to ensure normal and uniform flow through all the barrage or weir bays as far as possible. A barrage or weir aligned at right angles to the river course will have the minimum length and normal flow thereby minimizing the chances of shoal formation and oblique flow a skew alignment should be avoided unless otherwise necessitated by site conditions. The alignment should as far as possible be finalized after hydraulic model studies.

1.4.5 Alignment of the Head Regulator

The upstream abutment of the head regulator should be set back from the line at right angles to the barrage axis. The head regulator is usually aligned at an angle of 90 degree to 110 degree to the barrage axis for minimizing sediment entry into the canal.

1.4.6 Upstream floor and Crest Levels

The upstream crest floor levels in the under sluice bays may be kept at the general lowest bed level of the deep channel of the river subject to the cost of foundations and the difficulty in construction dewatering.
The upstream floor level of the remaining bays should be kept normally 0.5 to 1.0m above the upstream floor level of the under sluice bays or the general river bed level.

1.4.7 Position and Length of Divide Wall

A divide wall is constructed at right angles to the axis of the barrage or weir to separate under sluice bays from the barrage or weir bays. Under adverse flow conditions additional divide walls may be required in the weir or barrage bays.

The main function of the divide wall on the upstream side is to provide a comparatively still pocket in front of the canal head regulator resulting in deposition of sediment in the pocket and entry of clear water into the canal.

A divide wall of one-half to two-thirds the width of the head regulator normally gives satisfactory flow conditions when only one canal takes off from a barrage or a wire. In the case of more than one canal on the same bank, the divide wall should be taken up to the uppermost head regulator. Model studies are advised to determine the position and length of upstream divide wall for most effective action.

It is necessary to continue the divide wall on the downstream side to ensure adequacy of tail water depth in the under sluice bays for the formation of jump and to avoid cross flow in the close vicinity of the structure which may result in objectionable scours. The divide wall is generally extended to the end of impervious floor or to the end of loose apron on the downstream side. The exact length required may be determined on the basis of model studies.

A second pocket of river sluice, adjoining the under sluices has been found to improve the flow conditions considerably where the river curvature is not favorable to silt free entry of water into the canal by inducing convex curvature opposite the head regulator. The provision of the second pocket can also be adopted in case a wide river to guide the river to flow centrally minimize cross flow and inhibit formation of shoals near the head regulator. The location and layout of the river sluice should be decided by model studies for satisfactory performance.

1.4.7 River Training Works

River training works are required to check the out-flanking of the structure and to guide the river to flow axially through the barrage or weir. The guide banks are necessary to arrest the meandering tendency, obliquity of flow and to maintain deep channel through the under sluice bay adjacent to the canal off-take.

Proper alignment of guide bunds is essential to ensure satisfactory flow conditions on to the barrage. The most effective alignment, length and shape of guide bunds should be decided by model studies.

In case of wide alluvial banks the length and curvature of the head of guide bunds should be kept such that the worst meander loop is well away from either the canal embankment or the approach embankment. If the alluvial bank is close to the barrage, the guide bunds may be tied to it by providing suitable curvature, if necessary. If there are any out-crops of hard strata on the banks it is advisable to tie the guide bunds to such control points.

1.4.7.1 Preliminary Investigations

These investigations should include the following

- Study of available maps,
- Regional and site geology,
- Study of foundation strata,
- Study of available run-off and flood flow data,
- Assessment of water requirement,
- Availability of construction material, and
- Communication to the site of work.

1.4.7.2 Detailed Investigations

After preliminary selection of site, these investigations shall be carried out in much greater detail with a view to collecting data for the design of the main structure and its appurtenant works for the site chosen. These should include the following:

a) Detailed topographical survey, b) Collection of hydrological data, c) Sediment studies,
d) Surface and subsurface investigation including laboratory tests,

- ***Detailed Topographical Survey***

Topographical survey consisting of contour plan of the area, cross-sections and longitudinal section of the river should be carried out. conforming to IS : 6966-1973*. The survey should be plotted to suitable scale. The survey should show all the salient features like firm banks, rock outcrops deep channels, large shoals and islands, deep pools and important land marks, etc. The length of the survey may depend upon the nature of stream, the size of the barrage or weir and the purpose of diversion. If the river course on the upstream and downstream of the site is straight, the length of survey can be shortened whereas in case of meandering rivers the length of survey may be increased so as to cover at least two fully developed meanders on the upstream of the barrage axis and one meander length on the downstream or as may be required for detailed model studies.

- ***Collection of Hydrological Data***

The aim of the collection of hydrological data is two-fold, namely:
(a) for computing the design flood.
(b) for assessing the available weekly and monthly run-off on a more realistic basis.

The following data should be collected for estimating the maximum anticipated flood :

a) Rainfall—Daily rainfall recorded at different stations in the catchments area and data regarding storms in respect of successive positions of the center of the storm on the catchments should be collected. Storms causing peak discharges should be separated for unit hydrograph analysis;

b) Flood hydrographs for isolated rain storms for working out unit hydrograph;

c) Catchment characteristics, such as shape, slope, orientation, drainage system and infiltration capacity for developing synthetic hydro- graph, inadequate data are not available ;

d) Peak flow data for the river for as many years as possible for frequency analysis;

e) Flood marks by local enquiry to estimate maximum flood by slope area method; and

f) A gauge discharge site should be established at a suitable point in the vicinity of the barrage site- The gauge discharge data should be utilized to evolve a gauge-discharge curve for computing the discharges for the period for which river gauge data are available. The run-off data thus obtained should be utilized for estimating dependable yield.

- ***Sediment Studies***

Sediment observations should be started immediately with the gauge-discharge observations as soon as the project is contemplated. The quality and quantity of sediment carried by the water, especially during flood season, is necessary for planning sediment excluding or preventing works and to frame a suitable mode of regulation.

- ***Surface and Sub-surface Investigation Including Laboratory Tests***

Bore holes should be driven at specified intervals covering the barrage or weir area and appurtenant structures. The locations of borings shall be correctly marked and numbered on the survey sheets. These borings should be carried to hard rock level or to a depth of 15 to 25 m below the deepest river bed level depending on the strata and the structure (pier, abutments, floor etc).

Trial pits may be excavated to determine the depth of overburden and loose deposits, if shallow. In case of large depths of overburden comprising large size boulders where ordinary boring methods may prove inadequate, geophysical method may be employed to locate the rock surface. The following investigations should be carried out in alluvial foundations:

a) Borings at suitable locations should be made in the barrage or weir area and detailed borelogs prepared giving details of various strata encountered in accordance with 18:4464-1967*. The spacing of bore holes should be planned in a manner so as to cover the foundations of various structures.

b) For sandy foundations, dynamic and static penetration tests should be performed below each structure (abutments, piers, etc) and in each bay to estimate bearing pressures, likely settlement and necessity of settlement joints. For boulder strata, plate bearing tests shall be required.

c) Soil classification, unit weight of soil, angle of internal friction of soil, void ratio and specific gravity up to foundation level.

d) In case of clayey and silty foundations undisturbed sampling should be done and tests conducted for determination of unconfined compressive strength and consolidation characteristics.

e) Modulus of subgrade reaction in case of raft foundation.

In case of rock foundations at shallow depth, drill holes should be carried out to ascertain the depth of weathered zone, extent of joints and fissures and to determine the necessity or otherwise of grouting to avoid excessive seepage losses. Depth of impervious layer should be located in the entire barrage and head regulator area. Detailed foundation investigation comprising penetration tests should be conducted in the barrage and appurtenant structure area to mark out soft foundation area, if any, for special treatment or examining the possibility of avoiding it by suitably changing the layout.

Some other factors which influence the layout and planning are as follows:

- Availability of Construction Material
- Nature of the River Silt
- Slope of the River
- Condition of Banks
- Width of the River Bed
- Confluence of Tributaries
- Foundation Conditions
- Suitability of Site for Under sluices
- Head Reaches of the Canal
- Quantity of Floating Debris

1.4.8 Alignment

1.4.8.1 Alignment with reference to the course of the river

The alignment can be fixed in two ways with respect to direction of current viz, (i) at right angles to the course of current and (ii) Oblique to the current. Each one has got its own advantages and disadvantages

i) Alignment at right angles to the course of River

Generally the diversion structures are aligned at right angles to the general flow of the river. The advantages over such an alignment are:

(a) The unit discharging capacity would be maximum

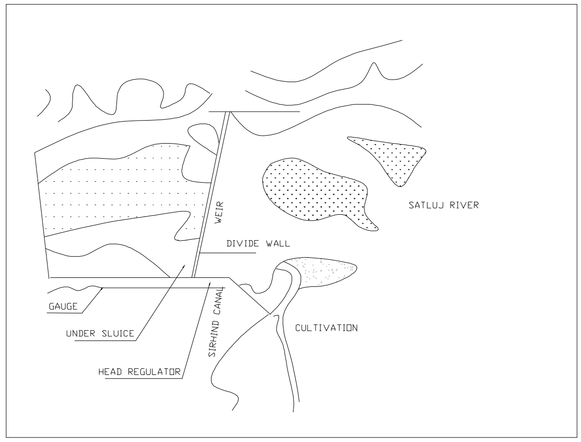
(b) The flow would be ore or less uniform over the length of structure

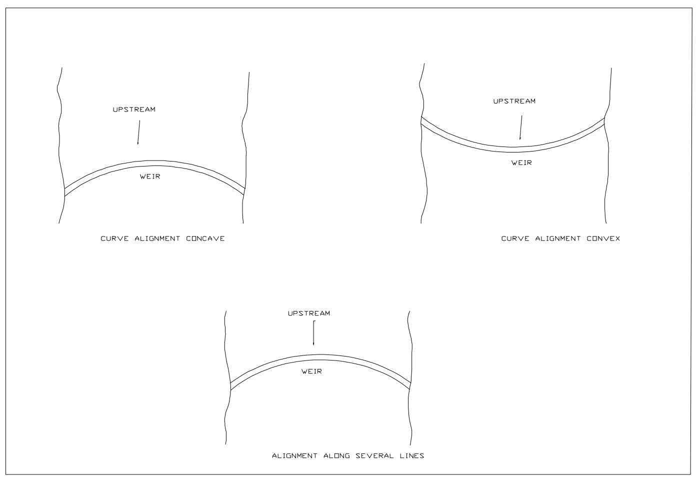
(c) It is most suited for sandy and silty foundations.

(d) It is more economical and practical

ii) Alignment Oblique to the current

Oblique alignment of diversion structure to the current is avoided as far as possible. In case, this can not be avoided due to certain other pressing reasons, the river training has to be done so as to provided uniform flow across the structure and attract the channel to the bank when canal takes off.





1.4.8.2 Alignment with Reference to Geometric Shape

The alignment of the diversion structure can be arranged in three ways, viz. (i) Straight along a single line (ii) Curved (iii) Straight or curved along two or more lines.

i) Straight Alignment in Single Line

This is the alignment generally followed in many of the diversion structures. The flow lines on the downstream of the structure would be parallel to the banks

ii) Curved Alignment

The curved alignment of the diversion structure can be had with concavity upstream or convexity upstream both as viewed from the downstream side (See Figure 4.1) The alignment with the concavity upstream is suitable when the location of the structure is proposed at a point where the river after having widened is narrowing down to normal course. The flow would converge to the center.The alignment with convexity upstream is not recommended as it diverges the current towards the banks exposing them to scour. Moreover, this type of arrangement is also weak structurally.

iii) Polygonal Alignment

This type of alignment is only a combination and modified version of the other two alignments. In this case also concavity upstream is preferable.

1.5 Hydraulic Model Studies

While some of the minor diversion structures can be located and aligned at a suitable site after considering the merits and demerits of some selected sites and also by experience, in the case of major diversion structures, recourse is generally taken to select a good and exact location in the vicinity of some selected reach and also to determine the satisfactory alignment at that particular location through the help of 3-D Hydraulic model studies. It is always difficult to select an ideal site for the location of the diversion structure and its regulators satisfying all the requirements discussed in the earlier paras. Hence it often becomes necessary to select a site satisfying most of the requirements and for the rest, some corrective measures will have to be incorporated in the layout and designs. These include properly designed guide bunds, spurs, flood protection embankments, pilot channels, silt excluding devices etc.

Chapter - 2

Barrage Design on Alluvial Soils (Hydraulic Design)

2.0 Introduction

Design of diversion structure comprises of two parts, namely hydraulic and structural design. In hydraulic design overall dimensions and profile of the main structure and a few of the components are calculated to get the satisfactory hydraulic performance of the structure. Structural design is carried to get the different sectional and reinforcement details. Fixed dimensions and layout obtained from hydraulic design is tested by model studies and the recommendations of model study are incorporated in structural design.

In hydraulic design the diversion structure has to be properly designed for both the surface and sub-surface flow condition. The design for surface flows will include the fixing of waterway, top profile of various components, energy dissipation arrangements, protection works, scour values, length and protection of divide walls, alignment, and levels and protection of guide bunds, afflux bund, etc. The design for sub surface flows will include fixing of the depth and section of cut-off, uplift pressure calculation, exit gradient, etc.

2.1 Basic Parameters Required for Hydraulic Design

The following basic parameters to be obtained before taking up detailed hydraulic designs:

(i) Design flood discharge
(ii) Afflux
(iii) Pond level
(iv) Rating curve and down stream retrogression of water levels
(v) Silt grade
(vi) Specific gravity, permeability and depth of permeable strata.

2.1.2 Design flood discharge

For purposes of design of items other than free board, a design flood of 1 in 100 year return period flood is generally adopted for important / permanent barrages. For designing free board, a minimum of 500 year frequency flood or standard project is generally adopted.

2.1.3 Afflux

The width of the barrage/weir is governed by the value of afflux (at the design flood) to be permitted and the proposed crest levels. It is also important for the design of downstream cistern, flood protection and river training works, loose protections and cut-offs. The maximum permissible value of afflux has to be carefully evaluated depending upon the river conditions upstream and after considering the back-water effect, the area of submergence and its importance.

The afflux is generally limited to 1.0 m for structures on alluvial rivers at higher reaches and 0.3 m in lower reaches. In very steep reaches of the river with boulders or rocky bed, the afflux may be safely be higher, say of the order of 2-3 meters or higher.

2.1.4 Pond level

Pond level, in the under-sluice pocket, upstream of the canal head regulator shall generally be obtained by adding the working head to the designed full supply level in the canal. The working head shall include the head required for passing the design discharge into the canal and the head losses in the regulator and losses in trash-rock if provided.

2.1.5 Rating curve and downstream retrogression of water levels

In the design of the diversion structures, the rating curve, otherwise called as Gauge-Discharge Curve, plays a very important part. Hence it is very imperative to have good rating curve of the river at the site of the structure. If no data is available on the Gauge-Discharge values, the same may be worked out by Manning's formula. This is the un-retrogressed G-D curve to be used for waterway and free-board calculations.

Silt free water flowing down the diversion structure causes degradation or retrogression of downstream bed. The lowering of bed levels affect the exit gradient and energy dissipation. If the retrogression of downstream bed is not duly allowed for in designs, it may result in failure of the structure. A value of 1.25 to 2.25 m may be adopted at lower river stages and 0.3 to 0.5 m at the design flood. For intermediate discharges, the effect of retrogression may be obtained by plotting the retrogressed high flood levels on log – log graph.

2.1.6 Silt grade

Maximum depth of scour hole along the structure at various points depends on the grade of bed material. Silt grade and silt load also influence the design of silt extractor near the canal head regulator. It also gives an indication about the looseness factor to be provided.

2.1.7 Specific Gravity, Permeability and depth of Permeable Strata

The floatation gradient or critical exit gradient is a function of specific gravity. For most of the permeable soils the specific gravity lies in between 2.6 to 2.7. The critical gradient for different soils is given in table of article 5.4.

Permeability is generally used to find the loss of water in the form of seepage. Depth of permeable strata decides what kin of cut-offs to be provided. If the depth of permeable strata is less than the scour depth requirement, a positive cut-off is provided with extensive drainage arrangement below the downstream glacis and cistern. The method of analysis to be adopted for determination of uplift pressures and exit gradient also depend on the depth of permeable strata.

2.2 Waterway Calculations

The waterway includes that of under sluice, spillway and river sluice bay, if any. In deep and confined rivers with stable banks the overall waterway between the abutments would normally be adjusted to the actual width of river at design flood level. For shallow and meandering alluvial rivers for minimizing the shoal formation, the following looseness factor shall be applied to Lacey's waterway for determining the primary value of the waterway.

Silt Factor	Looseness Factor
Less than 1	1.2 to 1
1 to 1.5	1 to 0.6

Lacey's waterway is given by the formula, $\mathbf{P=4.83\ Q^{1/2}}$
Where, Q = design flood discharge in cumecs for 100 year frequency flood.

For deciding the final waterway, the following additional considerations may also be taken into account:

a) Cost of protection works and cutoffs,
b) Repairable damage for floods of higher magnitude, and
c) Afflux constraints as determined by model studies.

With the selection of span width of each of bay and adjustment of the no. of under sluice bays, spillway bays and also fish passage required if any, the width between the abutment can be calculated by adding the thickness of single and double piers. The actual waterway thus provided is checked for its adequacy by assuming the crest level of under sluice equal to the average bed level and spillway crest is usually taken 1 to 2 m higher than the crest level of under sluice.

2.2.1 Adequacy of waterway

Adequacy of waterway is checked for passing Design flood which is found by the following formula:

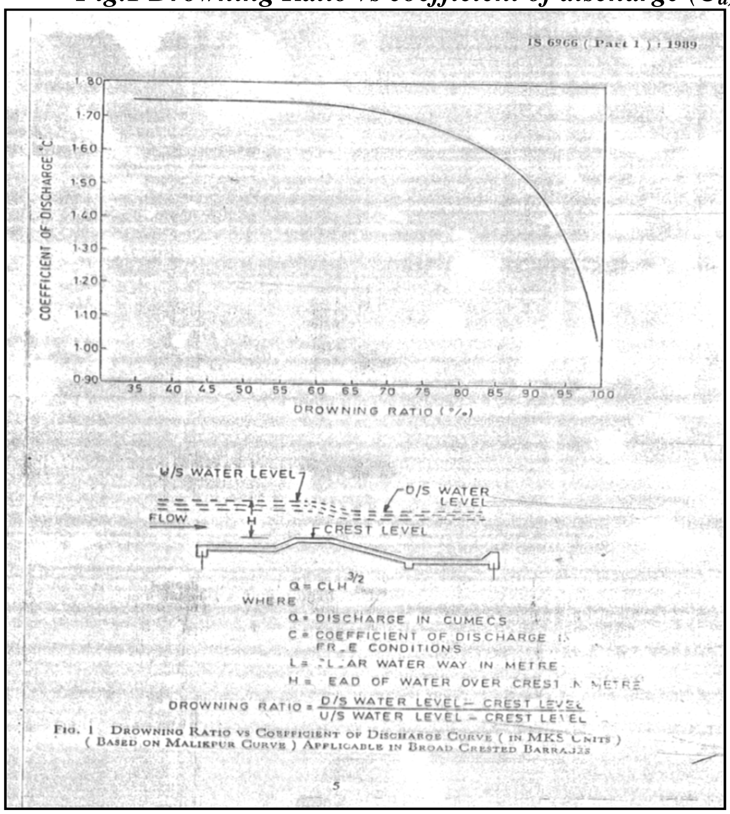
$$Q = CLH^{3/2}$$

Where,

Q = Discharge in cumecs,
C = Coefficient of discharge,
L = clear waterway of the barrage or weir in m, and
H = total head causing flow in m.

The coefficient of discharge C depends on many factors including the head over the sill, shape and width of the sill, its height over the upstream floor, roughness of its surface, and down stream water level. Based on the studies made in the fast, a curve indicating coefficient of discharge (C) *vs* drowning ratio has been developed and shown in fig 1. Drowning Ratio is the ratio of downstream and upstream water levels above the crest level. For design discharge the downstream water level can be determined from the unretrogressed Gauge-Discharge curve. Upstream waterway level is initially assumed by adding afflux to the downstream water level. Then by finding out the drowning ratio, the co-efficient of discharge is determined with help of Mallickpur curve (Fig.1).

Fig.1 Drowning Ratio vs coefficient of discharge (C_d)



Substituting the value of C,L and H, the value of discharge that can be passed through the structure can be worked out. This should be greater than the design discharge if not so process is repeated to get the near value.

From the above, it will be clear that the waterway, crest levels and afflux are all inter-linked and hence various combinations of these can be had for passing the same design discharge. The final features to be adopted in the design should be in consistent with economy and safety.

2.3 Hydraulic Jump And Energy Dissipation

Hydraulic jump results when there is a conflict between upstream and downstream controls which influence the same reach of channel. For example, if the upstream control causes a supercritical flow while the downstream control dictates a sub critical flow, then there is a conflict which can be resolved only if there is some means for the flow to pass from one flow regime to the other. The phenomenon of jumping of water from super-critical flow to sub-critical flow is known as Hydraulic jump(HJ). Hydraulic jump is generally accompanied by large scale turbulence, dissipating most of the kinetic energy of super critical flow which has got detrimental effect on the surface.

For a structure to be safe, the formation of jump should be confined to the sloping glacis and not allowed to be formed on the cistern level beyond the toe of glacis. The cistern level and its length are to be worked out for various set of condition imposed on the structure on the basis of the gate regulation proposed. The most critical condition gives the lowest cistern level and its length. These are generally determined by the use of curves available for various discharge intensities and water depths or by analytical method.

2.3.1 Basic relations of Hydraulic Jump

The different parameters of the HJ are shown in figure 2. The main unknown parameters of the HJ are the conjugate depths linked by the following formulae

$$y_1 y_2 (y_1 + y_2) = {2q^2}/{g} \text{ ....(1) and}$$

$$\mathrm{H}_L = \frac{(y_1 - y_2)^3}{4y_1 y_2} \text{..........................(2)}$$

$$E_{f1} = y_1 + \frac{q^2}{2gy_1^2} \text{..........................(3)}$$

$$E_{f2} = y_{21} + \frac{q^2}{2gy_2^2} \text{..........................(4)}$$

where y_1 and y_2 are pre and post jump depths, q is the intensity of discharge, H_L is the head loss in the jump, E_{f1} and E_{f2} are the specific energies before and after the jump.

In a hydraulic jump, there are six independent variables viz., y_1, y_2, E_{f1}, E_{f2} , q and H_L, which are interrelated by 4 equation. If any two variables are known, the remaining four can be determined. In actual design of irrigation structures, generally the discharge intensity (q) and the loss of energy H_L are known. The solutions for the cubical equations are given below.

2.3.2 Determination of Cistern Level by Analytical Method.

Various steps involved in the determination of cistern level by analytical method are as follows:

i) Estimate the d/s total energy line (D/S TEL) = d/s retrogressed water level +velocity head

ii) Then u/s total energy line (U/S TEL) = D/S TEL + afflux
The velocity head can be worked out as, h_{va} = q/R
Where, R = 1.35 $(q^2/f)^{1/3}$ for looseness factor <1, and
R = 0.475$(Q/f)^{1/3}$ for looseness factor >1

iii) Assume an arbitrary cistern level for the particular discharge.

iv) then, E_{f1} = U/S TEL – Assumed cistern level

v) from the known values of E_{f1} and q with concentration, y_1 can be calculated from the relationship,

$$E_{f1} = y_1 + \frac{q^2}{2gy_1^2}$$

vi) Calculate the pre-jump Froude,s number as $F_1 = \frac{q^2}{2gy_1^3}$

vii) From the calculated values of y_1 and F y_2 can be calculated from the relation ship,

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

viii) The required cistern level = d/s water level (retrogressed) – y_2

ix) Compare the assumed cistern level (step iii) and the actual cistern level (step ix). If they are not same, the cistern level assumed initial needs to be raised or lowered accordingly and the steps iii) to ix) are repeated till a nearer value is obtained.

x) Cistern length = $5(y_2-y_1)$.

Repeat the steps i) to x) for various discharge conditions and gate regulations. Then take the lowest value of step viii) is taken as the cistern level and the highest value of the step x) is taken as the cistern length. The above process of solving the cubical equation for y_1 is cumbersome because it involves trial and error. To simplify the laborious process the help of curves can be taken.

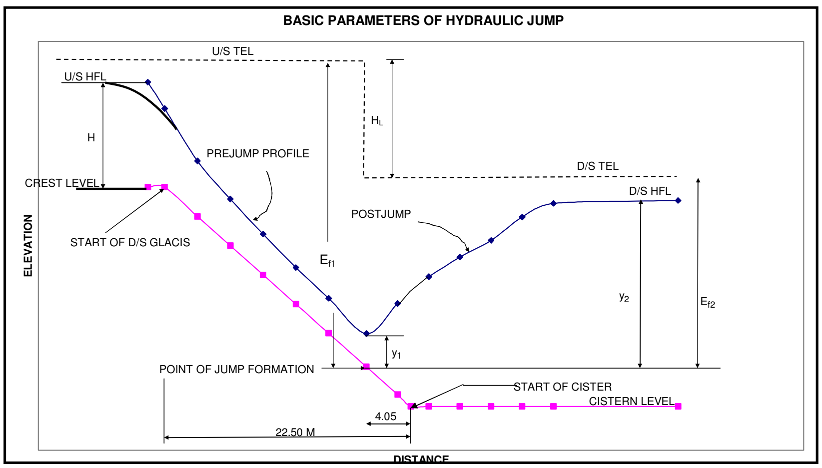


Fig 2. Hydraulic jump parameters

2.3.3 Determination of Cistern Level by the use of Curves.

a) Use of (H_L/y_c) curve: *preparation of curves*

Dividing the equations (1) and (2) an either side by y_c^3 and rearranging the terms gives dimensionless equations as follows.

$$Y_1 Y_2 (Y_1 + Y_2) = 2 \quad \text{.....................(5), and}$$

$$\frac{H_L}{Y_C} = \frac{(Y_1 - Y_2)^3}{4Y_1Y_2} \text{...........................(6)}$$

where

$Y_1 = y_1/y_c$, which is always less than 1, and $Y_2 = y_2/y_c$, which is always greater than 1 .

Assuming the value for Y_1 the value of Y_2 can be worked out easily from equation (5) and substituting the values of Y_1, Y_2 in esq. (6), the value of H_L/Yc can be determined. Then prepare a curve for Y_1 vs H_L/Yc.

Solution process

i) The critical depth is obtained from $y_c = (\frac{q^2}{g})^{1/3}$

ii) Head loss H_L = (U/S TEL) - D/S TEL.

iii) Work out the actual value of H_L/Yc , and read the value of Y_1 and Y_2 from the curve earlier prepared.

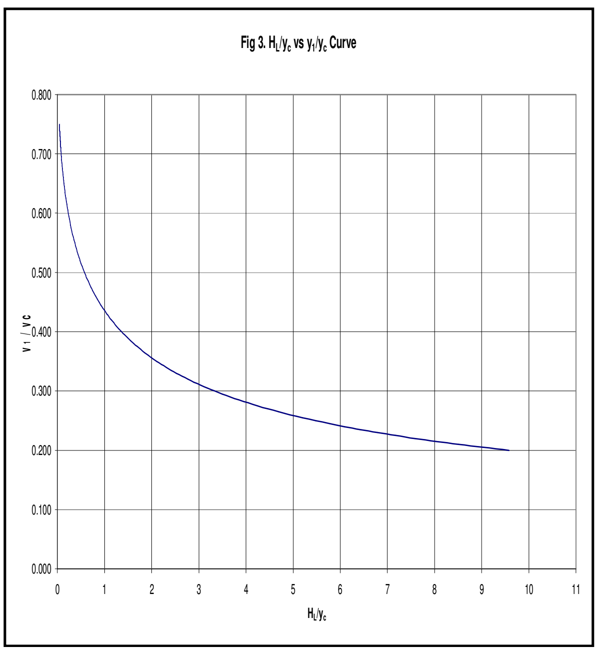
iv) Then $y_1 = y_c\, Y_1$ and $y_2 = y_c\, Y_2$

then rest of procedure for determining cistern level and length is same as analytical method.

b) **Use of Blench and Montague Curves**

For the determination of cistern levels, a set of curves known as Blench Curves and Montague curves are generally used. For different cases and their discharge values the procedure for determining the HJ parameters is as follows.

i) The procedure is same as above analytical method except the finding of E_{f2} and y_1, y_2. From the known values of the intensity of discharge with concentration and the head loss, using Blench curve (fig. 4), the values of E_{f2} (energy of flow d/s of the jump formation) can be obtained for different discharges. E_{f1}, i.e., energy of flow just upstream of the point of the jump formation is found by adding H_L. From the known values E_{f1} and the intensity of discharge, using Montague Curves (fig.4), the values of y1 and y2 i.e., the hypercritical and sub-critical depths of water before and after the hydraulic jump can be read out.



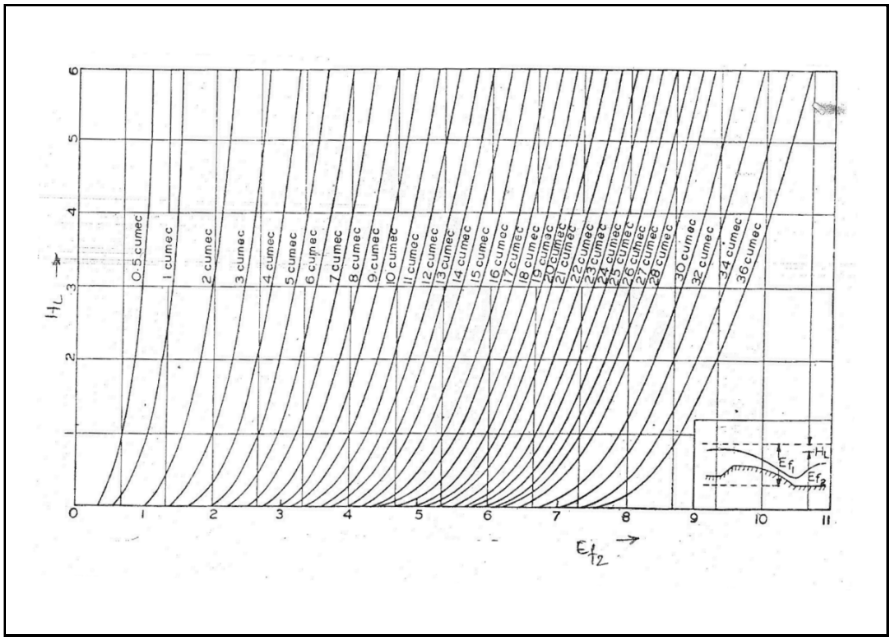


Fig 4 (Blench curves)

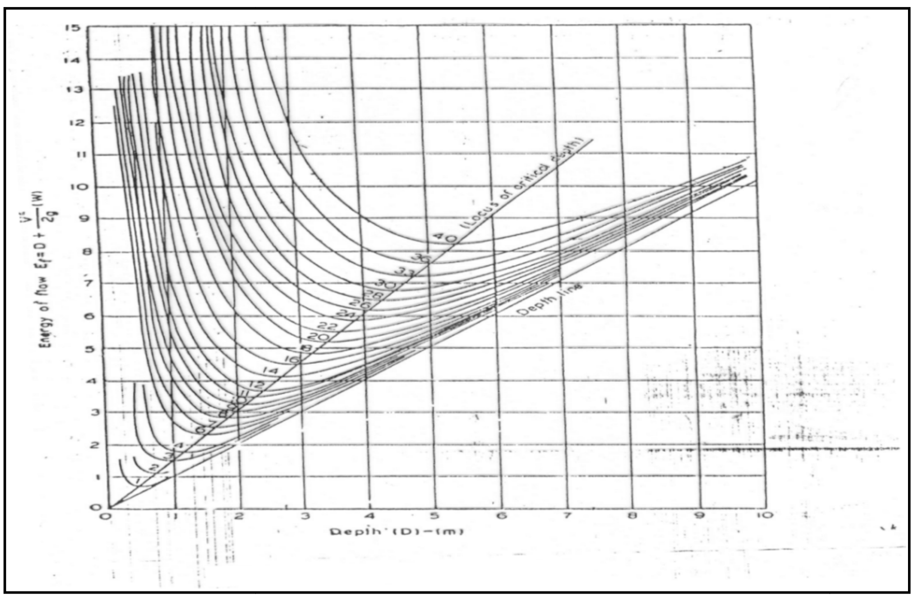


Fig.5 (Montague curve)

Comparison in determining the conjugate depths and stilling basin parameters is given in the below table for the following data.

u/s TEL is +323.00, d/s TEL is +321.00, intensity of discharge with concentration, q is 22 m^3/s/m and d/s retrogressed water level is +320.80.

Sl No.	description	Analytical method	H_L/y_c curve	Blench and Montague curves
1	Cistern level	+314.50	+314.393	+314.400
2	Cistern length	21.47 m	22.72 m	23.25 m
3	Limitation on Intensity of discharge(q)	Works for any q	Works for any q	Max q is 36
4	accuracy	most	moderate	least
5	cumbersome	most	less	least

From the a above comparison it can be concluded that for preliminary designs Blench and Montague curves can suitable used but for accurate values analytical method or H_L/y_c table are appropriate.

2.4 Theories of Sub-Surface flow

Whenever a structure is founded on a pervious foundation, uplift pressure will be exerted beneath the structure by seeping water, in addition to all other forces. The water seeping below the body of the hydraulic structure endangers the stability of the structure and may cause its failure either by uplift or by undermining.

2.4.1 Bligh,s creep theory

The failure of hydraulic structures due to sub-surface flow were introduced by Bligh on the basis of experiments and the research work conducted after the failure of Khanki Weir, which was designed on experience and intuition.

According to Bligh's theory, the percolating water follows the outline of the base of the foundation of the hydraulic structure. The length of water thus traversed by water is called the creep length. He assumed in his theory that the loss of head is proportional to the length of the creep. If H_L is the total head loss between u/s and d/s, and L is length of the creep, then the loss of head per unit of creep length (i.e. H_L/L) is called the hydraulic gradient. Bligh made no distinction between horizontal and vertical creep.

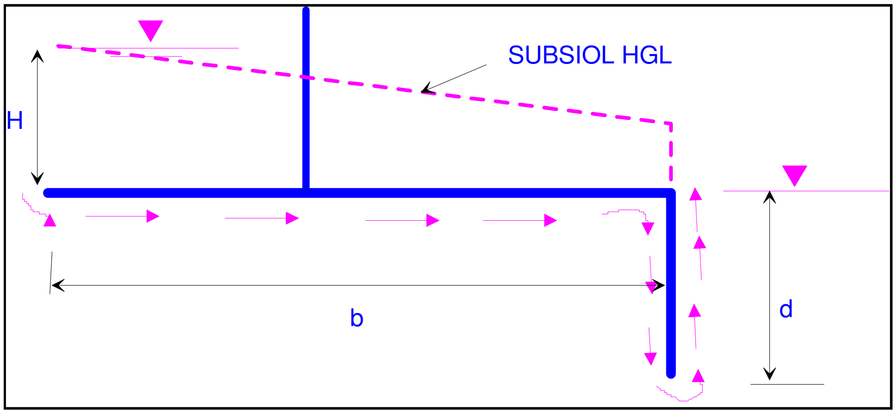
Let us consider a section as shown below:

Water will seep along the bottom contour as shown by arrows. It starts percolating at A and come out at B.

Total creep length, L = b+d+d
= b+2d
Head loss per unit length or hydraulic gradient
= H_L/ (b+2d)
= H_L/L

According to Bligh, the safety against piping can be ensured by providing sufficient creep length given by L= CH_L, where C is the Bligh's co-efficient of the soil. Different values of C for different type of soils are given below:

Fig. 6 *Bligh,s creep theory*



Values of Bligh's Safe Hydraulic Gradients

S.NO.	Type of Soil	Value of C	Safe Hydraulic Gradient should be less than
1	Fine micaceous sand	15	1/15
2	Coarse grained sand	12	1/12
3	Sand mixed with boulder and gravel, and for loam soil	5 to 9	1/5 to 1/9
4	Light sand and mud	8	1/8

Safety against uplift pressure: The ordinate of HGL above the bottom of floor gives the residual uplift at that point. If h in meter is the ordinate measured from the base of the floor then water pressure equal to h will act at that point and has to be counterbalanced by the weight of floor of thickness, t.

Uplift pressure = wh, where w is the density of water
Downward pressure = ***(wG)*** x **t**, where G is the specific gravity of floor
For equilibirium
W h = wGt
h = Gt
or residual pressure, h-t=Gt-t= t(G-1)
or the minimum thickness of the floor, t = (h-t)/(G-1)

Where h-t is the ordinate of the HGL above top of floor. Hence the thick of floor is determined using this formula and it is generally increased by 33% so as to allow the suitable factor of safety.

Limitations of Bligh,s creep theory;

The main limitations of Bligh,s theory are summarized below.

i) Bligh,s theory gives same weightage to the horizontal and vertical creep. But actually the vertical creep is more effective than the horizontal creep.

ii) The theory assumes that the head loss variation is linear, but it is nonlinear.

iii) The theory does not emphasis the importance of the d/s cut off without which piping failure occurs.

iv) The theory does not give any theoretical or practical method for the determination of the creep coefficient C. it has to be determined from experience or from actual observation. Because of the above limitations, Bligh,s theory is not used for the design of important irrigation structures. However it can be used for the preliminary designs.

2.4.2 Lane's Weighted Creep Theory

Based on the statistical analysis of observed data at the then existing barrages, Lanes concluded that horizontal creep is less effective in dissipating uplift than the vertical creep. He therefore, suggested a weightage factor of 1/3 for the horizontal creep, against 1.0 for the vertical creep. Thus for the example cited before Lane;s creep length is given by

$$L_l = 1/3(b) + 2\ d$$

To ensure safety against piping, according to this theory, the creep length L_l must not be less than C_1H_L , where H_L is the head causing flow and C_1 is Lane's co-efficient.
Table given below shows the Lane's Safe hydraulic gradient:

S.No.	**Type of Soil**	**Value of Lane's co-efficient, C_1**	**Safe Lane's Hydraulic gradient should be less than**
1	Very fine sand or silt	8.5	1/8.5
2	Fine sand	7.0	1/7
3	Coarse sand	5.0	1/5
4	Gravel and sand	3.5 to 3.0	1/3.5 to 1/3
5	Boulders, gravels and sand	2.5 to 3.0	1/2.5 to 1/3
6	Clayey soils	3.0 to 1.6	1/3 to 1/1.6

Lane's theory is having only a theoretical importance.

2.4.3 Khosla's Theory

Many of the important structures such as weirs and barrages were designed on the basis Bligh's theory between the periods of 1910 to 1925. In 1926-27, the Upper Chenab canal Siphons, designed on Bligh's theory, started posing undermining troubles. Investigation started and which ultimately led to Khosla's theory.

Khosla propounded that seeping water does not flow through the bottom contour of pucca floor as suggested by Bligh but this moves along a set of stream-lines. Every particle entering the soil at a given point upstream of the structure will come out by its own path and will represent a streamline. Khosla's theory of flow nets made it very clear that the loss of head does not take place uniformly in direct proportion to creep length as stated by Bligh. It depends upon the whole geometry of the structure, i.e. the shape of foundations, depth of impervious boundary and levels of u/s and d/s beds. He stated that safety against piping can not be obtained by providing sufficient floor length, as stated by Bligh, but can be obtained by keeping the exit gradient well below the critical value.

He further stated that undermining starts only when the exit gradient is unsafe for the soil on which it is founded and it is necessary to have a deeper d/s cut-off to prevent undermining. The depth of d/s cut-off is determined by 1). Maximum depth of scour
2). Safe exit gradient.

While designing a weir, d/s cut-off from the maximum scoured depth consideration is first taken and checked for exit gradient. If a safe value of exit gradient is not found then the depth of cut-off is increased. Barrage might also fail due to surface flow (i.e. when flood water flows) may cause scour, dynamic action; and in addition, will cause suction pressures in jump trough. These uplift pressures must be checked for different condition of flow. The maximum uplift due to this dynamic action (i.e. for surface flow); should be compared with the maximum uplift under steady seepage (i.e. subsurface flow); and the maximum of the two chosen for designing the aprons and the floor of the Barrages.

2.4.3.1 Khosla's solution for a horizontal floor

Khosla and his associate gave the mathematical solution of the Lapace equation for different types of structures on the permeable foundation.

$$\textit{Lapace equation}\ \frac{\partial^2\phi}{\partial x^2}+\frac{\partial^2\phi}{\partial y^2}=0\ \text{, where } \phi=-kh$$

The solution of the equation for a simple structure with an impervious horizontal floor as shown in fig 7. gives that if the depth of permeable strata is infinite the flow lines become confocal ellipses and the equipotentioal lines become confocal hyperbolas. The residual pressure or uplift pressure at any point x is given by

$$P=\frac{H}{\pi}Cos^{-1}(2x/b)$$, where H is head causing flow, b is length of impervious floor and x is measured from the center of the floor (+ve d/s and –ve u/s). The uplift pressure distribution is shown in fig 7. it may be noted from the fig that i). In the d/s half of the floor, the Bligh theory under-estimates the uplift pressure and hence it is unsafe, ii). The actual uplift pressure distribution is a cosine curve and not a linear, as assumed by Bligh and iii). The hydraulic gradient at d/s exit is theoretically infinite which absolutely necessitates a pile at d/s end to prevent failure against piping.

2.4.3.2 Khosla's method of independent variable

To know the behavior of seepage flow for a given boundary conditions, it is required to plot flow net. For that we need to solve the flow net either by electrical analogy method or mathematical solution of Laplacian equations. These are complicated method and for this Khosla suggested a simple, quick and accurate method of Independent Variables.

In this method, a complex profile like that of barrage is divided into a number of simple profiles, each of which can be solved mathematically. He presented the solution of flow nets for these simple standard profiles in the form of curves that can be used for finding out percentage pressures at different key points. The simple profiles which are most useful are:

i) A straight horizontal floor of negligible thickness with a sheet pile line on the u/s or d/s end (Fig.8 a and b) .

ii) A straight horizontal floor depressed below the bed but without any cut-off(Fig.8c)

iii) A straight horizontal floor of negligible floor thickness with a sheet pile line at some intermediate point. (Fig. 8 d)

The key points are the junction of the floor and the pile lines on either side, and the bottom point of the pile line, and the bottom corners in the case of a depressed floor. The percentage pressure at these key points for the simple forms into which the complex profile has been broken is valid for the complex profile itself, if corrected for

(a) Correction for the mutual interference of piles;
(b) Correction for thick. of floor ;
(c) Correction for the slope of the floor.

(a) Correction for the Mutual Interference of Piles. The correction C to be applied as the percentage of head due to this effect, is given by

$$C = \pm 19\left(\frac{d+D}{b}\right)\sqrt{\frac{D}{b'}}$$

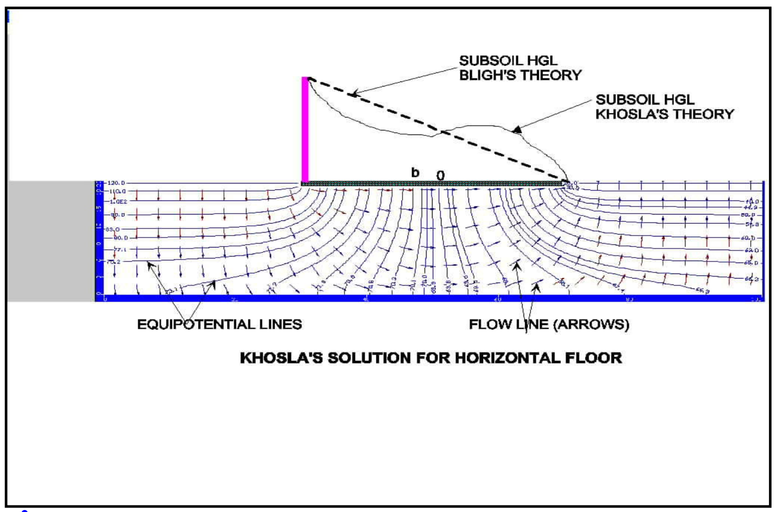
Where b' is the distance between the two pile lines, D is the depth of the pile line, the influence of which has to be determined on the neighboring pile of depth d. D is to be measured below the level at which interference is desired, d is the depth of the pile line on which the effect is considered and b is the total floor length.

This correction is positive for the points in the rear or back water and subtractive for the points forward in the direction of flow.

(b) **Correction for the thickness of floor**

In the standard form profile it is assumed that the floor has negligible thickness. So percentage pressure calculated by Khosla's graphs shall pertain to the top levels of the floor. But actual junction point E and C (Fig. 6) are at the bottom of the floor. So the pressures at the actual points are calculated by assuming a straight line pressure variation. Since the corrected pressure at E_1 should be less than the calculated pressure at E'_1, the correction to be applied for the point E_1 shall be –ve. Similarly, the pressure calculated at C'_1 is less than the corrected pressure at C_1, and so. The correction to be applied at point C_1 is +ve.

Fig 7. KHOSLA'S SOLUTION FOR HORIZONTAL FLOOR



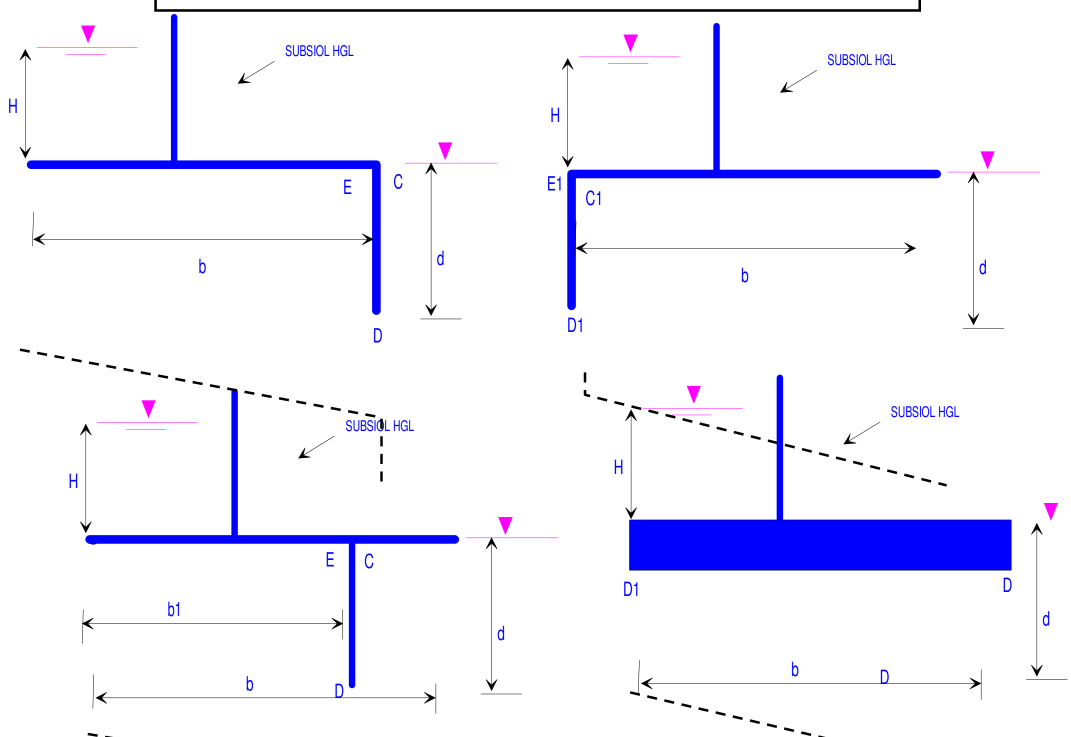


Fig. 8 (a) to (d): Khosla's simplified standard profiles

(c) Correction for the slope of the Floor

A correction is applied for a sloping floor, and is taken as +ve for the down and –ve for the up slopes following the direction of floor. Correction factor to be applied for slope is tabulated below:

Slope Horizontal:Vertical	**Correction factor**
1:1	11.2
2:1	6.5
3:1	4.5
4:1	3.3
5:1	2.8
6:1	2.5
7:1	2.3
8:1	2.0

The correction factor is multiplied by the horizontal length of the slope and divided by the distance between the two pile lines between which the slope is located.

2.4.3.4 Exit Gradient

For a floor of length b with a vertical cut-off of depth, the exit gradient at its downstream end is given by :

Type of Soil	**Safe exit gradient**
Shingle	1/4 to 1/5
Coarse Sand	1/5 to 1/6
Fine Sand	1/6 to 1/7

$$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$$

where

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}, and ... \alpha = {}^{b}\!/\!_{d}$$

The exit gradient calculated should lie within safe limits as given in Table below:

2.5 Modeling of seepage flow for finite depths of permeable strata

Khosla's theory of seepage flow assumes that depth of permeable strata is infinite. In real problem, infinite depth of alluvium never exists. But extensive electrical analogy model studies have shown that the infinite depth of pervious strata will give practically the same results as those obtained by assuming the pervious strata depth equal to five times the depth of cut-off or two and half times the length of barrage, whichever is larger. If the above condition is not satisfied the Khosla's theory gives erroneous values. In that case the problem of seepage flow is solved by mathematical modeling or physical modeling.

Typical Hydraulic Design of a Barrage

Solved example

1) Waterway fixation for design flood

Design flood discharge, Q = 12172 cumecs
D/s HFL from G&D curve = 43.80 m
Pond level required = 44.00 m
Average river bed level(spillway portion) = 34.00 m
Average river bed level(under sluice portion) = 34.00 m

Lacey's waterway required = $4.83\sqrt{Q}$

= $4.83\sqrt{12172}$ = 533.00m

Waterway provided is:

25 nos. spillway bays @ 12.00 = 300.00m
8 nos. under sluice bays @ 12.00 = 96.00m
19 piers @ 2.40m thickness = 45.60m
7 nos. of double pier/divide wall @ 4.80m = 33.60m
1 fish ladder @ 2m = 2.0m

Total waterway (L) = 494.00m

∴Looseness factor = Total width between abutments / Lacey's regime width
=494.0/533
=0.93

a) check for discharge capacity of waterway provided

Scour depth, R = $1.35\left(q^2 / f\right)^{1/3}$ as looseness factor<1
(Clause 19.1 of IS: 6966(part1):1989)
f = 1.48
q=24.64 cumec/m
R = 10.61m
Velocity of approach **Va** = Q/(L.R)
= 12172 / (494x10.61)
= 2.32 m/s
Head due to velocity of approach, **Ha** = $Va^2/2g$
= $2.32^2/19.62$
= 0.275 m
D/s HFL from G&D curve = 43.8
Assume afflux = 0.6m
∴ U/s HFL = 43.8+0.6 = 44.4m

i) ***Discharge through spillway***

Crest level of spillway = 35.00

Drowning ratio = d/s HFL - crest level of spillway

u/s HFL - crest level of spillway
= 0.94

Corresponding coefficient of discharge, C, read from *fig1 of IS: 6966(part1):1989* is 1.37

Head over crest, **H** = u/s HFL - crest level
= 44.4-35.00
= 9.4 m
Ha = 0.275 m **(as determined earlier)**

Clear width of waterway through all spillways, Ls = 25x12
= 300m

Using the equation,

Q = C. Ls[(H+Ha)3/2 - Ha3/2]
Discharge through spillway **Qspl** =12273 cumecs

ii) ***Discharge through under sluice***

Crest level of spillway = 34.00

Drowning ratio = d/s HFL - crest level of under sluice

u/s HFL - crest level of under sluice
= 0.94

Corresponding coefficient of discharge, Cd, read from fig1 of IS: 6966(part1):1989 is 1.37

Head over crest, **H** = u/s HFL - crest level
= 44.4-34.00
= 10.4 m
Ha = 0.275 m **(as determined earlier)**

Clear width of waterway through all under sluice bays, L_u = 8x12
= 96m

Using the equation,

$$\mathbf{Q = Cd.\ B.\ [(H+Ha)^{3/2} - Ha^{3/2}]}$$

Discharge through under sluice, Qus = 4435 cumecs

Total discharge which can be passed through the system,

Qt = Qus + Qspl

= 4435+ 12273
= 16708 >12172 design discharge

Hence the **design afflux = 0.60 m**

2.0 Energy Dissipation and Design of Hydraulic Jump Type Stilling Basin

2.1 *Stilling basin design for spillway bays*

Discharge through spillway, Qspl = 12273 cumecs***(as determined earlier)***

Clear width of waterway	=	25 x12
	=	300 m
Intensity of discharge,**q**	**=**	12273/300
	=	40.91cumecs/m
qwith 20% concentration	=	49.09 cumecs/m

Table 1 Tail water Rating curve(G-D Curve) at 200m d/s of Barrage site

Discharge	TWL	Retrogression	Retrogressed TWL
12172	43.80	0.5	43.30
9129	42.52	0.8	41.72
6086	40.92	1.2	39.72
3043	38.68	1.75	36.93

i) cistern parameters for design flood discharge

retrogression for design flood = 0.5m
(Read from the retrogressed rating curve)
U/s W.L. = 44.40m (HFL+afflux)
D/s W.L. = 43.80m (HFL Before Construction of Barrage)
D/s Retrogressed W.L. = 43.30 m
Head loss, H_L = $(D_2-D_1)^3/4D_1D_2$ =1.10m (U/s WL-D/s WL)
qwith 20% concentration = 49.09cumecs/m

Solving the head loss and discharge relations with conjugate depths,

Initial depth, D_1 = 3.93m, and
Sequent depth, D_2 = 9.38 m

Froude's Number is given by:

$$\mathbf{F^2 = q^2/ (g\, D_1^3)}$$

Where
F = Froude's Number
q = Discharge intensity = 49.09 cumecs/m
g = acceleration due to gravity = 9.81 m/s ^{2}
D_1 = Pre-jump depth = 3.93m

∴ F =2.01

Cistern length = $5x(D_2-D_1)$
= $5x(9.38-3.93)$ = **27.30m**
Required cistern level = D/s retrogressed level - D_2
D/s Retrogressed W.L. = 43.3 m
Hence,
Required **cistern level** = **33.91**

Similarly for different discharges with partial gate opening, the cistern parameter are tabulated as below

Table 2: Spillway cistern parameters

Discharge, q (1)	U/s WL (2)	D/s WL (3)	H_L (4)	D_1 (5)	D_2 (6)	F (7)	L_b (8)	Cistern Level (9)
49.09	44.4	43.30	1.10	3.93	9.38	2.01	27.30	33.91
36.82	44	41.72	2.28	2.74	8.76	2.59	30.16	32.95
24.55	44	39.72	4.28	1.68	7.74	3.59	30.34	31.97
12.27	44	36.93	7.07	0.788	5.863	5.6	25.37	31.07
49.09	44	43.3	0.7	4.203	8.919	1.82	23.58	34.38

Minimum cistern level required = 31.07

*Hence provide the basin floor level at **30.80***

MaximumBasin length required is 30.34m, hence provide the basin length of ***44.90m*** form the toe of d/s glacis to the end of end sill.

2.2 Stilling basin design for under sluice bays

Discharge through under sluice, Qus = 4435 cumecs**(*as determined earlier*)**

Clear width of waterway = 8x12
= 96 m
Intensity of discharge, q **=** 4435/96
= 46.2cumecs/m
qwith 20% concentration = 55.44 cumecs/m

Design discharge with HFL condition and for different discharges with partial gate opening, the cistern parameter are tabulated as below

Table 3: Under sluice cistern parameters

Discharge, q (1)	U/s WL (2)	D/s WL (3)	H_L (4)	D_1 (5)	D_2 (6)	F (7)	L_b (8)	Cistern Level (9)
55.44	44.40	43.3	1.10	4.31	10.08	1.97	28.84	33.21
41.58	44	41.72	2.28	3.02	9.39	2.53	31.89	32.32
27.72	44	39.72	4.28	1.86	8.28	3.48	32.10	31.43
13.86	44	36.93	7.07	.878	6.25	5.38	26.89	30.67
41.32	44	43.3	0.7	4.60	9.58	1.79	24.93	33.71

Lowest cistern level required = 30.67

Provide cistern level at +29.50

Maximum cistern length required is 32.10m; provide the basin length of 44.00m from the toe of d/s glacis to the end of end sill.

3.0 Determination of Scour depth and Cut-off levels

3.1.0 Spillway bays

Maximum Discharge For spillway = 12273 cumecs

Maximum Discharge with 20% concentration Q =14727cumecs

Clear water way 25No.@ 12m = 300 m

Piers 19 No.@ 2.4m = 45.60 m

Double Pier / Divide wall 6 nos @ 4.8 = 28.80 m

Total water way = 374.4 m

q = 39.33 cumec/m

Scour Depth **R** is given by:

$$\mathbf{R = 1.35\ (q2/f\ )^{1/3}} \quad \text{for Looseness factor} < 1$$

(as per clause 19 of IS: 6966-1989)

Here, Silt factor f = 1.25

Hence R = 14.50 m

3.1.1 Bottom elevation of u/s cutoff

U/s Water Level = 44.40m

Bottom level of u/s cutoff = u/s W.L. - R

= 29.90

Here, u/s floor level = 34.00

Provide the u/s cut off up to **EL 29.00**

3.1.2 Bottom elevation of d/s cutoff

Taking the scour factor as 1.25 for downstream (clause 17of IS: 6966(part1):1989

1.25 R =18.12m =R1

D/s Retrogressed W.L. = 43.30

Bottom level of d/s cutoff = d/s retrogressed W.L. – R1

= 25.18

Provide the d/s cutoff up to **EL 24.00**

3.1.3 Check of d/s cut-off against exit gradient

Given safe exit gradient for the particular type of soil at site =0.19

Exit gradient Ge is given by:

$$G_E = \frac{H}{d} x \frac{1}{\pi\sqrt{\lambda}}$$

Where $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$ and $\alpha = \frac{b}{d}$

Depth of d/s cutoff d = End sill level - d/s cutoff level

= 32.50-24.00

= 8.50m

U/s to d/s length of barrage, b = 80m

$\alpha = \frac{80}{8.50} = 9.41$, $\lambda = 5.23$

Head causing flow, H = 44.00-32.50

= 11.5m

Exit gradient, G_E = 0.188< 0.19

Hence safe

3.2.0 *Under sluice bays*

Maximum Discharge For under sluice = 4435 cumecs

Clear water way 8No.@ 12m = 96 m

Piers 7 No.@ 2.40m = 16.80 m

Double pier 1 no. @ 4.80m = 4.80m

Total water way = 117.60m

qwith 20% concentration = 45.25cumec/m

Scour Depth **R** is given by:

$R = 1.35 (q2 / f)^{1/3}$ for Looseness factor <1

Hence R =15.91m

3.2.1 Bottom elevation of u/s cutoff

U/s Water Level = 44.4m

Bottom level of u/s cutoff = u/s W.L. - R

= 28.49

Provide u/s cut-off up to **EL 27.50**

3.2.2 Bottom elevation of d/s cutoff

Taking the scour factor as 1.25 for downstream

(as per clause 17 of IS: 6966(part1):1989)

1.25 R =19.89 =R1

D/s Retrogressed W.L. = 43.30

Bottom level of d/s cutoff = d/s retrogressed W.L. – R1

= 23.41

*Provide d/s cut off up to **EL 22.00***

3.2.3 Check of d/s cut-off against exit gradient

Given safe exit gradient for the particular type of soil at site =0.19

Exit gradient Ge is given by:

$$G_E = \frac{H}{d} x \frac{1}{\pi\sqrt{\lambda}}$$

Where $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$ and $\alpha = \frac{b}{d}$

Depth of d/s cutoff d = End sill level - d/s cutoff level

= 31.75-22

= 9.75m

U/s to d/s length of barrage, b = 80m

$\alpha = \frac{80}{9.75} = 8.2$, $\lambda = 4.63$

Head causing flow, H = 44.00-31.75

= 12.25m

Exit gradient, G_E = 0.185 < 0.19

Hence safe

4.0Flexible Protection works on u/s and d/s of barrage:

4.1 Spillway bays

Maximum Discharge For spillway = 12273cumecs

Clear water way 25N0.@ 12.0m = 300m

Piers 19 N0.@ 2.4 m = 45.60 m

Double pier 6 nos. @ 4.8m = 28.80m

Total Waterway of Spillway = 374.4m

Scour Depth **R** is given by:

$R = 1.35 (q2 / f)^{1/3}$ for Looseness factor <1

(as per clause 19 of IS: 6966-1989)

q=12273/374.4 =32.78cumec/m

Silt factor f =1.25

Scour depth without concentration, R = 12.84m

(Clause 20.3.2 of IS: 6966-1989)

4.1.1 U/S protection works

For u/s , scour depth is taken as 1.5R (Table 1 of IS: 6966-1989)

∴ 1.5R =19.25m

Bottom level of scour hole = 44.40-19.25

= 25.15

4.1.1.1 C.C. Blocks

Depth of scour below floor level, **D** = 34.00-25.15

= 8.85m

Provide 6*nos 1.50m x 1.50m x 0.90m C.C.Blocks, length =6 x 1.5= 9.0 > 8.85m*

Hence ok

4.1.1.2 Loose stone protection

The launching apron is assumed to launch in a slope of 2H: 1V

∴ Sloped length $= D\sqrt{5}$ =19.80 m

Quantity of loose stone with 1m thick on launching slope = 19.80m3

Required length of loose stone apron with 1.5m thick=13.20m

Provided length of loose stone apron =**14.00m**

4.1.2 D/S protection works

For d/s, scour depth is taken as 2.0R (Table 1 of IS: 6966-1989)

∴ 2.0R =25.67m

Bottom level of scour hole = 43.30-25.67 (d/s retrogressed water level-2.0R)

= EL 17.63m (deepest scour level)

4.1.2.1 C.C. Blocks

Depth of scour below floor level, **D** =32.50-17.63 (end sill level – deepest scour)

= 14.87m

Length of block protection required is 1.5D = 22.30m

Provide C.C.Blocks of1.50m x 1.50m x 0.90m as follows:

(i) 9rows over inverted filter of 0.6m thick with 75mm jharries

= 9x 1.575 + 0.075 =14.25m

(ii) Curtain wall-1 = 1m

(iii) 5 rows over stone spalls of 0.6m thick with 75mm jharries

= 5 x 1.575 + 0.075 = 7.95m

(iv) Curtain wall-2 =1m

Total length provided =24.20m ,Hence O.K.

4.1.2.2 Loose stone protection

The launching apron is assumed to launch in a slope of 2H:1V

$\therefore$ Slope length $= D\sqrt{5}$ =33.25m

Quantity of loose stone with 1m thick on launching slope = 33.25m3

Required length of loose stone apron with 1.5m thick=22.16m

Provided length of loose stone apron =23m

4.2 Under sluice bays

Maximum Discharge For spillway = 4435cumecs

Clear water way 8N0.@ 12.0m = 96m

Piers 7 N0.@ 2.4 m = 16.80m

Double pier 1 nos. @4.80m = 4.8m

Total Waterway of Spillway = 117.6m

Scour Depth **R** is given by:

$R = 1.35 (q2 / f)^{1/3}$ for Looseness factor <1

Scour depth without concentration, R = 14.09m (clause 20.3.2 of IS: 6966-1989)

4.2.1.0 U/S protection works

Bottom level of scour hole = 44.4-1.5 x 14.09

= EL 23.26 m

4.2.1.1 C.C. Blocks

Depth of scour below floor level, **D** = 34.00-23.26

= 10.74

Provided 9 nos of 1.50m x 1.50m x 0.90m C.C.Blocks, = 9 x1.5

Total length provided = 13.50m

4.2.1.2 Loose stone protection

The launching apron is assumed to launch in a slope of 2H:1V

$\therefore$ Slope length $= D\sqrt{5}$ =24.01m

Quantity of loose stone with 1m thick on launching slope = 24.01m3

Required length of loose stone apron with 1.5m thick= 16.01m

(Loose stone apron of under sluice bays is merged with launching apron of divide wall and hence the length of loose stone apron is much more than the required 16mlength)

4.2.2.0 D/S protection works

For d/s, scour depth is taken as 2.0R (Table 1 of IS: 6966-1989)
$\therefore$ 2.0R = 28.19
Bottom level of scour hole = 43.3-28.19
= EL 15.11m

4.2.2.1 C.C. Blocks

Depth of scour below floor level, **D**=31.75-15.11 (end sill level – deepest scour)
= 16.64m
Length of block protection required is 1.5D = 24.96m

Provide C.C.Blocks of1.50m x 1.50m x 0.90m as follows:
(i) 9rows over inverted filter of 0.6m thick with 75mm jharries
= 9x 1.575 + 0.075 =14.25m
(ii) Curtain wall-1 = 1m
(iii)9 rows over stone spalls of 0.6m thick with 75mm jharries
= 9 x 1.575 + 0.075 = 14.25m
(iv) Curtain wall-2 =1m
Total length provided =30.50m,Hence O.K.

4.2.2.2 Loose stone protection

The launching apron is assumed to launch in a slope of 2H:1V
$\therefore$ Slope length $= D\sqrt{5}$ =37.20m
Quantity of loose stone with 1m thick on launching slope = 37.20m3
Required length of loose stone apron with 1.5m thick= 24.80m
Provided length of loose stone apron is more than 25m which is merged with launching apron of divide wall.

(3) Structural Design of components of Barrage

After the hydraulic design of barrage, the model studies are conducted and if satisfactory hydraulic conditions are observed then structural design of barrage is taken up. This is the detailed engineering stage and whatever design assumptions were made at the time of planning stage must conform to the actual site conditions and the.

Basically a barrage consists of components mainly cutoff/ sheet pile, RCC floor / gravity floor, piers, abutments, flank wall / wing wall / return wall, flared out wall, u/s and d/s protection works, road bridge, hydro mechanical accessories, guide bunds , afflux bunds. Once the hydraulic parameters are finalized and sizing of various components is done then the stability analysis of the barrage raft is done to check is safety against sliding.

Stability Analysis

One bay of the barrage is analyzed for stability and all the forces acting on the one bay e.g. raft, pier, road bridge, gate bridge, uplift, hydrodynamic forces, water pressures (both from u/s and d/s), boxed sand, inertia forces are summarized as vertical downward force (N) and horizontal force (H)

$$\text{Factor of safety against sliding} = \frac{C + N \tan \phi}{H}$$

Pier

In barrages, piers are generally constructed as RCC structure and constructed monolithically with the raft. The thickness of pier generally varies from 1.5m to 2.5m and is dependent on the loads and moments being transferred to the raft. The height of pier on the u/s of gate is generally kept above the pond level or HFL. Below the road bridge, care shall be taken that bearings are not submerged and on the d/s side the height of pier is provided for the consideration of counteracting the uplift requirement. Differential head moment between the adjacent bays is a major moment to be considered. In the block-out portion the minimum thickness of pier (apart from block-out thickness) shall be 600mm. A double pier is provided to separate one type of foundation strata from another type (e.g. sandy and rocky) or to provide expansion joint after a particular length of raft. After analyzing various downward forces (V) and moments (M) eccentricity is calculated and the reinforcement is calculated in the pier as per the allowable permissible stresses in the steel and concrete as per working stress method. For a sample calculation of pier CBIP manual of design of barrages on permeable foundation may be referred.

RCC Raft

Following are the two types of floors:

1. Gravity type where the uplift pressure is balanced by the self weight of the floor only considering unit length of the floor.
2. Reinforced cement concrete raft type where the uplift pressure is balanced by the weight of the floor, piers and other super-imposed dead loads considering a span as single unit.

The design of Reinforced cement concrete raft may generally be done as per the theory of beams on elastic foundation. The design will depend on the value of modulus of subgrade reaction (K), span length, total length of raft etc. However, for small spans upto 6 m, the floor shall be designed as a continuous beam resting on a homogenous foundation. The abutment, if necessary, may be made independent by providing a joint in the raft with suitable water seals. The raft shall be designed for the moments caused by worst combination of the following forces:

a) Uplift
b) Soil reaction
c) Moments transferred from the abutments and piers
d) Seismic forces, if any

For the purpose of analysis, the entire width of the raft may be divided into different sections, depending on the loads and moments anticipated to act over the different sections, such as upstream section (including glacis), downstream glacis and downstream cistern sections.

The raft of a Barrage is designed treating it as abeam on elastic foundation as per theory postulatedby M. Hetenyi in his book 'Beams on Elastic Foundations' (published by the 'University of Michigan Press'of U.S.A). CBIP manual may be referred for a brief introduction of this theory.

The unit load/moment acting at a particular location (at abutment, pier, etc.) generates moments at various points in the raft is given by the following equations:

i. $M = (-\frac{1}{\lambda})B_{\lambda x}$ ..Unit load acting on the extreme end Abutment

ii. $M = \frac{1}{4\lambda}\left[\alpha C_{\lambda x} - 2\beta D_{\lambda x} + C_{\lambda|a-x|}\right]$ ..Unit load acting on an intermediate Pier

iii. $M = M_a A_{\lambda x}$ .unit .Moment acting on the extreme end Abutment

Where,

$$\lambda = \sqrt{\frac{K}{4EI}}$$

$$\alpha = C_{\lambda a} + 2D_{\lambda a}$$

$$\beta = C_{\lambda a} + D_{\lambda a}$$

$$A_{\lambda x} = e^{-\lambda x}(\cos \lambda x + \sin \lambda x)$$

$$B_{\lambda x} = e^{-\lambda x} \sin \lambda x$$

$$C_{\lambda x} = e^{-\lambda x}(\cos \lambda x - \sin \lambda x)$$

$$D_{\lambda x} = e^{-\lambda x} \cos \lambda x$$

After calculating the moments generated in the raft due to various individual loads/moments, algebraic summation of all the moments is done and moments due to differential head & uplift forces are also accounted for.

Cut -Off

They are the barriers provided below the floor of the structure and extend from abutment to abutment both in the u/s and d/s. Cut off is provided all along the pucca floor and also below the wing wall return wall and flank wall. Cut – off may be provided either as a sheet pile or as an RCC Cut-off. Generally sheet pile is preferred because of ease in construction but in bouldary, gravelly and rocky strata the sheet pile cannot be driven and in that case an RCC Cut off also called a diaphragm is provided.

The depth of cut-off is governed by the scour and exit gradient considerations A factor of safety shall be applied to the scour depth. While calculating the safe exit gradient, care shall be taken to take the downstream retrogressed bed level.

In the design of cutoff, the foundation strata on one side of the cut off is assumed to be scoured or eroded and on the other side the active earth pressure is acting. Moment is calculated and thickness and reinforcement is provided. In sheet pile a minimum grip length below the scoured level is provided and the earth pressure is assumed to vary from active earth pressure to passive earth pressure in this region. Based on the section modulus the section of ztype sheet pile (most commonly used) is provided.

Abutment, flank wall and flared out wall

Generally the abutment is constructed monolithically with the raft. In that case the moment due to earth pressure behind the abutment are transferred to the raft. Flank wall is designed as a cantilever type of retaining wall. Two loading conditions are considered i.e. pond level and construction stage. In the construction stage laoding condition, the backfill is considered as dry and no or minimum uplift force is considered as acting on the wall footing. In the pond level design loading condition, the backfill is saturated and full uplift is taken in the design.Conventional method of analysis is used in analyzing the structure and calculating the bearing pressure at heel and toe. Reinforcement in stem, heel and toe is provided as per the moments and are also checked for shear failure which is critical.Care shall be taken such that the safe bearing capacity of the foundation soil is not exceeded below the footing. Flared out wall is designed as a gravity wall with its river side face gradually merging into the guide bund slope.

Chapter - 3

Stability Analysis

3.0 Introduction

Stability analysis is an important part of planning and design of any diversion structure (Barrage or Weir). Before the individual components of diversion structure are designed structurally for various design conditions, its stability as a whole is checked.

Stability in general is checked for

1. Sliding
2. Overturning
3. Floatation

The design conditions and forces are dependent upon the configuration of the diversion structure i.e. whether it is:

1. Independent pier with gravity floor (resisting forces mainly through gravity)
2. Raft (resisting forces through slab /beam action)
3. Others (combined action)

For independent pier with gravity floor type structure, each pier and abutment is structurally separate from the floor through PVC seals. The loads from Pier do not get transferred to floor. Therefore stability of every Pier / Abutment is checked independently. The floor resists the uplift force, its own weight along with water sediment etc placed directly over it. Stability analysis for such structures is very important. Sizes and thickness of various components are determined to resist the loading.

For Raft type structure, the piers and / or abutments are monolithic with floor and floor also participates in resisting the forces by slab / beam action. Raft structures are more stable against sliding or overturning. However floatation condition is checked for raft structures.

Sometimes, due to design considerations, the raft and gravity both type of components are provided. Under such case, the stability of all individual component is checked.

3.1 Load Combinations

Cl 41.1 of IS 6512 “Criteria for design of Soid Gravity Dam” provides for some Load combinations which are as follows:

4. LOAD COMBINATIONS

4.1 Criteria — Gravity dam design should be based on the most adverse load combination A, B, C, D, E, F or G given below using the safety factors prescribed. Depending on the scope and details of the various project components, site conditions and construction programme one or more of the following loading combinations may not be applicable *ipso-facto* and may need suitable modifications:

a) *Load Combination A (Construction Condition)* — Dam completed but no water in reservoir and no tailwater.

b) *Load Combination B (Normal Operating Condition)* — Full reservoir elevation normal dry weather tailwater, normal uplift; ice and silt (if applicable).

c) *Load Combination C (Flood Discharge Condition)* — Reservoir at maximum flood pool elevation, all gates open, tailwater at flood elevation, normal uplift, and silt (if applicable).

d) *Load Combination D* — Combination A, with earthquake.

e) *Load Combination E* — Combination B, with earthquake but no ice.

f) *Load Combination F* — Combination C, but with extreme uplift (drains inoperative).

g) *Load Combination G* — Combination E, but with extreme uplift (drains inoperative).

Such combinations may be considered in case of Weirs and for independent Piers / Abutments for Barrages as well. However, some of these conditions are applicable to dam only. In general, following loading combinations (Design conditions) are checked for diversion structures (Weirs and Barrages):

A. Construction Stage:

During construction there are many conditions when stability of the structure may be vulnerable e.g. when construction is only partial, useful downward loading (of Bridge, Hydro Mech Equip. etc) may not be placed etc. Stability under such conditions should be checked as per construction planning of the project and it may vary from case to case.

B. Normal Operation Stage:

During normal operation, the gates of barrage are partially or fully closed. The lateral thrust of water is transferred from Gates to Pier and from Piers to footing / Raft. The sliding / overturning of the structure is resisted by the weight of the structure.

The abutments are subjected to horizontal soil pressure (active or at rest). The stability against sliding / overturning is resisted by weight / slab/beam action depending upon whether the abutment is monolithic with floor or not.

C. High Flood Condition:

Stability of the structure is also checked during design flood condition (usually 1 in 100 year return period flood for important / permanent barrages). However as per cl 5.4.5.2 of IS 11130 “Structural dEsign of Barrages and Weirs” :

5.4.5.2 For checking the stability, two emergency conditions (like high flood level and seismic) shall not be combined together. Similarly wind forces may not be considered while taking seismic forces.

Under all Design condition (i.e. Loading combinations) the structure is subjected to may forces which have to be estimated for stability of the structure. As per IS 11130 "Criteria for structural Design of Barrages and Weirs" following forces are considered for checking stability

5.4.5 *Design*

5.4.5.1 *General* — For design of the pier, the worst combination of the following forces and moments shall be considered:

a) Dead loads;
b) Live loads due to road/railway bridges as in relevant Indian Roads Congress Codes;
c) Impact due to live loads;
d) Longitudinal forces caused by tractive efforts of vehicles or by braking of vehicles and/or those caused by restraint of movement of free bearings by friction or deformation;
e) Temperature forces transmitted through bridge bearings;
f) Dead and live loads of gates, stoplogs counter weights and the hoist bridge;
g) Braking effect of gantry crane;
h) Buoyancy;
j) Wind forces;
k) Water current forces;
m) Differential hydrostatic pressure with one side gate open and the other adjacent gate closed;
n) Seismic forces and movements, if any; and
p) Hydrodynamic forces due to seismic conditions, if any.

And in case of Abutments

5.6.3.1 For design of the abutment blocks, worst combination of the following forces and moments, pertaining to the block under consideration, shall be taken into account:

a) Dead load;
b) Live load due to moving traffic over the bridges;
c) Impact due to live loads;
d) Longitudinal forces caused by tractive efforts of vehicles or by braking of vehicles and/or these caused by restraint of movement of free bearings by friction or deformation;
e) Dead and live loads of gate and gate bridge;
f) Braking effect of gantry crane;
g) Earth pressure, live load surcharge and saturation pressure;

h) Uplift;
j) Wind forces;
k) Water current forces;
m) Seismic forces and moments, if any; and
n) Hydrodynamic forces due to seismic conditions, if any.

Apart from above, the other forces shall be as specified in relevant Indian Roads Congress Bridge Codes available.

However the above list is not exhaustive and in many cases, some other forces which are not mentioned above have to be considered as per site conditions.

A typical enumeration of design forces is as follows:

S1	Self Weight i.e. Concrete weight of pier and footing
S1.e	Seismic load due to inertia force for A1
B1	Bridge Self load i.e. Concrete weight of Bridge Deck, girder etc.
B1.e	Seismic load due to inertia force of B2
B2.1	Live Load class A or AA whichever applicable
B2.1.e	Seismic load due to live load of Class A or AA
B3.1a	Gantry Live Load Normal (Travelling with Stoplog)
B3.1b	Gantry Live Load Normal (Stationary)
B3.1.e	Seismic load due to Gantry (Dead Load)
B3.2	Gantry Live Load BDT condition
B4	Impact load due to live load
B5	Friction due to braking of vehicles and bearing restraint
B6	Temperature forces through Bridge Bearing
	(Seismic load for BDT load of gantry is not considered)
	One condition when only one Bridge span is in place has also been considered
G1	Dead Load of Gates, Tretle, hoist etc
G1.e	Seismic load due to Intertia loads of Gates, tretle etc.
G2	BDT condition for operation of Gates
	(Seismic load for BDT load of gantry is not considered)
	One condition when only one half testle is in place has also been considered
H1	Static Pressure of Water agaist gates at Pond Level (All gates closed)

H2	Silt Load (All gates closed)
H3	Hydrodynamic force due to water standing against gate
H4	One gate closed position (Jump formation on one side of Pier + Obique current force)
H5	One Gate repair (Stoplog in position on one side of Pier)
H6	Uplift for Pond Level / Buoyancy force
H7	Uplift for HFL (Buoyancy force) - Worst Uplift Case
H8	During HFL differential water pressure due to various reasons

After the forces (and moments) causing sliding or overturning as well as restoring forces are estimated, they are compared to provide adequate factor of safety. As per IS 11130 "Structural Design of Barrages and Weirs:

5.6.3.5 The factor of safety against overturning shall not be less than 2·00 under normal conditions of loading and not less than 1·5 under seismic conditions of loading.

5.6.3.6 The factor of safety against sliding shall not be less than 1·75 under normal conditions of loading and not less than 1·5 under seismic conditions of loading.

NOTE — For calculation of the loads and moments due to earthquake, reference may be made to IS : 1893-1975*. For calculation of the overturning and sliding factors, reference may be made to IS : 1904-1977†.

FoS against Sliding = Resisting Force/ Sliding Force = $\Sigma F_R / \Sigma F_S$
FoS against overturning = Moment of resistance/ Overturning moment = $\Sigma M_R / \Sigma M_O$

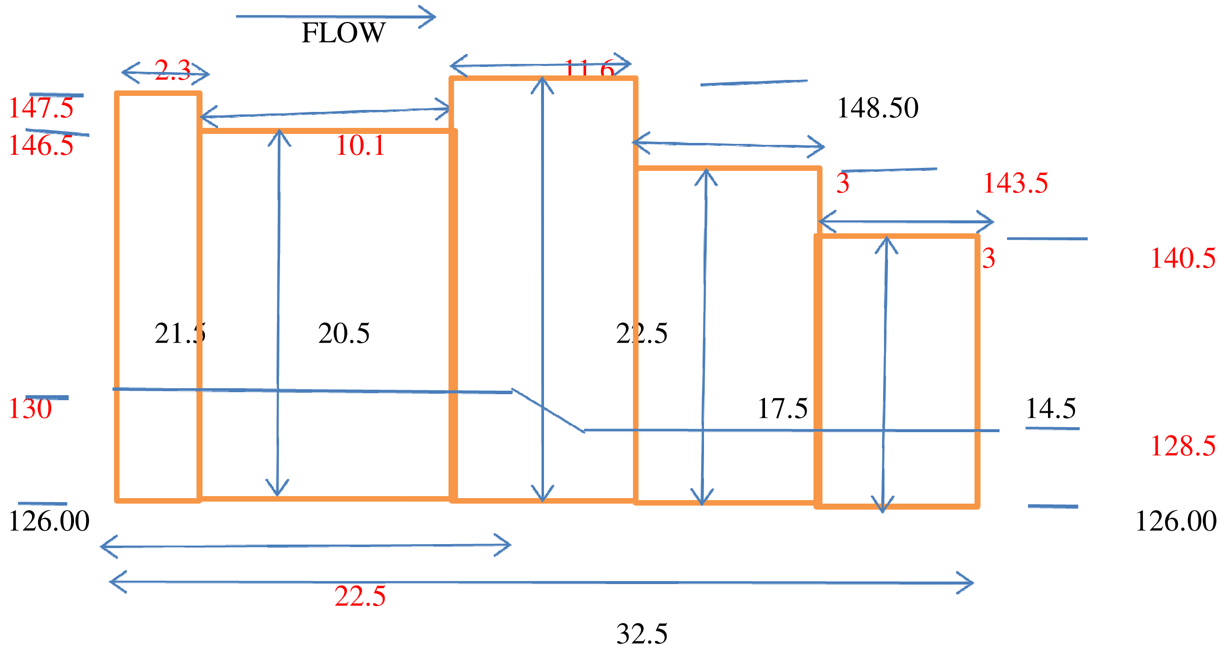
For computing the forces resisting sliding, dead load normal to sliding surface is multiplied with coeff of friction. As per IS 11130

5.6.3.7 The value of the co-efficient of friction shall be determined by field investigations. However, in cases where this is not possible, the values given below can be adopted for designs:

Material	*Co-efficient of Friction*
Soft clay, silt	0·25
Medium of stiff clay	0·30
Silty sand, sand and gravel with high clay content	0·35
Coarse sandy soil containing silt	0·45
Coarse sandy soil containing no silt or clay	0·55

A typical design calculation is provided for illustration:

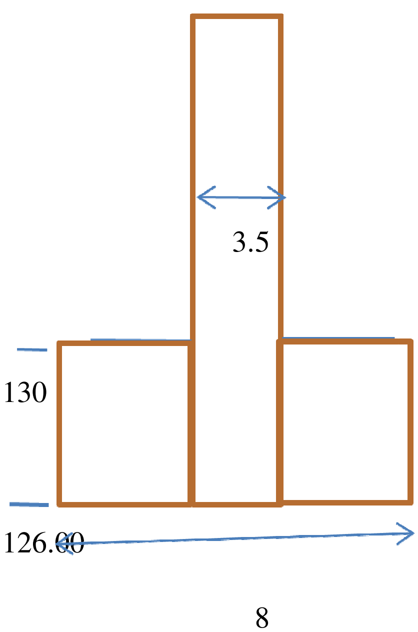
Self Weight of Pier + Footing   for Pier footing at EL 126.0 m



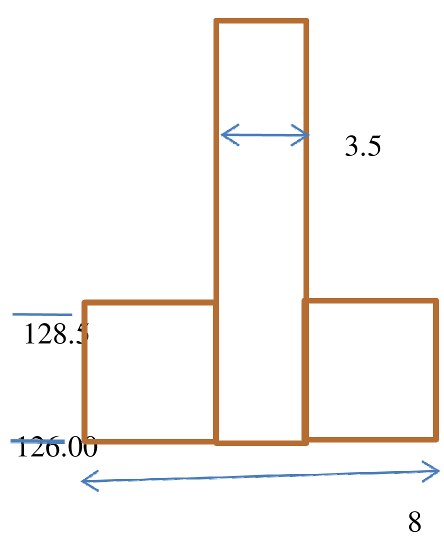
Sectional Elevation of Typical Pier

Area 1
U/S Section

Area 2
D/S Section



Upstream Section



Downstream Section

All Dimensions are in metres

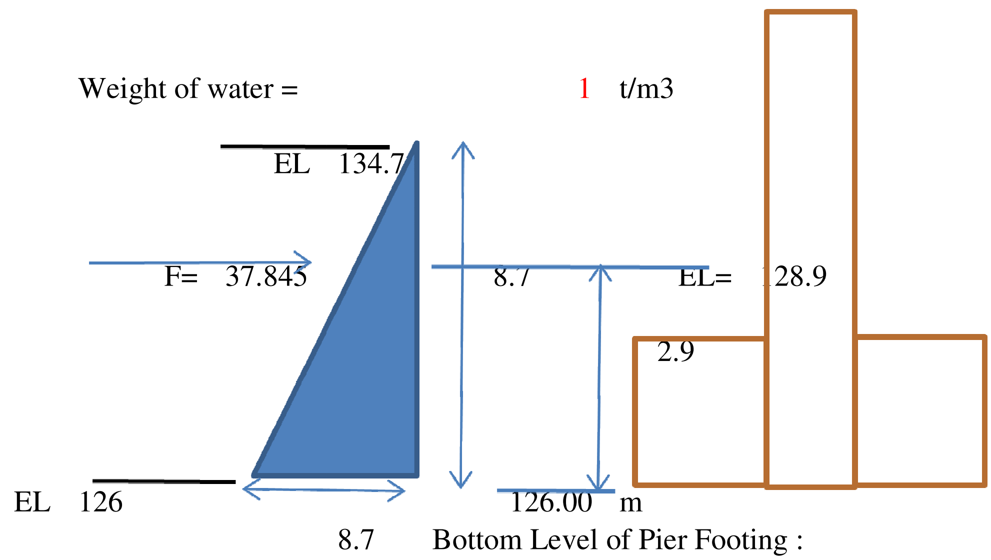
Weight with Concrete unit weight = 1.5 t/m3

	Area (m2)		Thk /Length (m)		Volume (m3)	Weight (t)
Pier =	613.5	x	3.5	=	2147.25	3220.875
Footing + floor Area1 =	18	x	22.5	=	405	607.5
Footing + floor Area2 =	11.25	x	10	=	112.5	168.75
					Total =	3997.125

As per cl 5.6.3.7 coeff of friction for soft clay or silt = 0.25

Restraining force 999.2813

Weight of water = 1 t/m3



The Hydrostatic force shall act over the gate and shall be transferred to Pier through wheels of the gate.

C/C Distance of Pier = 15.5 m

Total sliding force acting on pier at pond level = 586.5975 t

		V	H	LeverArm	M	V	H	LeverArm	M	V	H	LeverArm	M
1	RCC Self Weight (Dry) Pier + Footing	9446.25				9587.94	283.39	13.50	3825.73	9304.56	283.39	13.50	3825.73
2a	Uplift for same WL in adjacent bay (PondLevel)	2319.00				2319.00				2319.00			
2b	Uplift for same WL in adjacent bay (HFL)	3768.38				3768.38				3768.38			
3	Uplift for adjacent bay dry (stoplog condition)	1204.87				1204.87				1204.87			
4	Bridge RCC dead weight	329.70				334.64	9.89	27.00	267.05	324.75	9.89	27.00	267.05
5	Bridge Live Load traffic	67.00	13.40	27.00	361.77	67.00	13.40	27.00	361.77	67.00	13.40	27.00	361.77
6	Gantry Live Load	106.40	21.28	27.00	574.56	106.40	21.28	27.00	574.56	106.40	21.28	27.00	574.56
7	Trestle with service Gate	130.00				131.95	3.90	27.00	105.30	128.05	3.90	27.00	105.30
8	Friction on Bridge Dead Load		16.48	27.00	445.09		16.48	27.00	445.09		16.48	27.00	445.09
9a	Current force (HFL along flow)		675.66	13.50	9121.37		675.66	13.50	9121.37		675.66	13.50	9121.37
9b	Current force (HFL across flow)		245.92	6.75	1659.95								
10	Hydrstatic force on gate at pond level		1454.60		19006.74		1501.17		19269.51		1501.17		19269.51
11	One Bay with stoplog + adjacent service gate		210.30		29939.01		249.66		30161.09		249.66		29939.01
12	Head difference at flood level only NEQ		438.75		6823.92								

Besides checking factor of safety against sliding and overturning, the stability is also checked for following considerations:

1. The whole structure / its independent component when subjected to extreme uplift may be subjected to floatation. The self weight of structure should be
2. For gravity floor type structure, the thickness of floor should be sufficient to resist the uplift at all location for various design conditions i.e hydraulic jump formation, pond level formation, high flood condition etc.
3. The Pier thickness is also checked for adequate slenderness ratio as per design.

Chapter - 4

Structural Design of Barrage

4.0 Introduction

Various components of the diversion structures have to be very carefully designed for ensuring safety and economy. Due to different types of foundation materials encountered, such as clay, etc., some treatments may become necessary. These will have to be carefully evolved and their effects taken into consideration while designing the structure. The type of cut off proposed such as sheet piles or R.C.C. cut offs will have to be suitably selected depending on the construction difficulty and these will have to be carefully designed to ensure safety of the structure. In the ensuing paragraph, guidelines for the design of some of the important components of the main structure such as cut offs, pucca floor, pier, divide wall, abutment, flank wall and return wall are elaborated.

4.1 Data Required

In addition to the data listed in chapter 3 and analyzed values thereof, the following data are also required to be collected out for carrying out the structural design. There are some repetitions. However, these have been included for convenience.

(a) High flood level and Minimum water level in the river.

(b) Design Pond Level

(c) The lowest and highest tide levels in case of tidal streams.

(d) Intensity of silt charge in the river during high and low flood stages.

(e) Log of boreholes to a depth of about 15 to 25 m below the deepest bed level at a spacing of one per bay in at least three rows. One row of the bore holes may be along barrage / weir axis, the second at a distance of about 15 m upstream of the axis and third at a distance of about 30 – 40 m downstream of the axis. The location of the boreholes along the axis will be staggered with reference to those along the upstream and downstream lines. However, these are essential whenever presence of clay stratum is detected in the foundation. The extent, depth and location of clay layer should be carefully assessed.

(f) In sandy strata, standard penetration test results for a depth of at least 8 to 12 m at a spacing of 40 -50 m along the transverse direction (along river width) and at a spacing of 30 m in the longitudinal direction (along the flow).

(g) Whenever clayey strata are encountered, undisturbed samples of the clay layers from the proposed foundation level upto a depth of 8 m below the foundation level for each bay. The analyzed values of shear parameters, void ratio, consolidation characteristics, moisture content, in situ density, sensitivity and permeability.

(h) For sandy soil, grain size distribution curves from undisturbed samples obtained at 3 meter intervals from each bore hole. Dry densities, relative densities and angle of internal friction.

(i) Modulus of sub grade reaction at the proposed foundation level. If the structure is long or if there is wide variation in the properties of the foundation material, the values of modulus of sub grade reaction in each unit / block separated by double pier. If it is not feasible to determine the modulus of sub grade reaction at site, the values given in Appendix 7.3 may be adopted for design purposes, depending on the type of soil met with.

(j) Results of feasibility studies for driving sheet piles or otherwise if the river bed consists of boulders or is made up of stiff clay.

4.2 DESIGN CRITERIA

The design criteria is generally considered as guidelines and may be modified to suit the field conditions with due consideration for safety and economy. R.C.C. work shall be done in accordance with IS : 456-1978.

Cut-Offs:

The upstream and downstream cut-offs of the diversion structure may be of steel sheet piles anchored to the barrage/weir by means of R.C.C. caps, or of masonry or reinforced cement concrete of R.C.C. diaphragm wall. The sheet pile cut-offs shall be designed as sheet pile retaining walls anchored at top. They shall be designed to resist the worst combination of forces and moments considering the possible scour on the outer side, earth pressure and surcharge due to floor loads on the inner side, differential hydrostatic pressure computed on the basis of the percentage of pressure of seepage flow below the floor etc. The upstream and downstream masonry of R.C.C. cut-offs shall be designed as cantilever walls casted monolithically with the floor of the barrage/weir.

4.2.1 Impervious Floor

Following are the two types of floors:

a) Gravity type where the uplift pressure is balanced by the self weight of the floor only considering unit length of the floor and

b) Reinforced cement concrete raft type where the uplift pressure is balanced by the weight of the floor, piers and other super-imposed dead loads considering a span as single unit. While the gravity type can either be of plain concrete or masonry, the raft type would be reinforced concrete only.

4.2.2 Thickness of Floor:

Thickness of the impervious floor shall be adequate to counter balance the uplift pressure at the point under consideration for gravity type and reinforced cement concrete type of floor.

4.2.3 Uplift Pressure:

The uplift pressure at any point shall be calculated by any accepted practice taking into account the effect of the upstream and downstream cut-offs, intermediate cut-offs (if any), interference of cut-offs, thickness of floor and slope of the glacis.

4.24 Hydraulic jump:

The thickness of downstream floors shall also be checked under the hydraulic jump conditions.

4.2.5 Gravity Floor:

The thickness of the floor adopted for construction shall be at least 10 percent tmore than the thickness required to counteract the uplift pressure at that point under the worst possible combination of loads in different including seismic conditions

Abutments and piers may either be independent of the floor separated from it by means of water tight seals or may be monolithic with the floor. The latter case may be adopted only if the thickness of the floor is equal to or more than half the length of one span.

Where the floor is of plain concrete, suitable temperature reinforcement shall be provided.

4.2.6 Reinforced Cement Concrete Raft:

Spans up to 6 m:

The design of the raft may generally be done as per the theory of beams on elastic foundation. The design will depend on the value of modulus of subgrade reaction (K), span length, total length of raft etc. However, for small spans upto 6 m, the floor shall be designed as a continuous beam resting on a homogeneous foundation. The abutment, if necessary, may be made independent by providing a joint in the raft with suitable water seals. The raft shall be designed for the moments caused by the worst combination of the following forces:

a) Uplift;

b) Soil reaction;

c) Moments transferred from the abutment and piers;

d) Seismic forces, if any.

For this purpose, the loads transmitted by the abutments and piers may be assumed to be distributed uniformly on the foundation.

Spans above 6 m :

The floor shall be designed as a finite beam resting on elastic foundation and subjected to concentrated loads and moments at the pier and abutment points. Taking duly into account the effect of the width of the raft actually provided, the value of the modulus of subgrade reaction (Ks) shall be determined as prescribed in IS : 2950 (Part I) – 1973* and IS : 9214 – 1979.

For the purpose of analysis, the entire width of the raft may be divided into different sections, depending on the loads and moments anticipated to act over the different sections, such as upstream section (including glacis), downstream glacis and downstream cistern sections.

Piers:

In barrages and weirs constructed as a reinforced cement concrete structure, the piers are constructed monolithic with the floor of the diversion structure. However, the piers in the gravity type of floor are generally constructed independent of the floor. Proper joint and sealing arrangements between the gravity floor and the pier all around shall be provided.

Thickness of Pier:

The thickness of the pier shall be fixed from consideration of (i) forces and moments transferred by the pier to the floor/foundation (ii) minimum thickness required at the block outs for the main gate and stoplog grooves and (iii) the mass of the pier required for counter-acting the uplift pressure.

Length of pier:

In the case of a raft type floor of the diversion structure, the piers shall generally be extended up to the full width of the raft to avoid cantilever action of the raft at the ends. In the case of gravity type floor, the length of pier may however, be restricted according to the minimum requirement, from considerations of road rail bridges, hoist bridge, space required for housing instruments, if any, main gate grooves, stoplog grooves, space for storage of stoplogs, adequate length to prevent cross flows occurring which may cause damages to the floor and beyond.

Height of Pier:

On the upstream side, the pier shall generally be constructed above the pond level / affluxed HFL with adequate free board. The height shall also be fixed as per requirement of the mass of the pier in counteracting uplift pressure. The height of the pier shall also be such that under fully raised position above the affluxed HFL/pond level, about one metre of the gate still remains within the gate groove.

On the downstream side, the piers shall generally be constructed at least one metre above the high flood level, upto 1 to 2 m as found necessary beyond the end of the bridges and instrumentation platform, if any and thereafter the height could be reduced according to low flood levels on the downstream side. In the portions where road/rail bridges are provided, the height of the piers shall be fixed such that the bearings of the bridges are not hit by floating debris during high floods.

In the main gate portion, the height of the pier shall be fixed such that during high flood, the bottom of the gate is at least one metre clear of the affluxed high flood level. In earthquake regions, however, the top level of the pier could be restricted to the top level of the abutments and steel trestles provided over the piers for housing the hoist bridges for operation of the gates and stoplogs. This arrangement would reduce the loads and moments due to inertia during earthquake.

4.3 Design:

4.3.1 General:

For design of the pier, the worst combination of the following forces and moments shall be considered.

a) Dead loads;

b) Live loads due to road/railway bridges as in relevant Indian Roads Congress Codes;

c) Impact due to live loads;

d) Longitudinal forces caused by tractive efforts of vehicles or by braking of vehicles and / or those caused by restraint of movement of free bearings by friction or deformation;

e) Temperature forces transmitted through bridges bearings;

f) Dead and live loads of gates, stoplogs counter weights and the hoist bridges;

g) Braking effect of gantry crane;

h) Buoyancy;

i) Differential hydrostatic pressure with one side gate open and the other adjacent gate closed;

j) Seismic forces and movements, if any ; and

k) Hydrodynamic forces due to seismic conditions, if any.

Apart from above, the other forces shall be as specified in relevant Indian Roads Congress Bridge Codes available.

For checking the stability, two emergency conditions (like high flood level and seismic) shall not be combined together.

The reinforcement of the pier could be curtailed at convenient levels. The reinforcement shall be adequately taken into the raft/foundation slab as the case may be and anchored properly.

4.3.2 Blockout Zones:

For design of blockouts, the following points shall be kept in view:

a) The width of the pier between the blockouts on either face shall not be less than 60 cm.

b) The increase in the sectional area of the pier in the blockout zone due to the second stage concrete shall be ignored while calculating the reinforcement.

c) Blockout zones shall be designed to withstand the concentration of stresses due to the worst combination of forces and moments and differential hydrostatic pressure with the gate closed on one side and stoplogs dropped on the other adjacent side,

d) Adequate dowel bars and secondary reinforcement shall be provided in the second stage concrete to effectively bond it to the main pier concrete.

4.3.3 Double Piers:

Depending on the variation in the foundation characteristics, construction programme and design considerations of the raft, the water way of the barrage/weir is splitup into different units. Each unit comprises a number of bays, ranging usually from 5 to 10. Between each unit, double pier shall be provided with proper joint and sealing arrangement. Each unit shall be thoroughly boxed at the foundation by the upstream and downstream sheet piles/cut offs and cross sheet piles/cut offs below the double piers. The depth of cross sheet piles/cut offs shall be suitably varied from the upstream sheet pile/cut off level to the downstream sheet pile/cut off level.

4.3.4 Design:

Each unit of the double pier shall be structurally independent of the other without any transfer of load and moments from one to the other. The design criteria specified for single pier shall hold good for the design of double pier also. For this purpose, each unit of the double pier shall be treated as acting completely independent of the other pier.

Pier Cap:

Thickness:

The thickness of the pier cap shall not be less than 300 mm for spans up to 25 m.

Reinforcement:

The reinforcement for the pier cap should be distributed both at top and bottom in the longitudinal and transverse directions. In addition to this, two layers of mesh reinforcement of 6 mm diameter spaced at 75 mm centre to centre shall be placed under the bearings of the road/rail bridge beams.

4.3.5 Position and Length of Divide Walls:

Final layout and exact length of the divide walls shall be determined on the basis of hydraulic model studies to ensure adequacy of tall water depth in the under sluice and river sluice bays for the formation of hydraulic jump and to avoid cross flow in the close vicinity of the structure.

Foundation:

The foundation of the divide walls shall be extended below the bed upto a depth of 2.25 R below the high flood level at the nose portion with adequate grip lengths, where R is the calculated Lacey's depth of scour below high flood level. The depth of foundation may be decreased progressively towards the pucca floor of the barrage / weir up to a minimum depth of main cut-offs provided.

The foundation of the divide walls shall be made of steel sheet piles in portions, wherever it is possible to drive the sheet piles of sufficient depths and in other portions, it shall be made of walls. For design of well foundation, reference may be made to IS : 3955-1967

The upstream and downstream divide walls shall be separated out from the main pier beyond the pucca floor of the barrage/weir by joints. Joints shall also be provided in the divide walls at places where there are changes from sheet pile/cut-off foundation to well foundation.

Height:

The top levels of the divide walls shall be fixed above the high flood levels with sufficient free board.

Design:

The upstream divide wall shall be designed to resist the differential head on account of the velocity of flow in the pocket due to the gates adjacent to the divide wall on either side being kept in open and closed condition. The downstream divide wall shall be designed to resist the moments due to the differential head caused by the closure of gate on one side and opening of gate on the other side. The differential heads to be considered in the design are indicated by hydraulic model tests wherever conducted. However, for preliminary designs, a differential head of about 2m may be assumed.

The divide walls shall be checked for their overall stability. Forces due to self weight, water (including ice wherever applicable), uplift, etc, shall be considered along with earthquake forces acting on the walls. While checking the stability, two emergency conditions (like HFL and seismic) shall not be combined together.

Abutments:

Abutments are generally designed as retaining walls separated from the main floor by an expansion joint, settlement joint with seals. In some cases, if the total waterway between the abutments does not exceed 40 to 50 m, the abutments are constructed monolithic with the main floor and the whole section is designed as a trough. For analysis and design, along the flow direction, the abutments are generally divided into three to four blocks, namely, upstream block, gate bridge block, road/rail bridge block and downstream block, depending on the loadings. The blocks are usually separated by joints with seals and designed properly to resist the loads and moments acting on the blocks.

Top Width:
Top width of the abutment in each block shall be fixed as per the requirements due to the loads and moments, minimum width required for block-outs of main gate and stoplog grooves, bridge bearings, gate trestle foundation, etc. Minimum width of abutment clear of the block-outs and behind bearing niches shall be 60 cm. In the gate and road/rail bridge block, it is generally kept as 125 to 140 cm.

4.3.6 Design:

For design of the abutment blocks, worst combination of the following forces and moments, pertaining to the block under consideration, shall be taken into account:

a) Dead load;

b) Live load due to moving traffic over the bridges;

c) Impact due to live loads;

d) Longitudinal forces caused by tractive efforts of vehicles or by braking of vehicles and / or these caused by restraint of movement of free bearings by friction or deformation;

e) Dead and live loads of gate and gate bridge;

f) Braking effect of gantry crane;

g) Earth pressure, live load surcharge and saturation pressure;

h) Uplift;

i) Seismic forces and moments, if any ; and

j) Hydrodynamic forces due to seismic conditions, if any.

Apart from above, the other forces shall be as specified in relevant Indian Roads Congress Bridge Codes available.

The abutment section shall be checked for safety against allowable bearing pressure, overturning and sliding.

The factor of safety against overturning shall not be less than 2:0 under normal conditions of loading and not less than 1:5 under seismic conditions of loading.

The factor of safety against sliding shall not be less than 1.75 under normal condition and not less than 1:5 under seismic conditions of loading.

Flank Wall:

In continuation of abutments of the weir/barrage head regulators, flank walls are provided both on the upstream and downstream sides on both the banks. However, if site conditions permit, one or more of these flank walls may be omitted and the banks may be trimmed and adequately pitched with stones. The flank walls ensure smooth entry and exit of waters into and away from the barrage/weir. On the upstream side, the flank wall may be provided a transition from the slope of the guide bund to vertical face at the abutment and over a length of 2 to 2.5 L1, where L1 is the sloping length of the water face of the u/s guide bund. On the downstream side, this transition may be provided over a length of 2.5 to 3.5 L2 where L2 is the sloping length of the water face of the d/s guide bund. The total transition length is made up of two types of construction, one with solid concrete / masonry wall usually called flank wall and the other with concrete /masonry and cement concrete blocks, usually called the flared out wall. The water face of the flank wall (solid wall portion) is generally changed from vertical at the abutment end to a slope of 0.5 (horizontal) : 1 (vertical) and the stem is constructed of concrete / masonry throughout its height. The water face of the flared out wall is generally changed from the slope of the end section of flank wall (generally 0.5 : 1) to the slope of the guide bunds which is generally 2:1 to 3:1 The stem of the flared out wall is constructed of concrete / masonry for certain height, overlaid by interlocking cement concrete blocks one above the other. The number of interlocking cement concrete blocks varies as the slope of the flared out wall varies.

Top Width:

The top width of the flank wall shall not be less than 600 mm.

The top width of the flared out wall would be varying depending on the size of the cement concrete block. Generally a size of 1500 mm is adopted.

Design:

For the design of the flank wall and flared out wall as an earth retaining structure, worst combination of the following forces and moments shall be taken into account;

a) Dead loads;
b) Earth pressure, live load surcharges and saturation pressure;
c) Uplift;
d) Seismic forces and moments, if any, and
e) Hydrodynamic forces due to seismic condition if any. For checking the stability, two emergency conditions (lie HFL and seismic) shall not be combined together.

The flank wall and flared out wall sections shall be checked for safety against allowable bearing pressure, overturning and sliding as in the case of abutment. For calculation of earth pressures, any standard practice may be adopted.

Return Wall:

Return walls are generally provided at right angles to the abutment either at its ends or at the flank wall portion. Wherever return walls are constructed as part of the flank wall, it is desirable to separate the stem of the return wall from the stem of the flank wall by a joint. The base slab for the flank wall and return wall would be the same and shall be designed carefully to resist the loads and moments acting on the same.

Length:

Where the return-walls are provided at the ends of the abutment, they shall be extended atleast up to the end of the heel portion of the base slab of upstream or downstream abutment blocks as the case may be. In the case of the head regulator, the return-walls shall be keyed properly into the embankment of the canal for a length of at least 2 m beyond the top edge. Where the return wall is part of the flank wall, the same shall be extended up to the end of the heel portion of the base slab.

4.3.7 Design:

For design of the return wall, worst combination of the following forces and moments shall be taken into account.

a) Dead loads;
b) Earth pressure, live load surcharge and saturation pressures;
c) Uplift;
d) Seismic forces and moments, if any ; and
e) Hydrodynamic forces.

For checking the stability, two emergency conditions (like HFL and seismic) shall not be combined together.

Silt Excluder:

Silt excluder tunnels generally provided in the form of rectangular R.C.C. barrels. The number of under sluice bays and the number of silt excluder tunnels in each bay to be provided, their location, layout and dimensions shall be fixed on the basis of model tests in order to obtain optimum silt exclusion from the waters emerging into the head regulator.

4.3.8 Design:

For the design of silt excluders, the following loads shall be considered:

a) Load due to water of the pond, and

b) Load due to silt deposited.

The silt excluder tunnels shall be designed for the worst combination of loadings and moments with one or more tunnels considered empty at a time and all vertical loads including the load of water inside the other tunnels.

Silt excluders shall be checked for floatation and sliding also with no silt load on top of the tunnels and no water inside all the tunnels.

Chapter- 5

Design of Silt Excluders & Silt Ejector

SEDIMENT EXCLUDER

The sediment excluders are provided on the barrages or diversion weirs in the river pocket adjacent to the head regulator to minimize sediment entry into the canal water. On rivers, the excluders have to deal with alluvial material such as boulder, gravel sand or silt depending upon the parent bed material, that is, being transported by the river. The structural arrangement to reduce the amount of sediment passing into a canal head regulator is made by constructing a divide wall upstream so as to form a pocket in front of the canal intake. The wall divides the stream flow as it approaches the barrages and weir so that part of the flow is diverted to the sluiceway and part through spillway bays. The main function of the divide wall is to form a sluicing pocket to produce a ponding area of low velocity in which the sediment will deposit rather than enter the canal head works. The sediment thus deposited is to be scoured out by turbulent flow through the sediment excluders pocket when the sluice gates are opened. Hydraulic model studies are undertaken to finalize various parameters such as divide wall length and orientation, gate regulation, etc. Different types of excluders have been tried on various head works. Generally the excluders cover only a few bays of the undersluice but at certain head works they may cover more number of bays. The heavy sediment laden bottom layers flow through excluder tunnels towards the river stream downstream of undersluice leaving relatively sediment free top layers of water to flow through head regulator.

Excessive sediment load can cause damage in a variety of ways which results in many serious problems such as :

(i) Meandering of streams.

(ii) Reduction of channel capacity.

(iii) Silting up of canal.

(iv) Damage to power units on hydel canals.

(v) Obstruction to navigation.

(v) Sediment bars of stream junctions.

(vi) Silting up of reservoirs.

(vii) Shoaling of harbours at river mouths.

(viii) Destroying the value of streams for pisciculture and recreational utility, etc.

Silt excluders are required to be provided in the river pockets of weir/barrages when river characteristics predominantly occur as given below:

a) When there is high ponding upstream of the barrage to meet the canal discharge requirements.

b) When the river/tributary is bringing sediment load of the order of 1500 ppm and above, and contains significant percentage of coarse and medium sediment.

c) If the river is in aggrading stage or wherever formation of bed bars/shoals is noticed due to unfavourable approach condition.

d) Bed building stage of the river may occur due to barrage obstruction to flow as well as improper regulation on the barrage gates.

e) Due to adverse flow curvature upstream of the barrage head regulator, most of the sediment load in high river stage may likely to settle in front of the head regulator and may enter its way in the canal.

f) Inspite of suitable location of head regulator, river training measures for arriving favorable curvature of flow, providing divide walls for separating pockets from barrage bays and suitable gate regulation of barrages/undersluice bays for sand exclusion, a large quantity of coarse material may find its way into the pocket. In such cases for efficient working of canal, silt excluders are required to be provided in the pocket.

g) Exclusion of gravels and boulders could be achieved by providing barrage crest at river bed level and ponding operation only during non flood season.

DATA REQUIRED:

The data required to carry out the design comprises:

a) Stage-discharge curves on the upstream and downstream of barrage site.
(b) Sediment size distribution curve and concentration in river.
(c) River width, silt concentration and grade of sediment that can be permitted into the canal and slope at the site.
(d) Canal discharge.
(e) Dimensions of the barrage/weir head regulator and undersluice.
(f) Charge and grade of sediment in the river water coming into undersluice.
(g) Charge and grade of sediment that can be permitted to go into the canal.

DESIGN CRITERIA FOR EXCLUDERS:

<u>Approach</u>

The curvature of the river flow approaching the canal head regulator plays an effective role in the efficient working of an excluder and as such the locations of the mouths of the tunnels have to be decided, keeping in view the approach conditions. All possible river approach conditions are to be examined carefully while deciding the layout of excluder tunnels. The tunnels are located in front of the canal regulator and their alignment is kept parallel to the axis of the regulator as far as possible. Any deviation in their alignment, found necessary towards their tail ends, should be made on a smooth curve so that kinks are avoided.

Location of Tunnels

In case the excluder tunnels are staggered, the mouths of the two successive tunnels are so located that the zone of suction of the two adjacent tunnels adequately overlapped to avoid deposition between the mouths of the two tunnels. Each tunnel should have an opening only at the front, facing the approach. There should be no side openings.

Tunnels

Though circular or square tunnels are hydraulically more efficient, the experiments carried out show that the rectangular tunnels with width less than height are generally more effective for sediment transport. Generally sediment excluder tunnels are rectangular in shape. The design of sediment excluders is tested in hydraulic model along with the barrage design for visualizing hydraulic performance.

The excluder should normally span 25% of undersluice bays and shall be divided into an equal number of compartments or tunnels by vanes gradually converging so as to accelerate the escaping flow for delivering it to the outfall channel on downstream side of the barrage. The dimensions of tunnels should be such that they run full bore for the designed discharge.

Spacing and Bellmouthing of tunnels

In excluders, tunnels at the inlet end should preferably be bell mouthed by decreasing head loss. Each tunnel has a certain zone of influence which extends upstream to a certain distance in straight direction as well as sideways up to some distance upstream. The two successive tunnels should be so placed that the zone of suction of the second starts before the zone of suction of the first tunnel ends. To create enough suction in tunnels and carry coarse material like gravel and boulders in the river, a minimum head of 0.9 to 1.2m is necessary for satisfactory working. Smaller head suffices for finer material. The extent of zone of influence and efficiency are determined by models.

Escape Discharge Capacity

An excluder should be designed for minimum escape discharge which would secure maximum efficiency as well as satisfactory exclusion of sediment. This is essential to avoid unnecessary turbulence and churning in the pocket which is caused by higher discharges in the pocket. Escape discharge may vary between 20 and 30% of the canal discharge.

The tunnel dimensions at the entry and the exit shall be so fixed as to ensure velocities that would carry the size of sediment to be removed. The section of the tunnel at the entry shall be so chosen that the velocity of flow at the intake is slightly higher than the velocity of bottom filaments of water upstream of the excluder. The section of tunnels up to their exit, where these end into the outfall channel shall be reduced gradually in such a way that there is an overall increase of 10 to 15% in velocity of emerging flow.

Self Cleansing Velocity at the Tunnel Entry

A velocity of 2 to 2.5 m/s is generally treated as self cleansing velocity inside the tunnels in alluvial reach and 3 to 4 m/s as self cleansing velocity inside tunnels in shingles and cobbles reach. For excluders, therefore, velocity more than self cleansing velocity may be assumed.

tunnel The velocity at the exit end of the may be worked out from the working head and throttling effected to attain velocity higher than 3.0 to 3.5 m/s at the exit in alluvial reach and 4 to 5 m/s in shingles and cobbles reach. If the width of the tunnel is kept the same, throttling is done by lowering the underside level of the roof of tunnels in the case of excluders.

The tunnels normally run as pressure conduits. However, there could be situations when flow with free surface may take place near the tail end of the tunnels. For such conditions, it should be ensured that there is no possibility of a hydraulic jump forming inside the tunnel.

Each tunnel must take the same discharge as other in spite of their unequal lengths. This is achieved by suitably adjusting the cross-section at intermediate points such that total loss of head of each tunnel is the same.

Roof Level of Tunnels

The roof of a sediment excluder should normally be located at the sill level of the canal.

Preferably the height of tunnels should be kept adequate to facilitate inspection and repair work. The tunnels shall be designed to run full fore to secure the maximum efficiency.

Control Structure

The discharge from sediment excluder is controlled by gated regulation at the downstream end of the tunnels. The quantum of discharge to be run through sediment excluder and frequency of its operation would vary in different parts of the year depending on the permissible sediment load in the canal and the sediment load entering in the pocket. This is achieved by operating regulating gate as required. However, in practice, the gates are either fully opened or fully closed.

FLUSHING

During the period when sediment excluder is not required to function, it is desirable to operate the regulation gates frequently, for short periods to flush the tunnels. It is apprehended that if the silt excluder tunnels are closed for considerable time it may lead to the choking of the tunnel to the extent that it may become difficult to flush out hydraulically. Otherwise, the tunnels are likely to get choked and may require manual clearance which maybe possible only during closure of the canal.

At times during the normal operation of the sediment excluder, the approach channel and/or tunnels may require flushing. This may be done by running the tunnels in rotation to achieve higher velocities.

STRUCTURAL DESIGN

For the structural design of sediment excluders, load due to water of the pond and load due to the silt deposited inside and outside of the tunnel should be considered.

The silt excluder has to be checked for floating and sliding also with no silt load on top of the tunnels.

A trash rack may be provided at the inlet of tunnel to avoid entry of trees and big branches in the tunnels. A provision of jetting water or compressed air in the tunnels may also be made for convenience of flushing of consolidated sediment deposit in the tunnels.

The silt excluders which are in vogue in India are classified into two types, viz., Khanki type and Kalabagh Type. These two types have derived their name from the diversion structure where they have been provided. In the Khanki type, the location of the up-stream mouths of the tunnel is staggered so that the length of the tunnels decreases in a series of steps, the longest tunnel being nearest to the head regulator. In the Kalabagh type the mouths of the tunnels at the upstream end are in one straight line. General layout of the two works, are shown below.

The efficiency of any type depends on a complex combination of factors like approach conditions in the river, quality and quantity of the suspended and bed sediments, discharge, etc. Khanki type has been more commonly used, probably on account of its greater flexibility.

SEDIMENT EJECTOR FOR IRRIGATION AND POWER CHANNELS

Sediment ejector, also known as sediment extractor or silt ejector, is a contrivance to remove excessive sediment load after it has entered a canal. The extraction of sediment is effected by causing sediment concentration in the bottom layers and separating them in such a way that there is least disturbance in the sediment distribution of the approaching flow. Sediment ejectors are usually provided in head reaches of canals which carry heavy silt, especially canals taking off from diversion weirs, anicut or barrages across rivers.

DATA REQUIRED:

The following data relating to the canal are needed for design of sediment ejector:

a) Site plan.
b) Gross section and other design data of the canal upstream and downstream of theproposed location.
c) Canal discharge.
d) Sediment data.

    i) Silt load both suspended and bed load daily, fortnightly or monthly, as available.For suspended silt, data should be available at different depths along at least three equidistant verticals across the width, at the proposed site of the ejector. The bed load should be observed up to 0.2 *d,* where *d* is the depth of water(subject to a maximum of 0.5 m).
    ii) Permissible size of silt can safely be allowed downstream of the silt ejector. In case of power channels generally sediment size larger than 0.2 mm is intended to been ejected.

e) Data for existing reach of the outfall channel:
    i) Contour plan.
    ii) Cross sections.
    iii) Stage discharge curve and the hydrograph of the stream at the outfall; and
    iv) Discharging capacity.

LOCATION

While deciding the location of the silt ejector, availability of suitable outfall channel has to be kept in view. The approach channel upstream of the ejector preferably be straight as otherwise it is likely to change the sediment concentration across the channel ,and disturb the uniform distribution of the flow in front of the ejector. In certain unavoidable cases where silt ejector has to be provided in the curved reach of the channel, it should be done after conducting model studies. The ejector should not be sited too near the head regulator as the residual turbulence may cause the sediment load to remain in suspension and prevent its ejection to the desired extent. At the same time it should not be far away from the head reach otherwise the sediment may settle down earlier and reduce the channel capacity upstream.

The working head available, that is, the difference in water level in the canal upstream of the ejector and the outfall channel at the exit of the ejector tunnel, shall be sufficient to extract the desired sediment. A working head of about 1.0 m is generally considered satisfactory for the purpose.

Generally, a silt ejector has the following components:

a) Approach channel.
b) Main structure which consists of:
   1) diaphragm,
   2) tunnels, and
   3) control structure.
c) Outfall channel.

APPROACH CHANNEL

In order to increase the concentration of sediment in the bottom layers, the cross sections of the approach channel should be increased by depressing suitably the bed and/or increasing the width, to reduce the velocity of flow to the desired limit. At the start of approach channel suitable transition/splay of the order of 1 in 5 should be provided so as to obtain streamline flow. The setting velocity of the silt particles can be calculated by — Stoke's law which is given as under:

Settling velocity, $V_s = l/18((P_s\text{-}P)/ty*g*d^2$

Where.

Vs = Settling velocity in cm/s,
P_B - Density of the particle in g/cm^3,
P - Density of the fluid in g/cm^3,
g = Acceleration due to gravity in cm/s^2,
D = Absolute viscosity of the medium in poise, and
d - Diameter of the particle in cm.

The layout of the approach channel may be decided with the above guidelines. The reduced velocity should be maintained for a sufficient length to achieve the desired sediment concentration in the bottom layers.

MAIN STRUCTURE

Diaphragm: The diaphragm shall be so designed that it causes least disturbance in the sediment concentration attained in the bottom layers of flow upstream of the ejector tunnels. In fixing the diaphragm level due consideration should be given to the following factors:

a) Desired sediment size to be trapped and extracted,
b) Bed level and size of tunnels,
c) Thickness of diaphragm, and
d) Bed level of canals downstream of the silt ejector.

The location of diaphragm can be determined by drawing the integrated discharge depth curve by using Vanoni's logarithmic velocity distribution,

$v = V+($ 1/k)*sqrt(gy$_0$s)*(l+2.31ogioy/yo)

where

v - Velocity of flow at a height y from channel bottom,
V =a Mean velocity of flow,
k - Von Karman constant (assumed usually as 0.4), s = Slope of the channel, y - Height from channel bottom, yo =Total depth of flow, and g - Acceleration due to gravity.
(However these parameters should be confirmed by suitable model test experiments).

It is desirable to place the diaphragm at the downstream bed level of the canal. However, if the diaphragm has to be placed higher from other considerations, the condition of all particularly for low supplies should be checked and, if necessary, proper energy dissipation arrangements provided.

The diaphragm should be properly tied to the supports as otherwise the diaphragm is likely to be dislodged.

The diaphragm shall be extended beyond the pier noses and the underneath of the diaphragm shall be given a streamlined bell mouth (elliptical) shape conforming to the following equation:

$$\frac{x^2}{4a^2}+\frac{y^2}{a^2}=1$$

Where,
$a =$ the thickness of the diaphragm

Tunnels — The ejector should normally span the entire width of the canal and shall be divided into a number of compartments of tunnels by vanes gradually converging so as to accelerate the escaping flow for delivering it to the outfall channel on one side of the canal. These main compartments shall be subdivided into smaller compartments or sub-tunnels by vanes of radii varying from 3 to 4 times the width of sub-tunnels to avoid cross flow in the transition section. The upstream noses of vanes shall have cut water shapes. Downstream end of vanes shall be fish tailed.

The tunnel dimensions at the entry and exit shall be so fixed as to ensure velocities that would carry the size of sediment to be removed. The section of the sub-tunnel at the entry shall be so chosen that the velocity of flow at the intake is slightly higher than the velocity of bottom filaments of water upstream of the ejector. The section of sub-tunnels up to their exit, where these end into the main tunnels, shall be reduced gradually in such a way that there is an overall increase of 10 to 15 percent in velocity of emerging flow.

At the exit of sub-tunnels the section of the main tunnel shall be designed such that the flow velocities of the combined discharge are not less than the velocities emerging out from the sub-tunnels. The section at the exit of the main tunnels shall be so designed as to attain a velocity of 2.5 to 6 m/s depending on the grade of sediment to be ejected, but in all cases, the exit velocity shall be less than the critical velocity. The depth of the main tunnels should be kept about 1.8 m to 2.2 m to facilitate inspection and repair work- Their width shall be so adjusted as to have equal losses in each tunnel. The tunnels shall be designed to run full bore to secure maximum efficiency.

Control Structure — The discharge from the sediment ejector is controlled by set of emergency and regulating gates. The quantum of discharge to be run through sediment ejector and frequency of its operation would vary in different parts of the year depending on the sediment load carried in the canal and this is achieved by operating regulating gate as required. It would be desirable to operate the gates fully open or fully closed.

OUTFALL CHANNEL

The outflow from the ejector is led to a natural drainage through an outfall channel. The outfall channel should be designed to have a self-cleaning velocity so that the ejected material is transported without deposition. Adequate drop between the full supply level of the outfall channel at its tail end and the normal high flood level of the natural stream is desirable for efficient functioning of the channel.

Of the several formulae available, the following equation by Neil can be adopted for working out approximate flushing velocity:

$$(P/Dy)*(V_c^2/d_g) = 2.5(d_g/yo)^{-a2}$$

Where

V_c = Competent mean velocity for first displacement of bed material, P = Mass density of the fluid, d_g = Effective diameter of bed grains, *yo* - Depth of flow, and Dy = Specific weight of the bed material in the fluid.

The velocity adopted should be well in excess of that worked out above. The adequacy of the section of the escape channel, especially when the available bed slope is not steep, should be tested for its carrying capacity. A triangular distribution of sediment concentration can be assumed at the mouth of the ejector to work out the quantity of sediment entering the ejector. The section of the escape channel should be so designed as to have adequate capacity to transport the total quantity of sediment entering it.

ESCAPE DISCHARGE

The escape discharge will be governed by the following considerations:

a) Discharge required to remove the desired sediment size and load,and
b) Minimum discharge required for flushing individual tunnels.Generally an escape discharge equal to 10 to 20 percent of the full supplydischarge of the canal downstream of the ejector will be adequate for this purpose.

LOSSES IN TUNNELS

These shall comprise of the following losses:

1) Friction Losses
2) Loss Due to Bend
3) Contraction Losses
4) Expansion Losses.

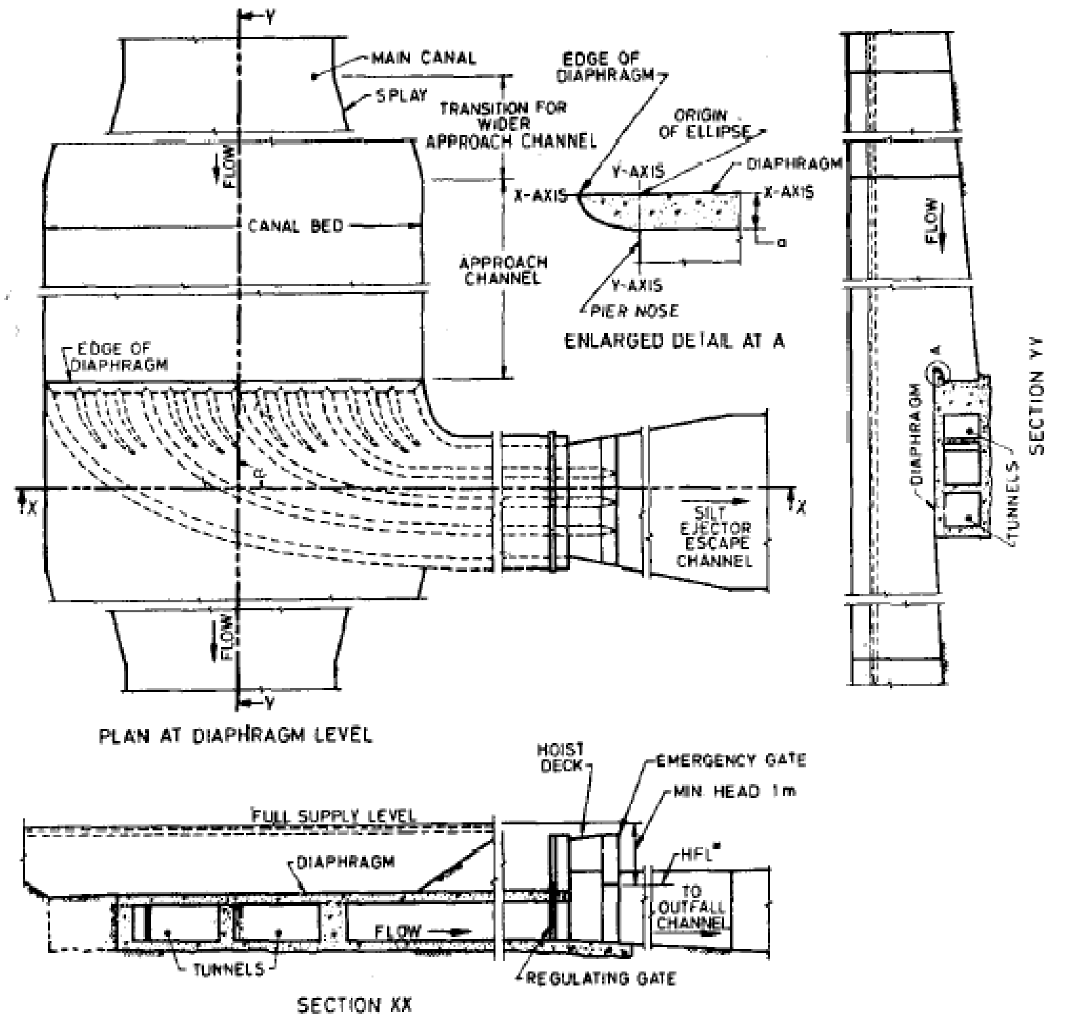
FLUSHING

During the period when sediment ejector is not required to function, it is desirable to operate the regulation gates occasionally for short periods to flush the tunnels consistent with the economy *in* water requirements for irrigation and power generation. Otherwise, the tunnels are likely to get choked and may require manual clearance which may be possible only during closure of the canal.

At times during the normal operation of the sediment ejector, the approach channel and/or tunnels or both may require flushing. This may be done by running the tunnels in rotation to achieve higher velocities.

TYPICAL DESIGN

A typical sketch of sediment ejector is shown below :



NOTE 1 — Energy dissipation arrangement d/s of channel and in the escape channel not shown.

NOTE 2 — Radii of vanes to be such that requirement of clauses **6.2** to **6.2.3** are fulfilled.

*HFL/water level in exit channel.

FIG. 1 TYPICAL DESIGN FOR SEDIMENT EJECTOR

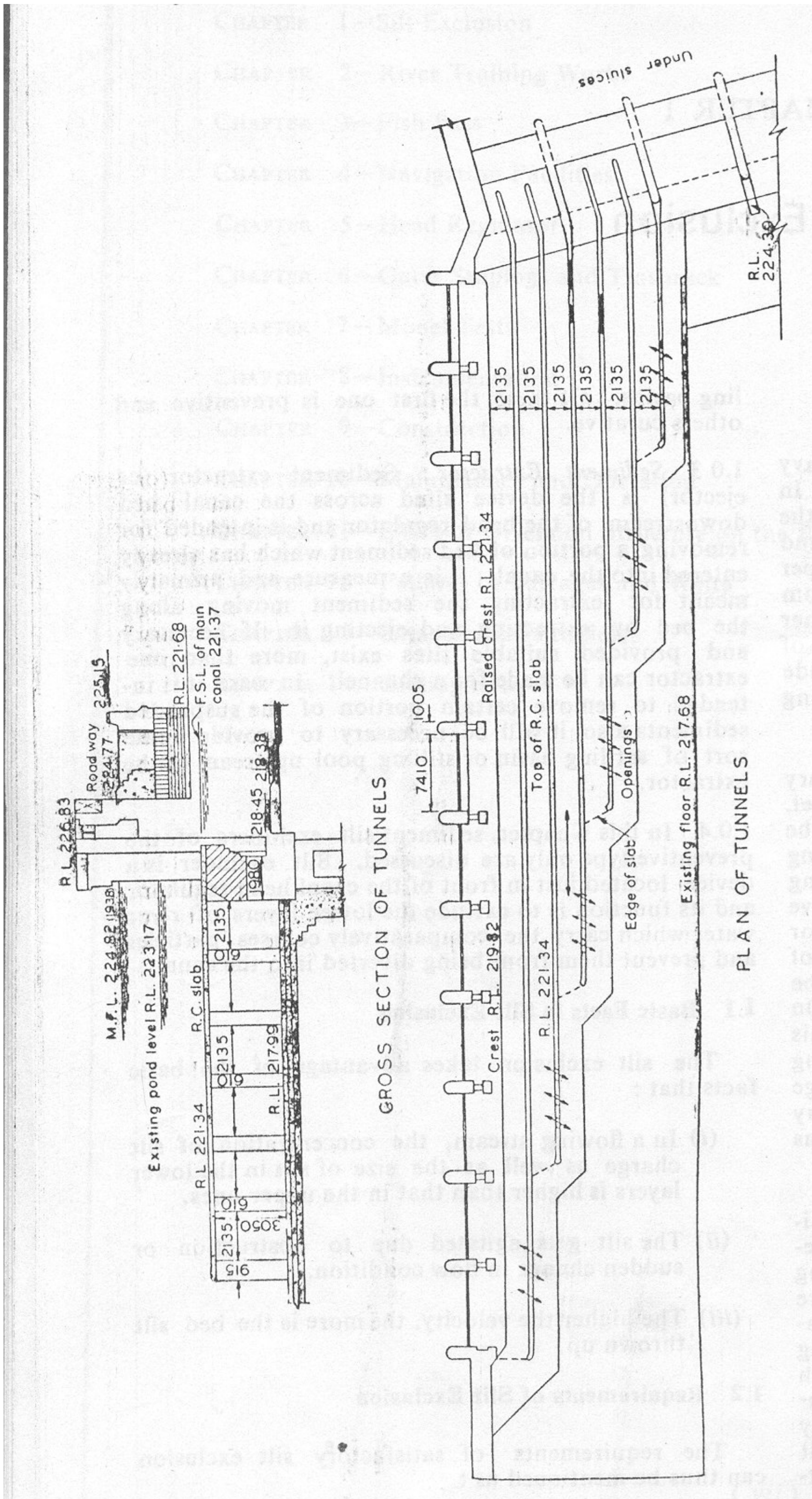


FIGURE 1.1 : General Layout of Khanki Type of Silt-Excluder.

HYDRAULIC MODEL STUDIES

There are many unknown factors in the design of silt ejector, such as the capacity of the silting basin in the approach channel, layout of the sub-tunnels and main tunnels, flushing velocity for the particular characteristics of the sediment to be ejected, and flow pattern of the bottom layers of the discharge, etc. As such it is essential that the layout based on the theoretical design be checked by model studies to ascertain the efficiency of the silt ejector.

Chapter-6

Planning and design of river training works

River training, in its broad aspects, covers all engineering works constructed on a river to guide and confine the flow to the river channel, and to control and regulate the river bed configuration for effective and safe movements of floods and river sediment.

5.0 CLASSIFICATION OF RIVER TRAINING PROBLEMS

River training aims at controlling and stabilizing a river along a desired course with a suitable waterway, for one or more purposes.

a) Flood Protection

River training for flood protection is also referred as high water training. It aims at providing sufficient cross section area for safe passage of flood. It consists of proper location, alignment and height of embankments for the design discharge.

b) Maintaining a safe & good navigable channel

This is sometimes referred as low water training. It aims to provide sufficient depths in the channel during lean periods for smooth navigation. It is done by concentrating the flow in a desired channel and closing other channels.

c) Sediment Control

This is referred as mean water training. This type of training aims at rectification of river bed configuration and efficient movement of suspended and bed load for keeping channel in good shape. The maximum aggrading capacity of a stream occurs in the vicinity of mean water or dominant flood discharge and therefore changes in the river bed should be attempted at this stage.

d) Stabilising the channel for prevention of Bank Erosion

Bank erosion is a common feature during the process of meandering. The river course also keeps on changing. There is a tendency of meanders either to shift progressively downstream or form cut offs. The process of meandering and consequently bank erosion continues and is often a recurring problem. Bank protection is therefore an important part of river training.

e) Directing the flow in a defined stretch

This is often required to protect hydraulic structures like barrages and bridges to avoid the risk of outflanking. It involves training over a considerable reach of a river. The structures required are Guide banks or Bunds.

5.1 Types of River Training Works

The following list enumerates various types of river training & control works in vogue:-

1) Levees or embankment
2) Bank protection and pitched bank
3) Groynes or Spurs
4) Guide bank system
5) Bed bars
6) Pitched islands
7) Miscellaneous methods.

5.1.1 LEVEES OR EMBANKMENT

A levee or dyke may be defined as an earthen embankment extending generally parallel to the river channel and designed to protect the area behind it from overflow of flood waters.

Embankments are the oldest known forms of flood protection works and have been used extensively for this purpose. These serve to prevent inundation, when the stream spills over its natural section, and safeguard lands, villages and other properties against damages.

5.1.2 Classification of Embankment

Embankment Manual, CWPC, 1960 stipulates that an embankment is designated as low, medium or major (according to its height above natural surface level (NSL).The details are as under in Table 1-1.

Table 1-1: Classification of Embankment

Classification of Embankment	Criterion
Low Embankment	Height < 10ft (3m)
Medium Embankment	10ft (3m) < Height < 30ft (9m)
Major Embankment	Height > 30ft (9m)

5.1.3 Design Flood for Fixing Crest Level

5.1.3.1 Embankment for Predominantly agricultural areas

The design flood for this type of embankment is kept 25 years for fixation of crest level.

5.1.3.2 Embankment for Township or areas having Industrial installations

The design flood for this type of embankment is kept 100 years for fixation of crest level.

In the cases where anti erosion measures are proposes along with the embankment then design flood may be kept as 50 years for rural areas and 100 years for urban / industrial areas. In certain special cases, where damage potential justified, maximum observed flood may also be considered for fixing the crest level

5.1.3.3 Alignment, spacing & length of embankment

5.1.3.1 Alignment

The embankments should be aligned on the natural bank of the river, where land is high and soil available for the construction of embankments. The alignment should be such that important township, vital installations, properties, cropped area is well protected by the embankment. The alignment should be such that high velocity flow is quite distant from the toe of embankment to avoid scouring of the same and if embankment's alignment is near the high velocity flow then slope and toe protection in the form of pitching along with launching apron using the boulders, geo-bags, sand filled geo-mattress may be given. RCC porcupine screens along the toe line may also be used to retard the flow to induce siltation and check scouring of the toe-line. Alignment should also be planned so that land acquisition is feasible and not prolonged.

5.1.3.2 Spacing

The spacing of the embankment and their alignment needs careful consideration with respect to their vulnerability to the river and the rise of high flood levels on account of reduction in flood plain storage by construction of the embankment.

The spacing of embankments along the jacketed reach of the river should not be less than 3 times Lacey's wetted perimeter for the design flood discharge. The minimum distance of the embankment from the river bank and midstream of the river should be one times Lacey's wetted perimeter and 1.5 times Lacey's wetted perimeter [Lacey's wetted perimeter $P = 4.75\ (Q_{design})^{1/2}$] respectively.

In the tidal reach of the river, embankments should be constructed with due regard to their effect on navigation requirements in the channel as embankments in such cases may reduce the tidal influx causing a reduction in available navigation depth.

5.1.3.3 Length of the embankment

The length of the embankment directly depends upon the alignment. However, it is to be ensured that both ends of the embankment are tied up to some high ground or existing highway or railway or any other embankment nearby conforming to the design height of the embankment.

5.2 Design of embankment

BIS code 12094: 2000 is used for design of the embankment

5.2.1 Types of embankment

As per Embankment Manual, CW&PC, 1960 and Irrigation and Hydraulic structures by S.K. Garg, embankments can be classified into three types as given below.

5.2.1.1 Homogenous embankment

This is the simplest type of earthen embankment and consists of a single material and is homogeneous throughout. Sometimes, a blanket of relatively impervious material (stone pitching) may be placed at river side. A purely homogenous section is used, when only one type of material is economically or locally available. Such sections are used for low heights.

A purely homogenous section, made of pervious material, poses problems of seepage, and huge sections are required to make it safe against piping, stability etc. Due to this, homogenous section is generally provided with an internal drainage filter like horizontal filter so that Hydraulic gradient line (HGL) is confined to the section. The length of horizontal filter may be kept as 3 times the height of embankment.

Rock toe or Toe filter of (height = 25% To 35% of water height) consisting of fine sand, coarse sand and gravel, as per filter criterion requirement, may also be provided to check the seepage.

5.2.1.2 Zoned embankment

It consists of an inner core or section which is impervious and which checks the HGL. The transition zone prevents piping through cracks which may be developed in the core due to shrinkage and swelling of core material. The outer zone gives stability to the central impervious core and also distributes the load over a larger area of foundations.

The core of the embankment may be constructed using the clay mixed with the fine sand or fine gravel. Pure clay is not best material for core as it shrinks and swells too much. Silt or silty-clay may also be used as core. Sand filled geo-tube, which is a relatively impervious material, may also be used as core of the embankment.

5.2.1.3 Diaphragm type embankment

Diaphragm type embankment has a thin impervious core, which is surrounded by sand. The impervious core, called as diaphragm, is made of imperious soils, concrete, steel, timber or any other material. It acts as water barrier to prevent seepage. The diaphragm must be tied to the bed rock or to a very impervious foundation material.

The diaphragm type of embankment is differentiated from zoned embankment, depending upon thickness of core. If thickness of diaphragm is less than 10m or height of embankment, the embankment is to be considered as diaphragm type embankment. Sand filled geo-tube may also be used as core or diaphragm of the embankment.

5.2.1.4 Free board

The top of the embankment should be so fixed that there is no danger of overtopping. Even with the intense wave wash or any other unexpected rise in water level due to sudden change in river course or aggradations of river bed or settlement of embankment.

The waves are generated on the surface by the blowing winds. Height of water wave mainly depends upon the wind velocity (V in km/hr) and fetch or straight length (F) of water expanse in Km. Wave height may be calculated using the equation given below.

$$H_w \text{ (in m)} = 0.032\,(V.F)^{1/2} + 0.763 - 0.271\,(F)^{1/4} \text{ for } F < 32 \text{ km}$$

$$H_w \text{ (in m)} = 0.032\,(V.F)^{1/2} \text{ for } F > 32 \text{ km}$$

The freeboard for wave action may be taken as 1.5x wave height (h_w).

However in the absence of the wave data, free board should be taken as 1.5 for discharges less than 3000 cumecs and 1.8m for discharges more than 3000 cumecs. This should be checked also for ensuring a minimum of about 1.0m of free board over HFL corresponding to 100 years frequency.

The free board proposed above as 1.5m / 1.8m is recommended in case of less reliable and short duration hydrological data to take care of uncertainty. In case hydrological data is collected using the reliable sources and length of such data is sufficient long like 35 years, then lesser values of freeboard like 1.0m / 1.5m may be adopted.

It is also suggested to work out the maximum discharge corresponding to the crest level (adding the free board to the design HEL). So that this maximum discharge can be compared with the higher return period discharges like SPF, PMF etc.

5.2.1.5 Top width

The top width of the embankment should be sufficiently enough to accommodate the vehicular traffic. The top width of the embankment may be kept as 5.0m. Turning platform of length 15m to 30m and 3m width a c/s side slope at an interval of 1km or more may be provided.

5.2.1.6 Hydraulic gradient

It is desirable to know the approximated line of seepage or hydraulic gradient line (HGL). The following guidelines may be used for determining the HGL.

Clayey soil	:	4H : 1V
Clayey sand	:	5H :1V
Sandy soil	:	6H :1V

5.2.1.7 Side slope

The side slopes are dependent upon the material and height of the embankment. The side slope should be flatter than the angle of repose of the material of the embankment. For drainage purpose, longitudinal drains on the berm and cross drains at suitable places should be provided to drain out the water. In order to provide communication from one side of embankment to another side, ramps in a slope of 4H:1V at suitable places and all village paths should be provided as per requirement.

5.2.1.8 River side slope

The river side (R/S) slope should be flatter than the under-water angle of repose of the material. Up to an height of 4.5m, the slope should not be steeper than 2H:1V and in case of high embankments, slope should not be steeper than 3H:1V, when the soil is good and to be used in the most favorable condition of saturation and drawdown.

a) In case of higher embankment protected by rip-rap / pitching, the slope of embankment up to 6m high may be 2H:1V or 2.5H:1V depending upon the type of slope protection.

b) If the construction material is sandy, the slope should be protected with a cover of 0.6m thick good soil; and

c) It is usually preferable to have more or less free draining material on the river side to take care of sudden drawdown. In case of high and important embankment, slopes may be protected by the stone pitching, concrete block with open joints or sand filled geo-mattress to protect against sudden drawdown or erosive action of river flow.

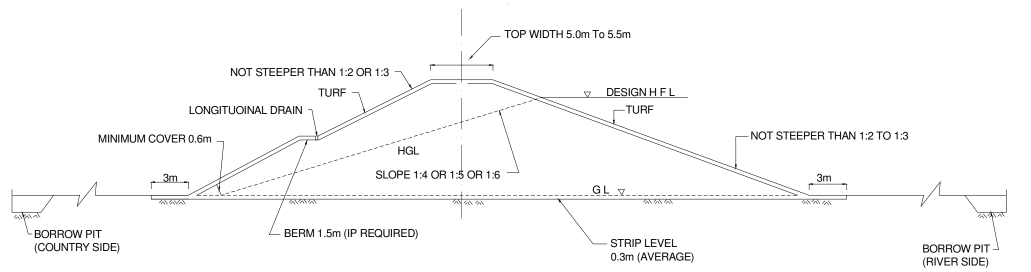
d) For embankment with height more than 6m, line of saturation should be found by Kozeny’s base parabola method and stability analysis should be carried by slip circle method for finalizing river side slope (IS 7894).

5.2.1.9 Country side slope

A minimum cover of 0.6m over the HGL should be maintained. For embankment up to height of 4.5m, the country side slope should be 2H:1V from the top up to the point where the cover over HGL is 0.6m after which a berm of suitable width, with country side slope of 2H:1V from the end of the berm up to the ground level should be provided.

For embankment of height from 4.5m to 6.0m, the country side slope should be 3H:1V from the top up to the point where the cover over HGL is 0.6m after which a berm of suitable width, with country side slope of 3H:1V from the end of the berm up to the ground level should be provided.

For embankment of height more than 6m, detailed design should be done. Typical cross section of an earthen embankment is shown as under as Figure given below.



TYPICAL CROSS-SECTION OF AN EMBANKMENT

5.2.1.10 Borrow Pits

As per BIS code 11532, for taking out soil for use in embankment, borrow pit should be preferred on the river side and located at minimum distance of 25m from the toe of embankment. In order to obviate development of flow parallel of embankment, cross-bars of width 8 times the depth of borrow pits @ 50-60m c/c shall be let in the borrow pits. When adding new earthwork to existing embankment, the old bank shall first be cut and benched into steps with treads sloping slightly towards centre of the embankment. Surface of old work should be properly wetted so that new earth may adhere to old.

5.2.1.11 Drainage

For drainage, longitudinal drains should be provided on the berm and cross drains at suitable places should be provided to drain the water from the longitudinal drains. Toe drain should be provided to prevent sloughing of toe. Perforated pipe embedded in properly designed graded filter with arrangements for disposal of water in the country side should be provided.

5.2.1.12 Causes of failure of embankment

As stipulated by the CBIP publication-1989 River Behavior Management and Training Volume-I. In the absence of proper maintenance and supervision, embankments are susceptible to breaches due to various causes given below.

a) Improper compaction and settlement of embankment.

b) Transverse cracks due to unequal settlement.
c) Inadequate drainage and pore pressure development.
d) Erosion of riverside slope due to river current and wave wash.

5.3 Preventive measures

As stipulated by the CBIP publication-1989 River Behaviors Management and Training Volume-I, breaches/failures can be avoided by adopting suitable preventive measures mentioned briefly underneath.

a) Toe drainage.
b) Placing sands bags near toe (with drains covered by wooden planks) in order to increase shear resistance actuating forces causing slip.
c) Reducing seepage by lowering seepage head by constructing ring wells with sand bags near the toe.
d) Plugging piping holes with divers using tarpaulins soaked with bitumen form river side face of the hole.
e) Raising height of embankment (in case of overtopping) by using wooden planks without endangering stability against slip.

5.4. Bank protection and pitched bank

5.4.1 Design of bank revetment

IS code 14262:1955 provides for following provisions regarding design of bank revetment.

5.4.2 Weight of stones/boulders

Stones / boulders, used in revetment for bank protection, are subjected to hydrodynamic drag and lift forces. These destabilizing forces are expressed in terms of velocity, tractive forces etc. The stabilizing forces acting against these are component of submerged weight of the stones and downward component of force caused by contact of the stones.

The weight of stones on slopes (W in kg) may be worked using the formula given below:

$$W = (0.02323 \text{ x } S_s \text{ x } V^6) / [K \text{ x } (S_s - 1)^3]$$

Where,

K = correction factor of slope

$$K = [1-(\sin^2 \theta / \sin^2 Ø)]^{½}$$

And,

S_s = specific gravity of stone

θ = angle of sloping bank

$ø$ = angle of repose of material of protection works

v = velocity in m/s at bank

For pitching, the natural bank is to be graded to a stable slope depending upon the angle of repose and cohesion of bank material under saturated condition. For high banks, berm needs to be provided. For important works, stability of bank with designed slope and berm should be checked by slip circle method or by soil dynamic testing procedures. For normal bank protection works, a slope of 2H:1V or flatter is recommended.

5.4.3 Size of Stone/Boulder

Size of stone (D_s in m) may be determined from the following relationship.

$$D_s = 0.124 \text{ X } (W / S_s)^{1/3}$$

Where,

W = Weight of stone in kg, and

S_s = Specific gravity of stones.

Generally, the size of stone should be such that its length, width and thickness are more or less same is stones should be more or less cubical. Round stones or very flat stones having small thickness should be avoided.

2.1.3 Thickness of Pitching

Minimum thickness of pitching (T) or protection layer is required to withstand the negative head created by the velocity. This may be determined by the following equation.

$$T = V^2 / 2g (S_s - 1)$$

Where,

S_s = Specific Gravity of Stones.

Therefore thickness of pitching should be higher than thickness obtained from the above equation.

However, two layers of stones of minimum size `T' should be provided, when pitching is being provided with boulders in loose.

5.4.3.1 Pitching in crates

At high velocity, required weight of stones comes out to be higher, which makes handling and placing of stones a bit difficult. In such cases or in case when requisite sized stones are not available, small size stones filled in GI (Galvanized Iron) wire crates may be used for pitching purpose. In this case single layer of GI wire crates filled with stones having thickness more than minimum thickness of pitching (T) may be used as pitching. The specific gravity of the crate is different from the boulders due to presence of voids. Porosity of the crates (e) may be worked out using the following formula.

$$e = 0.245 + 0.0864 / (D_{50})^{0.21}$$

Where,

D_{50} is mean diameter of stones used in crates

The opening in the wire net used for crates should not be larger than the smallest size of stone used. The mass specific gravity of protection (S_m) can be worked out using the following relationship.

$$S_m = (1\text{-}e) \times S_s$$

Crates should be laid with long dimension along the slope of the bank. Crates must be tied to each other by 5 mm GI wire as additional protection. If crates are being provided in layers then each layers should be tied to each other at suitable interval using the 4mm GI wire.

5.4.3.2 Filter

A graded filter of size 150 mm to 300 mm thickness may be laid beneath the pitching to prevent failure by sucking action by high velocity. Geo-synthetic filter may also be used as that is easy to lay, durable, efficient and quality control is easy. A 150 mm thick sand layer over the geo-synthetic filter may be laid to avoid rupture of fabric by the stones.

5.4.3.3 Paneling

Paneling may be provided in the pitching where slope length is more so that slopes may remain stable. The size of panel may be varied depending upon the length of river reach to be protected and the length of slope length

5.4.3.4 Top key / Berm

In case of revetment on slopes up to NSL, which is below HFL, a top key or capping berm should be provided for allowing flow of water over the top surface of the revetment.

5.4.3.5 Pitching in mortar

Is code 14262:1995 mentions following provisions regarding pitching in mortar.

5.4.3.6 Size of stones

Stones bricks or concrete blocks may be used for construction of pitching in mortar. Size of stones / bricks / concrete blocks in this type of pitching is not a critical aspect of design as very individual complement is bounded by mortar. Average size of available stone can be used for this purpose. But thickness of such pitching should be more than minimum thickness of pitching (t).

5.4.3.7 Panelling

Mortar revetment should not be constructed in continuous or monolithic form. To avoid cracks, joints at suitable interval may be provided. Generally revetment may be provided in panels of size 3m x 3m or 3m x 5m. The size of panel may be varied depending upon the length of river reach to be protected and length of slope. Standard stone filter or geo-synthetic filter may be provided beneath the joints.

5.4.3.8 Drain holes

Drain holes or weep holes may be provided in each panel for free drainage of pore water from saturated bank soil beneath it. Depending upon the size of panel, one or more weep holes may be provided for a panel. The pipe provided in the drain hole should be up to the natural bank. Stone graded filter or geo-synthetic filter may be provided at the end at the conduct of the bank soil.

5.4.3.9 Pitching by geo-textile bags

As per design practices following guidelines may be adopted for pitching by geo-textile bags.

5.4.3.10 Size of geo-bag

The pitching may also be provided using sand filled geo-synthetic bags. The size of bags may be 1.1m x 0.7m x 0.15m. The weight of such bags is around 126 kg which is generally safe for the velocity up to 3 m/s. For higher velocities, size of Geo-bag may be higher so that weight of bag is higher than the required weight which is the weight of stones on slope. The geo-synthetic material should be safe against the UV rays and abrasion.

5.4.3.11 Thickness of geo-bags pitching

The thickness of Geo-bag pitching may be decided as per procedure given above at para 2.1.3. To summarize again, thickness of pitching should be more than minimum thickness of pitching (T). Pitching may be provided in double layers of geo-bags (in loose) and single layer (encased with nylon / polypropylene ropes).

5.4.3.12 Filter

If the pitching is being provided in geo-bags, then generally filter is not provided because material of Geo-bags itself work as filter. But for safety purpose (for taking care of bank soil in joints), a geo-synthetic filter layers beneath the geo-bags may be provided.

5.5 Toe protection

IS code 14262:1955 mentions the following provisions regarding toe protection. To prevent the sliding and failure of the revetment on slope, toe is required to be protected. This may be in the form of simple toe-key, toe wall sheet pile or launching apron.

5.5.1 Toe key

Simple key may be provided at the toe (may be called as toe key) when rock or un-erodible strata is available just below the river bed and the overlaying banks are erodible. The key is in the form of stone / bricks or concrete blocks filled in the trench below the hard river bed for depth equal to the thickness of pitching (t) for proper anchorage. Sole purpose of this key is to provide lateral support to the pitching. The key may be of mortar or in geo-bags, if the pitching is provided in mortar or geo-bags.

5.5.2 Toe wall

When a hard stratum is available below the river bed at reasonable depth, toe wall is recommended. The thickness of the toe wall depends upon height of wall and height of overlaying pitching. The toe wall may be design as retaining wall and be constructed in masonry along with provisions of weep holes etc.

5.5.3 Sheet piles and launching apron

When a firm stratum is not available at reasonable depth below the river bed, toe protection in the form of sheet pile or launching apron may be provided. The sheet pile may be made of RCC, steel or bamboo. The sheet piles may be drilled below the river bed up to maximum scour depth.

Sheet piles are difficult to drive; therefore Launching apron is preferred and provided with revetment. Launching apron should be laid at low water level (L.W.L). The launching apron may be laid using the stones or geo-bags. The stones/geo-bags in the apron should be designed to launch along the slope of scour and provide a protection layer so that scouring is checked. The size of launching apron should be such that it should form a protection layer up to level of maximum scour depth. Slope of launching apron may be taken as 2H:1IV. Filter below the launching apron may also be provided so that river bed material is safe against sanction.

5.5.4 Design of launching apron as per BIS 10751

The thickness of launched apron should be 25% to 50% more than the thickness of pitching on slopes. (as shown in para 2.1.3)

Design of launching apron may be carried out as given below:

Depth of scour (D): depends on angle of attack, discharge intensity, duration of flood and silt concentration etc and is given as:

For waterway ≥ lacey's waterway and flow is non uniform then the depth of scour should be as evaluated by

$$\text{Depth of Scour (D)} = 0.473(Q/f)^{1/3} \qquad \text{(Single channel)}$$

For waterway < lacey's waterway then the depth of scour should be as evaluated by

$$D = 1.35\,(q^2/f)^{1/3} \qquad \text{(Multi braided channels)}$$

Where,

Q = Discharge in m^3/sec

q = Discharge intensity in m^3/sec / m

f (Silt factor) = $1.76\sqrt{d}$

d = mean diameter of river bed material

Maximum / design scour depth: the (D) calculated as above shall be multiplied by a factor varying from 1 to 2.5 depending upon the reach (straight, curved, u/s & d/s etc) to estimate maximum / design scour depth

Deepest scour level = HFL – Maximum scour depth

D_s = depth of scour for calculation of apron stone may be calculated as:

$$D_s = \text{LWL} - \text{deepest scour level}$$

If the launching apron is being laid at LWL then width of the launching apron should be calculated using the following formula.

$$\text{Width of launching apron} = 1.5\,D_s$$

Slope of apron after launching: generally, the slope of launching apron may be taken as 2H:1V for loose boulders or stones. Hence the sloped length of launched apron is

$$(\sqrt{5}) * D_S$$

Quantity of stone required for launched apron:

$$(\sqrt{5}) * D_S * T$$

Where,

T = thickness of launched apron which is 25% to 50% more than the thickness of pitching on slopes.

Shape of launching apron: the quantity of stone so calculated may be provided in a width of $1.5D_s$ and average thickness (T). Thickness of laid apron may be kept as 0.8Tnear the toe of pitching and 1.2T at the river end.

Apron should be laid at lowest water level, as far as economically viable.

5.6 Anchoring

IS code 14262: 1955 mentions following provisions regarding anchoring.

Proper anchor is required for keeping the revetment in place and serving the desired function. Upstream edge from where the revetment starts should be secured well to the adjoining bank. Similarly, downstream edge where the revetment ends also needs to be secured well to the adjoining bank. Anchorage is also required to be provided on the top of submerged bank. If the top of bank is above HFL, the revetment should be provided above HFL with an adequate free board say 1.0m. Under such situation, anchorage at top is not required.

5.7. GROYNES OR SPURS

This type of protection comes under anti erosion works. Groynes are structures, constructed transverse to the river flow and extended from the bank into river. These types of works are provided to keep away flow from the erosion prone bank. The spurs are provided along with launching apron to prevent scouring under the water.

5.7.1 General Design Features

5.7.1.1 Alignment

Groynes may be aligned either normal to the dominant flow direction or at an angle pointing upstream or downstream.

A Groyne pointing upstream repells the river flow away from it and is known as **repelling groynes**. It may be kept at an angle of 5^o to 10^o against the direction of flow. A Groyne pointing downstream attracts the river flow towards it and is known as **attracting groynes.** When a short length groyne changes only direction of flow without repelling, it is known as **deflecting groynes**.

5.7.1.2 Functions of Groynes

They serve one or more of the following functions:

a) Training the river along the desired course to reduced the concentration of flow at the point of attack.
b) Protecting the bank by keeping the flow away from it.
c) Creating a slack flow with the object of silting up the area in the vicinity of the river bank.
d) Improving the depth of river for navigation purpose.

5.7.1.3 Classification of Groynes

These can be classified as follows:

a) The methods and material of construction, namely permeable, impermeable, and slotted.
b) Height of groyne with respect to water level, namely submerged, non-submerged and partially submerged (sloping).
c) Action, namely deflecting, attracting and repelling.
d) Special shapes, namely T-headed, hockey type or Burma type, kinked type.

5.7.1.4 Orientation of Groynes

IS code 8408:1994 and draft for revision mention following provisions regarding orientation of groynes / spurs.

It can be used single or in series, depending upon the reach of length to be protected. It can be used in combination with other river training measures. The spacing, orientation and length of groynes may be decided by model study.

5.8 Design of Groynes / Spurs

5.8.1 Design Discharge

The design discharge for the groyne should be equal to that for which structure in close proximity is designed or 50 year flood whichever is higher.

5.8.2 Length of Groyne

Length of groyne should be decided on the basis of availability of land on the bank. Length should not be less than that required to keep the scour hole formed at the nose away from the bank. Thus assuming angle of repose of sand to be 2.5H:1V and anticipated maximum depth of scour below bed be (d_s). The length should be more than 2.5 (d_s).

Normally the effective length of groyne should not exceed $1/5^{th}$ of width of the flow in the case of single channel. In case of wide, shallow and braided river, the protrusion of the groyne in the deep channel should not exceed $1/5^{th}$ of the width of channel on which the groyne is proposed excluding the length over the bank.

The spacing of the groyne is normally 2 to 2.5 times of its effective length. For site specific cases model studies may be conducted.

5.8.3 Top Level of Groyne

The top level of groyne will depend on the type namely, submerged, partially submerged or non-submerged and will be best decided by model experiments. In case of non-submerged groynes, the top level should be above design flood with adequate free board.

5.8.4 Top Width of Groyne & Freeboard

The top width of groyne should be 3 to 6 m as per requirements. A freeboard of 1 to 1.5 m should be provided above the design flood level.

5.8.5 Side Slopes

Slopes of the side and nose of the groyne would be 2:1 and 3:1 depending upon material used.

5.9 GUIDE BANK SYSTEMS

Guide banks are constructed in the direction of flow both upstream and downstream of structure, on one or both flanks as required.

5.9.1 Alignment

- The alignment should be such that the pattern of flow is uniform with minimum return currents.
- The alignment and layout are best decided based on model studies.

- In case of a head regulator of a canal, constructed adjacent to the main structure, the alignment of the guide bank should further endeavour to induce favorable flow conditions for the entry of water with minimum silt into the canal.
- In other cases, guide banks should be so aligned that the flow is uniformly distributed across the waterway as far as possible.

5.9.2 Classification of Guide Banks

Guide banks can be classified according to

a) Their bank in plan, and
b) Their geometrical shape.

5.9.3 According to Form in Plan

Guide banks can be divergent upstream, parallel and convergent upstream.

5.9.3.1 Guide banks divergent upstream

They exercise an attracting influence on flow and they may be used where the river has already formed a loop and the approaching flow has become oblique to keep flow active in bays adjacent to them. However, the approach embankment gets relatively lesser protection in worst possible embayments compared to equal bank length of parallel guide banks.

Divergent guide banks require a longer length in comparison to parallel guide banks for the same degree of protection to approach embankments. However, hydraulically they give better distribution of flow across the waterway.

5.9.3.2 Parallel guide banks

Parallel guide banks with suitably curved heads have been found to give uniform flow from the head of the guide banks to the axis of the structure.

5.9.3.3 Guide banks convergent upstream

Convergent guide banks have disadvantage of excessive attack and heavy scour at the head and shoaling all along the bank rendering the end bays inactive.

5.9.3.4 According to Geometrical Shape

The guide banks can be straight or elliptical with a circular or multi-radii curved head (see in Fig.). Elliptical guide banks have been found more suitable in case of wide flood plain rivers for better hydraulic performance. In case of elliptical guide banks, the ratio of major axis to the minor axis is generally in the range of 2 to 3.5.

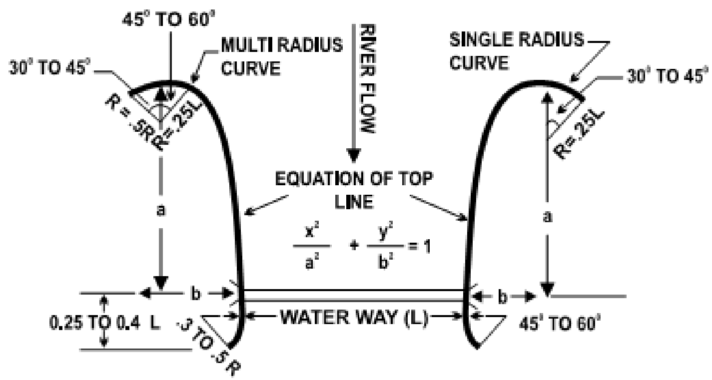


FIG. : ELLIPTICAL SHAPE OF GUIDE BUNDS

Due to gradual change in curvature in elliptical guide banks the flow hugs the guide banks all along its length as against separation of flow occurring in case of straight guide banks after the curved head which leads to obliquity and non-uniformity of flow.

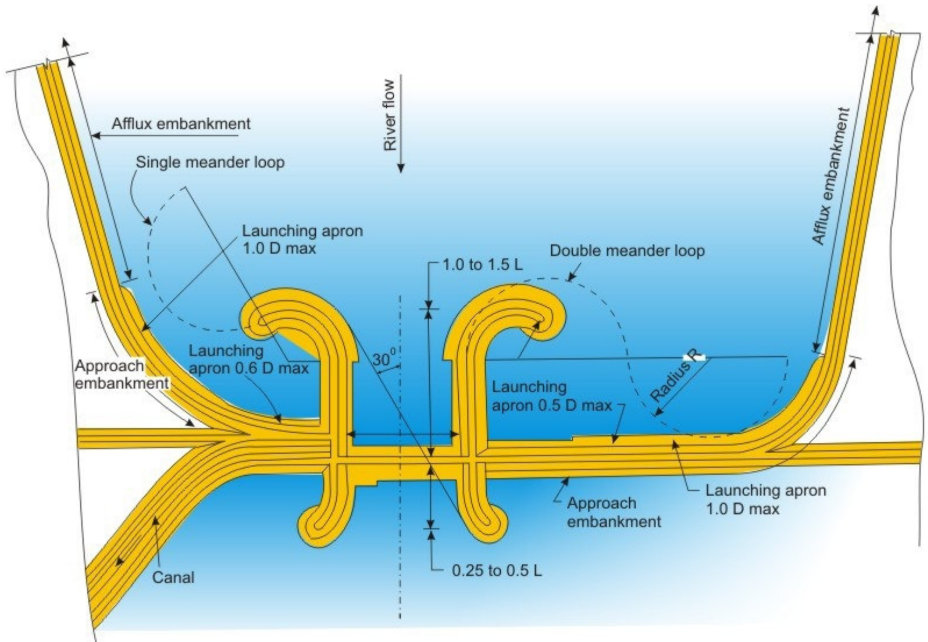
5.10 Length of Guide Banks

5.10.1 Upstream Length

- The general practice is to keep the upstream length of guide banks as 1.0 L to 1.5 L, where L is the length of structure between the abutments. For elliptical guide banks the upstream length (that is semi major axis a) is generally kept as 1.0 L to 1.25 L. This practice is generally applicable where the waterway is within the close range of L that is, Lacey's waterway.
- For wide alluvial belt the length of guide banks should be decided from two important considerations. Namely (a) the maximum obliquity of current (it is desirable that obliquity of flow to the rive axis should not be more than 30"), and (b) the limit to which the main channel of the river can be allowed to flow near the approach embankment in the event of the river developing excessive embayment behind the guide bank. The radius of worst possible loop should be ascertained from the data of acute loops formed by the river during past. In case of river, where adequate past surveys are not available, the radius of worst loop can be determined by dividing the average radius of loop worked out from the available surveys of the river by 2.5 for river having a maximum discharge up to 5000m^3/s and by 2.0 for discharging above 5000m^3/s. The limit to which the main channel of the river can be allowed to flow near approach embankment has to be decided based on importance of structure and local conditions.

5.10.2 Downstream Length

On the downstream side the river tries to find out to regain its natural width. The function of guide bank is to ensure that the river action does not adversely affect the approach embankment. The downstream length will therefore, has to be determined so that swirls and turbulence likely to be caused by fanning out of the flow downstream the guide bank do not endanger the structure and its approach. The length of 0.2 L to 0.4 L is recommended.



Typical layout of river training work.

5.11 Radius of Curved Head and Tail

- Function of curved head is to guide river flow smoothly and axially to the structure, keeping end spans active. A too small radius gives a kick to river current making it oblique and therefore larger radius to attract and guide the flow is needed, but it is uneconomical. Radius should be kept as small as possible consistent with proper functioning of bank. Radius of curve head equal to 0.45L has been found to be satisfactory.

- Radius of curved tail may be 0.3 to 0.5 times the radius of curved head.

- Considerable economy consistent with smoother conditions at the head may be achieved by adopting a composite curve of two or three different radius instead of a single large radius. This can be best decided by model studies.

5.12 Sweep Angle

The sweep angle is related to the loop formation. For curved head the angle of sweep may range from 120^{o} to 145^{o} according to river curvature. For curved tail it varies from 45^{o} to 60^{o}.

5.13 Design of Guide Banks

5.13.1 Material

Guide banks may be made of locally available materials from river bed, preferably silt, sand or sand-cum-gravel.

5.13.2 Top Width

The top width should be 6 to 9 m to permit transport of material. At the nose of guide banks, the width may be increased suitably to enable vehicles to take turn and for stacking stones.

5.13.3 Free Board

A free board of 1.0 m to 2.0 m may be provided above the design flood level. Where heavy wave action is apprehended and / or aggravation is anticipated, a higher free board may be provided.

5.13.4 Side Slope

It depends on the angle of repose of the material of guide banks and the height Side slopes of 2:1 to 3:1 are generally recommended.

5.13.5 Protection of Structures

- Curved head is prone to damage due to concentration of discharge caused by collection of over bank flow and direct attack of current obliquely. The shank is subjected to attack by parallel / oblique flow. The curved tail is subject to attack by fanning out of current.
- The effect of these attacks is the formation of deep scour holes at toe and erosion of river side slopes. Hence toe and slope both have to be protected

5.13.6 Toe Protection

Launching apron should be provided for protection of toe and it should form a continuous flexible cover over the slope of the scour hole in continuation of pitching up to the point of deepest scour. Launching apron should be laid at normal low water level, or at as low a level, or at as low a level as techno-economically viable. The stone in the apron should be designed to launch along the slope of the scour hole so as to provide a strong layer that may prevent further scooping out of the river bed material. The size and shape of apron depends on the size of stone, thickness of launched apron, the depth of scour and slope of launched apron.

The size of stone, thickness of launched apron, the depth of scour and slope of launched apron are covered in topic 2.13 &2.44.

5.14. BED BARS

A bed bar is a submerged structure dividing the flow horizontally. Flow above top level of the bed bar is comparable with weir flow while flow below top level is obstructed by the bar and diverted towards the nose as in case of a full height spur.

In case of flow over a weir, if its alignment is skewed with respect to approach flow, the magnitude and direction of the flow downstream of the weir is governed by the head due to afflux and that due to approach velocity. When the alignment of a bed bar is skewed, a pressure gradient is formed. When bed bar is facing upstream this pressure gradient helps sediment deposition on the upstream side of the bar while the surface flow gets deflected away from the bank. On the other hand, bed bar facing downstream directs the bottom current away from the bank while surface flow is deflected towards the bank protection and bed bar facing downstream is provided upstream of an offtake point for Sediment exclusion.

5.15. PITCHED ISLAND

The device of a pitched island is a recent innovation in the armoury of river training hydraulics. The basic principle underlying the behavior of a pitched island, used as a river training measure, is its ability to cause re-distribution of velocity and tractive force. The tractive force near a pitched island with the result that deep scour begins to form round the island and gradually draws the main river channel into itself and, ultimately, holds it permanently. Because of this property, the pitched island can be used, either singly or in series, for the following purposes:

- **a)** Correcting oblique approach upstream of weirs, barrages and bridges by training the river to be axial.
- **b)** Rectification of adverse curvature for effective sand exclusion.
- **c)** Redistributing harmful concentration of flow for relieving attack on marginal bunds, guide banks and river bends.
- **d)** Training the river in the reach away from control points, such as weirs and bridges.
- **e)** Improving channels for navigation.

It has been found by experience that the working of the pitched island is effective, especially in the vicinity of control points, such as weirs and bridges.

5.16. MISCELLANEOUS METHODS

River Training without Embankments

This is a special technique evolved on flashy streams in Burma and, in essence, amounts to building fences along their banks. The method has been employed on two groups of small streams emanating from the Pegu mountains, one flowing eastward and joining the Rangoon River and the other flowing westward to join the Sittang River, The catchment areas vary from 101 to 1,036 sq km (40 to 400 sq miles) and the maximum discharges from 200 to 400 cusecs per sq mile of catchment area. The method depends, for its effectiveness, upon the heavy sand charge carried by the streams during floods, the maximum intensity of charge amounting to about one per cent by weight. These hill streams develop and change their course, at random, on the plain and cause damage to communications. The objects of training are reclamation of swampy areas for paddy cultivation, improving the streams for transport of logs and facilitating their passage through rail-road bridges.

The stream depression is aligned and demarcated while stake fences consisting of bamboos 1.5 to 1.8m long spaced 25.4cm apart, are driven parallel to the selected concrete 25.4cm apart, are driven parallel to the selected centre line a predetermined distance. The fence is secured by lashing horizontal bamboos 15cm below the top. More rows of fences are installed where strong fences are required, or if the line passes through a depression, or again if the slope is steep. Sometimes, even 4 to 5 rows are driven on a sloping ground with successive rows getting closer.

During floods, the fences slacken velocity of flow and heavy sedimentation occurs. In falling floods, banks form over these fences. Over the new banks a second system of fences is erected and the process is repeated until a deep river channel is formed with natural high banks on both sides. This bank formation is continued gradually from upstream downwards. In the initial season, work is completed over a stretch of 2.4 to 3.2 km and then extended by a mile or so every season. The result is very beneficial. Logging has been increased from 10,584 to 50,640 logs annually, while 20,230ha (50,000 acres) of swamps have been converted into fertile lands with another 24,280 ha (60,000 acres) improved to yield 2.5 tonnes of paddy per hectare per year instead of 0.8 tonnes before improvement, to say nothing of the uninterrupted communications and diminished flood damage. Same type of construction is used for closing breaches and building groynes.

This system of river training is not designed for facilitating flood flow nor to improve navigation. It is more correctly, the systematic reclamation of low lands by guiding the spill of flood flow with stakes and fences to induce uniform deposition. The method can well be applied to reclamation works on heavy sediment carrying rivers.

REFERENCE AND APPLICABLE BIS CODES

- IS 8408:1994 Planning and design of Groynes in alluvial river – Guidelines
- IS 10751: 1994 Planning and design of Guide banks for alluvial river – Guidelines
- IS 14262 : 1996 Planning and design of Revetment – Guidelines

Survey and investigation for Barrage and Canal Network

Chapter-I Barrage/Weirs

1.1 Topographical Data :

1. An Index Map showing important features in the vicinity of the site.

2. A contour plan of the area (on a scale of 1:4000) around the proposed site of the barrage with contour intervals of not more than 0.5m up to an elevation of about 2.5m above HFL. The contour plan shall extend to about 5 Km. on the upstream and downstream of the proposed site and up to an adequate distance on both flanks up to which the effect of backwater is likely to extend. In case of meandering river, the plan should cover atleast 2 meanders on upstream side and one meander on downstream side of the axis.

3. Cross Sections of the river at the proposed site at intervals of 200m both on upstream and downstream up to at least 600m from the proposed site. Besides this, Cross sections may be taken at 2 Km interval up to the distance; the backwater effect of ponding is likely to extend on the upstream of the site. The cross levels in the river bed may be spaced at 10 to 30 m depending on topography of the river. The cross section should extend on both banks to about 2.5m above HFL.

4. Longitudinal Section of the river up to 5 km on the downstream side and 15 Km on the upstream side or the distance up to which back water effect is likely to extend on the upstream side,

1.2 Geotechnical Data:

1. Plan showing location of bore holes, also Log charts of initial bore holes drilled 100 m apart along the axis of the barrage and under the divide wall and abutments to a depth of 20m below the deepest river bed level plotted on a cross section of the river showing the corresponding 'N' i.e. S.P.T. Values covering foundation of all the major components of barrage. The grain size distribution analysis, moisture content, voids ratio, in-situ density, submerged density, coefficient of permeability etc. may also be determined and furnished.

2. Shear parameters (C-Φ values) of the foundation and backfilled material.

3. Modulus of sub-grade reaction at the proposed foundation level of the barrage. This value to be obtained by conducting in-situ tests conforming to I.S. 1888-1982 - Method of Load Test on Soils.

4. If clayey strata is met, undisturbed sample of clay layers from the proposed foundation level or up to 8m or more depth may be taken. These samples may be

analyzed for unconfined compressive strength, swelling index, consolidation characteristics and other parameters of soil as stated above and results furnished.

5. Safe Bearing capacity of the soil at the proposed foundation level of abutment, barrage floor in under-sluice and spillway bays, divide walls etc. by actual load tests.

6. Values of Design Seismic Co-efficient to be adopted, in case the barrage is situated in earthquake zone III and above.

7. Permeability coefficient of foundation material as well as material on banks.

8. Water table data in monsoon and spring for past few years.

9. If loose sand is met, liquefaction potential studies may be undertaken.

10. Salient features of other WR Structure in the immediate vicinity of the barrage.

1.3 Hydrological Data:

1 Flood discharge corresponding to 50, 100 and 500 year frequency and corresponding water levels as well as minimum water level in the river. HFL observed during the last 50 years at barrage locations (as obtained from local enquiry) may also be indicated.

2 G-D curve at the barrage site up to HFL.

3 Salient levels for the barrage i.e. the pond level, MDDL, if any, etc.

4 Sediment load during high and low flood stages.

- Any data already provided with the Prefeasibility note may be confirmed to be used for the purpose of DPR preparation.
- All the data especially the drawings may be provided in the soft copy as well.
- Any photographic or video graphic details of barrage site, if available, may also be provided.

Chapter -II CANAL SYSTEM

2 Preliminary Planning and Layout of Canal System for Irrigation

2.1 Planning

After deciding the head discharge of canal, the area to be irrigated by canal system shall be worked out. This shall be done by preparing land use maps, preferably on a scale of 1:15, 000, showing on them area already under cultivation, soil types, habitations, roads, drainages, and contours of the area. The intensity of irrigation to be provided on the project shall be decided after taking into account the socio-economic factors for the area and intensity of irrigation being achieved on other projects in the neighborhood. The important crops of the area and their water requirements shall be determined in consultation with the department of agriculture and the agriculturists of the area proposed to be served, allowing for the anticipated change in crop pattern due to introduction of wet farming in the area. Knowing thus the duty for various crops, the area under cultivation, area under various crops, the intensity of irrigation, the culturable area to be commanded shall be worked out and marked on the map. Areas that are higher and may not be supplied with flow irrigation should also be marked on the map and excluded from the culturable commanded area.

2.2 Capacities

Capacity of a canal system shall be fixed on the basis of the following considerations:

a) Culturable commanded area,
b) Water allowance and
c) Transmission losses.

A capacity statement in the sample tabular form as shown in ***Annexure- A*** shall be prepared to determine the capacity of the canal at control points i.e. off taking points of all branch canals, distributaries, minors, falls regulators, control structure, CD/CM works.

2.3 Detailed Investigation Stage

2.3.1 Topographical survey

1. Index plan on a scale of 1:15,000 showing the head works and canal alignment.

2. Survey maps shall be prepared or produced preferably to a scale of 1:15000 showing the contours (with contour interval of 0.5 m or less), spot levels and important land features for the whole area to be developed.

3. Alignments of main canals, branches and distributaries shall be tentatively marked on the map. The main canal should be generally carried on a contour alignment, Branch canals and distributaries should take off from main canal from or near the points where the main canal crosses watershed. While selecting the alignment, consideration of economy shall be born in mind. Deep cuttings or high embankments should be generally avoided by suitable detouring, after comparing

the overall costs of the alternative alignments. Carrying of a canal in high embankment involves risk of breaches.

4. Strip contour plan covering 250m on either side of centre line of the canal alignment or as per site requirement whichever is more. At the cross drainage works, the strip contour plan should cover a distance of 500 m along the flow of drain / river on either side of the centre line of the canal. The contour plan shall be plotted at 0.5m contour interval with 1:10000 or 1:20000 H-scale. Levels shall be taken at 50m or less interval along the C/L of Canal.

5. Supporting calculations for arriving at canal discharge considering seepage & transmission losses may be furnished. If there is any provision for drinking and industrial water supply, along the line (canal alignment) the same may be added duly considering losses.

6. Catchment Area of drains/rivers and the assessed Max Flood Discharge shall be indicated at the cross drainage works.

2.3.2 Soil Survey

1. Shearing properties i.e. cohesion and angle of internal friction of bed and bank material. In places where the canal runs in filling, if depth of filling is more than 6m, the borrow area soil characteristics shall also be provided.

2. In situ voids ration, moisture content and density of the bank and bed material.

3. Seepage characteristics or permeability of bed and bank material.

4. Ground water table, both in dry and monsoon condition along the alignment.

5. Grain size distribution of the bed and bank material.

6. SPT values (in case of cohesion less soils) / unconfined compression test values (in case of cohesive soil) and Safe bearing capacity of the foundation strata under various canal regulation / cross drainage / cross masonry works.

7. If the canal runs through expansive soils, the swelling index tests may be provided The soil report may be submitted in the sample tabular form as given in **Annexure-B.**

2.3.3 ***Location for sub-soil exploration*** : exploratory holes and pits should follow the central line of the proposed canal alignment and also the axis of the cross drainage structures.

2.3.4 ***Spacing for sub-soil exploration*****:** For canals, exploration by pits at a spacing of 500 m depending upon the nature of the soil may be carried out. However, where there is an apparent change in characteristics, the pits may be dug at 200 to 300m or even lesser spacing. Under the major cross drainage structures the holes or pits should be selected at specific points, where the proposed piers, wells and abutments are to rest.

2.3.5 ***Depth of exploration***: In case of canals, exploration should reach at least 3m below the proposed bed of the canal. If the canal is passing through rocky reach this depth may be reduced. If the strata appear to be changing, the depth of exploration below the bed level should go up to an extent of canal depth.

In case of CD structures the depth of the holes or pits should be up to the bottom of the expected bulb of pressure under the abutments or the piers or down to the hard firm strata or rock, if available. In case of well foundation the depth of exploration should reach the stable strata, which can support the foundation.

3.0 Cross Drainage Structures

For Cross Drainage structures following data is needed:

i) Hydraulic Particulars of canal & drain /river in the sample tabular form as per ***Annexure-C***.
ii) Site plan with flow direction of canal & drain / river with levels @10 m interval & contours at 0.5 m interval.
iii) Longitudinal section of drain / river covering 500 metres on U/S & D/S of canal crossing with levels @ 10m to 20m interval.
iv) Cross section of drain / river at canal crossing and at 100 m interval on U/S, D/S of the crossing for a length of 500 m. The Cross levels shall be taken at 3m to 5m interval in the gorge portion and 10 m intervals on the flanks extending up to Max Flood Level.
v) The catchment area shall be marked on the Topo Sheet for all the catchment areas more than 2.5 Sq. Km. If the catchment area is less than 2.5 Sq.km. the area is to be traversed on ground and furnished.
vi) Observed Max Flood Discharge may be computed from the observed flood levels and shown on the longitudinal / cross sections. If observed flood levels are not available, then 25yr, 50yr and 100yr return flood may be provided as stipulated in IS:7784_part 1.
vii) Bore hole data / Trial Pits up to Hard strata or for min. depth of 2m for shallow foundations & up to 1/3rd embedment depth below maximum scour depth, along the Centre Line of canal crossing at suitable intervals depending upon the importance of the structure with minimum 5 Nos. of Trial Pits covering both the Drain & Canal at the crossing and on U/S & D/S sides of the crossing
viii) If any road bridge is combined with the CD structure the details of the road, no. of lanes etc may be provided.
ix) Salient features of existing U/S & D/S structures on river/drain to be crossed.
x) The soil parameters shall be provided as per clause 1.2

4.0 Road bridges (Cross Communication Structures)

i) Report accompanying the Site survey along with Hydraulic Particulars of canal with road details in the sample tabular form as per ***Annexure-D***.

ii) Site plan along with flow direction of canal, road way, angle and direction of skew if any, levels at 10m intervals covering the approaches to a sufficient distance not less than 1/4 km on either side.

iii) Longitudinal section of canal as per approved Hydraulic Particulars and of Road covering 500 metres on U/S & D/S of the canal crossing with levels at 10m intervals.

iv) Important details of Road Bridge and type of road bridge with carriage way width.

v) Bore hole data / Trial Pits up to Hard strata or for min. depth of 2m for shallow foundations & up to 1/3rd embedment depth below maximum scour depth along the Centre Line of the canal crossing at suitable intervals depending upon the importance of the structure with minimum 3 Nos covering canal centre line, right and left abutment. Also the safe bearing capacity of foundation strata is to be furnished.

4.0 Regulators/ Fall / Escapes

i) Report accompanying the Site survey along with Hydraulic Particulars of parent canal and off-take canal in the sample tabular form as per ***Annexure-E&F***.

ii) Longitudinal section of canal as per approved Hydraulic Particulars and of Road if any covering 500 metres on U/S & D/S with levels @ 10m intervals.

iii) Important details of Road Bridge and type of road bridge with carriage way width.

iv) Bore hole data / Trial Pits up to Hard strata or for min. depth of 2m for shallow foundations & up to 1/3rd embedment depth below maximum scour depth along the Centre Line of canal at suitable intervals depending upon the importance of the structure with minimum 3 Nos covering Canal centre line, right and left sides. Also the safe bearing capacity of foundation strata is to be furnished.

References:

Working Group Report on Guidelines for Preparation of Detailed Project Reports of Irrigation and Multipurpose Projects 1980

IS Codes:

1) Barrage

IS: 7720 Criteria for Investigation Planning and Layout of Barrages and Weirs
IS: 6966 Hydraulic design of Barrages and Weir Part 1 Alluvial Reaches
IS: 11130 Criteria for Structural Design of Barrages and weirs
IS: 13578 Code of Practice for subsurface exploration of Barrages and weirs

2) *Canals and related CD works:-*

IS: 11385 Code of practice for subsurface exploration of Canal and CD works;
IS: 5968 Code of Practice for Planning and Layout of canal System
IS: 7784_Part 1 – Design of cross drainage works – Code of practice.

Annexure A

TABLE-1.1

FSL STATEMENT FOR L- SECTION OF UJH LEFT MAIN CANAL

Canal offtake	Structure	Discharge Required	R.D. metres	Ground level (NSL)mts	Command level required at distry. head	Head Loss in structures/ working head (MC) (metres)	Section Length	Slope (1 in X)	Fall due to slope	Proposed FSL		Head Loss in structures/ working head in Dy. (metres)	FSL at Distibutary head	Remarks
										U/S	D/S			
Branch Canal 1								7000					-	
	Cross Drainage 1					0.200		7000						
	Cross Drainage 2					0.200		7000						
Branch Canal 2						0.100		7000						HR
	Cross Drainage 3					0.200		7000						
	Cross Drainage 4					0.200		7000						
	Road crossing					0.000		7000						
Distributary	Cart track crossing							7000						CR & HR
	Railway Line Crossing													
	Canal Crossing													
	Major Pipe line Crossing													

Annexure A

TABLE-1.2

FSL STATEMENT FOR L- SECTION OF Branch Canal-1 off take Metres xxxL Ujh Main Canal

Additional as per SOI

Canal offtake	Structure	R.D. metres	Discharge Required	NSL	Command level required at distry. head	Head Loss in structures/ working head in MC (metres)	Section Length	Slope (1 in X)	Fall due to slope	Proposed FSL		Head Loss in structures/ working head in Dy. (metres)	FSL at Distibutary head	Remarks
Distributary-1								5000		U/S	D/S		-	
	CD							5000						
	CD							5000						
Minor-1.						0.100		5000				0.300		HR
	CD							5000						
Distributary-2.						0.200		5000				0.500		CR & HR
	Road Crossing					0.100		5000						CR/Road Xing
	Road Crossing					0.100		5000						
Distributary-n						0.200		5000				0.300		HR
	Cart track Crossing					0.100		5000						

Total No of Structures:

Cross drainage	x
Kaccha/Pucca Rd.Crossing	y
Head Regulator+Cross regulator	z
Cross Regulator	a
Head Regulator	b
Falls	f
Railway line crossing	r

Soil Testing Laboratory

Project : Construction of (Structure name) **Annexure B**
Soil Testing - Communication of test results of the samples collected at ---

File No -

TABLE I
(IDENTIFICATION TESTS)

Sl No	Lab Ref. of Soil Sample	Soil Reference No given by Department			Atterberg Limits on Soil passing 0.425 mm size mesh				% passing 0.075 mm size	% passing 4.75mm size	Gradation Analysis I.S.I.				I.S.I. Classification	Differential free swell of soil passing 0.425mm size (%)
		B.H. No.	at Chainage or @ KM	Depth	Liquid limit	Plastic limit	Plasticity index	Shrikage limit			Gravel	Sand	Silt	Clay		
	No.			M	%	%	%	%			%	%	%	%		

Soil Testing - Communication of test results of the samples collected at --- **Annexure-B**

File No -

TABLE II
Test Results of Soils (ENGINEERING PROPERTIES)

Sl No	Lab Ref. of Soil Sample	Specific Gravity	FDD gm/cc	FMC of dry weight of soil %	Void ratio	Saturation %	Shear strength on soil passing 4.75 mm size				
							Box Shear Test (quick)		Unconsolidated undrained triaxial quick test without pore pressure measurements		Permeability on soil passing 19.1 mm size x10 cm/sec
							At FDD & SMC		At FDD & SMC		
							C' in KG / Sq. Cm.	Ø in Degrees	C' in KG / Sq. Cm.	Ø in Degrees	

Annexure C

. . . . PROJECT - HYDRAULIC PARTICULARS OF . . . . MAIN CANAL FROM KM. . . . . TO KM.. . . . .

<u>STATEMENT SHOWING THE CROSS DRAINAGE WORKS</u>

S.NO.	**NAME OF**	**CHAINAGE**	**PARTICULARS OF DRAIN**				**PARTICULARS OF PARENT CANAL**					**LOSS OF**	**REMARKS**
	C.D.WORK	**IN K.M**	**BED**	**C.A.**	**M.F.D**	**O.M.F.L**	**BED WIDTH**	**F.S.D**	**B.L**	**F.S.L**	**T.B.L**	**HEAD**	
			LEVEL	**SQ.KMS.**	**CUMECS**		**MTS**					**IN MTS.**	

Annexure D

. . . . PROJECT - HYDRAULIC PARTICULARS OF . . . . MAIN CANAL
FROM KM. . . . . TO KM. . . . .

STATEMENT SHOWING THE CROSS MASONRY WORKS

S.NO.	DESCRIPTION	NAME OF CROSS MASONRY WORK	CHAINAGE IN KM.	EXISTING ROAD LEVEL / ROAD WIDTH	BED WIDTH IN M.	F.S.D. IN M.	HYDRAULIC PARTICULARS OF CANAL						LOSS OF HEAD	REMARKS
							B.L		F.S.L.		T.B.L.			
							U/S	D/S	U/S	D/S	U/S	D/S		

Annexure E

STATEMENT SHOWING THE DETAILS OF REGULATORS

S.NO.	STRUCTURE	CHAINAGE IN KM.	EXISTING ROAD LEVEL	HYDRAULIC PARTICULARS OF CANAL										LOSS OF HEAD IN MTS.	REMARKS
				BED WIDTH		F.S.D		B.L		F.S.L.		T.B.L.			
				U/S	D/S	U/S	D/S	U/S	D/S	U/S	D/S	U/S	D/S		

Annexure F

. . . . PROJECT - HYDRAULIC PARTICULARS OF . . . . MAIN CANAL
FROM KM. . . . . TO KM. . . . .

STATEMENT SHOWING THE DETAILS OF DROPS/Escapes/Regulators

S.NO.	CHAINAGE	DESCRIPTION	HYDRAULIC PARTICULARS OF CANAL										SOILS AT	REMARKS
	IN KM.	OF DROP	BED WIDTH		F.S.D		B.L		F.S.L.		T.B.L.		FOUNDATION	
			U/S	D/S	U/S	D/S	U/S	D/S	U/S	D/S	U/S	D/S	LEVEL	

Survey, Investigations, Planning & Design of Canal System
Regime Theory and Design of Unlined Canals

Vijai Saran
Director
CWC

1.0 INTRODUCTION

Irrigation projects are very important for development of the country. It is therefore necessary that irrigation projects need to be carefully planned and investigated so as to optimize benefits and minimize the undesirable effects. An irrigation project consists of a dam /barrage /weir for storage and diversion of water, a head regulator and a canal system. In case, demand for irrigation in an area is high, then it is desirable that a storage dam is planned to meet the irrigation demand with a greater degree of dependence. However, if suitable site for construction of dam does not exist, then a barrage or weir is to be planned.

The first step in planning an irrigation project is assessment of availability of water. In order to assess water availability, daily discharge data of the river at a nearby location is required. In case, the canal off takes from a reservoir project, it should be designed for a discharge which depends upon the live storage provided in the reservoir for irrigation and in case it off takes from a diversion work (barrage / weir), it should be designed for 75% of the river discharge available as determined by flow duration curves. The flow duration curves should be prepared for crucial months at suitable close intervals. For canals off taking from diversion works, the study of discharge data will determine the type of canal system to be planned and laid out, that is inundation canal, intermittent canal or permanent canal.

The next step is to make a preliminary assessment of water demand for irrigation based on command area, culturable command area, existing cropping pattern, proposed cropping pattern, existing irrigation facilities and rainfall etc. The important crops of the area and their water requirements shall be determined in consultation with the department of agriculture and the agriculturist of the area proposed to be served allowing for the anticipated change in crop pattern due to introduction of irrigation in the area.

Based on desktop studies and preliminary investigations, decision whether a dam or barrage / weir is to be investigated is taken and tentative alignments for dam / barrage / weir are marked for detailed investigation. Further the proposed command to be served is also identified for detailed command area survey and marking the layout of the canal system.

2.0 SURVEY AND INVESTIGATION FOR CANAL SYSTEM AND CROSS DRAINAGE WORKS:

The planning and layout of a canal system is controlled by the area to be irrigated and the source of supply. Hence, first of all the daily flow data of the river at the head works should be obtained. As already stated, if the canal takes off from a reservoir project, it should be designed for a discharge depending on the live storage provided in the reservoir for irrigation and in case it takes off from a diversion work, it should be designed for 75% of the river discharge. Then, the area to be irrigated by canal system shall be worked out by carrying out command area survey and preparing command area survey maps in a scale of 1:10000 or 1:15000 and contour interval of 0.5m. The command area survey maps will show the contours of the area in a contour interval of 0.5 m, area already under cultivation, soil types, roads, natural drainage, spot levels and important land features for the whole area to be developed.

The intensity of irrigation to be provided on the project shall be decided after taking into account the socio economic factors for the area and intensity of the irrigation being achieved on the other projects in the neighborhood. The important crops of the area and their water requirements shall be determined in consultation with the department of agriculture and the agriculturist of the area proposed to be served allowing for the anticipated change in crop pattern due to introduction of irrigation in the area. Knowing, thus the duty for various crops, the area under cultivation under different crops, the intensity of irrigation, the cultuarble area to be commanded shall be worked out and marked on the map. Areas that are higher and can not be supplied water by canal should be marked on the map and excluded from the culturable command area.

The layout of main canals and branches is marked on the command area maps on the consideration of economy. For the layout of minors and distributaries, points of off take may be suitably selected but their layout is more or less governed by the blocks of areas to be irrigated taking into consideration watersheds and drainages. The main canals and branches are feeder channel for distributaries and generally no irrigation is done directly from them. Irrigation outlets are provided on distributaries or minors off taking from distributaries.

2.1 Data required

The following data is required for planning the layout of a canal system:

a) Topographical map of the area,
b) Subsurface data,
c) Texture and salt composition of the soil,
d) Soil characteristics including shear parameters,
e) Permeability of the soil in relation to the seepage loss,
f) Rainfall data,
g) Water availability,
h) Subsoil water level in the area and quality of the underground water,
i) Possibility of water logging and salination,
j) Availability of suitable construction material,
k) Existing irrigation and drainage facilities,
l) Existing crop patterns,
m) Existing communication and transportation facilities, and
n) Socio economic study and agro-economic survey of the project area.

For collecting above data different surveys and investigations, as explained below, shall be carried out.

2.1(A) Surveys:

a) **Command Area Survey:** Contour Plan of the area in a horizontal scale of 1:10000 or 1:15000 and contour interval of 0.5 m. This map is to be used for estimating crop water requirement, fixing canal alignment etc.
b) **Longitudinal Survey of Canal:** Longitudinal survey along the main canal is to be carried out on a horizontal scale of 1: 2000 and 1: 100 on a vertical scale. Leveling is to be carried out at 50 m or less interval.
c) **Cross-sections Survey of Canal:** Cross sections of canal should be taken at an interval of about 50 m on a horizontal scale of 1: 2000 and 1 : 100 on a vertical scale
d) **Strip Contour Plan:** It should cover about 150 m on either side of the centre line of the canal at a horizontal scale of 1:1000 and contour interval of 0.5 m.
e) **Survey for cross drainage structures:** Grid Plan with contours of the site to cover an area upto 300 m on either side of the centre line of the canal in a scale of 1:2000 and contour interval of 0.5 m or upto 100m downstream of the point of exit of water and 100 m upstream of the point of water inlet.

Cross-section of drain / nallah along the centre line of the canal in a scale of 1:2000 is prepared. Bed level/bank level and FSL of the canal and maximum HFL of drain is to be indicated.

Survey of the drains both upstream and downstream of canal for adequate length is to be taken. The plan should be made in a scale of 1:10000 and the longitudinal and cross-sections of drain / nallah in a scale of 1:2000 should be prepared.

A list of all canal structures, its location and other necessary details like HFL of the drain, bed level / bank level of the drain / nallah etc. is to be prepared.

f) **Soil Surveys:** Plan in a scale of 1:10000 or 1:15000. Soil survey is necessary for irrigation planning to avoid problems of waterlogging, salinity etc. Detailed soil survey and irrigability classification maps should be prepared.

2.1 (B) Geotechnical Investigations

a) **Canal System**: Adequate investigation should be carried out to collect the data by digging trial pits and bore holes, where necessary, to ascertain the nature of soil encountered along different alternative alignments. Brief description of the overburden shall be given classifying clay, silt, sand and depth of water table. Soil characteristics including mechanical properties and shear parameters alongwith permeability shall be ascertained. For general guidance regarding the suitability of soils for use in canal embankment a reference may be made to IS 1498-1970 "Classification and identification of soils for general engineering purposes" and IS 4701 – 1982 "Code of practice for earthwork on canals"

b) **Canal Structures:** Test pit / borehole, as required, at each canal structure site is necessary to ascertain the nature of the foundation.

2.1 (C) Construction Material Surveys

- Qualitative and quantitative assessment of availability of construction material for lining, if required.
- Identification of borrow pits for obtaining earth for filling portion of canal.
- Testing of properties of the earth to be used for filling.

2.2 Data for Irrigation Planning

The purpose of irrigation planning is to estimate the crop water requirement and finalise the design discharge of the canal. In order to achieve this satisfactorily, proper field data is required in the prescribed proforma.

The field data required are

a) Existing / proposed irrigation facilities in the proposed project command area.

b) Existing cropping pattern
1. Existing area under rain fed cultivation
2. Area under each crop
3. Net increase in irrigation facilities due to project.

c) Soil surveys

1. Agro-climatic conditions
2. Rainfall
3. Temperature
4. Humidity
5. Sunshine
6. Wind velocity
7. Evaporation
8. Cloud Cover
9. Frost free days

d) Proposed cropping pattern

2.3 Fixation of Capacity

Capacity of a canal system shall be fixed on the basis of the following considerations :

(a) Culturable command area
(b) Water allowance (The outlet capacity in cumec per thousand hectares of culturable command area)
(c) Transmission losses (generally for lined canals a figure of 2 cusecs per million square feet and 8 cusecs per million square feet for unlined canals in Gangatic alluvial plains is taken)

A capacity statement in the prescribed proforma (Annexure B) is to be prepared to determine the capacity of the canal at control points.

2.4 Decision regarding lining of Canal

The subsoil water level in the area, soil characteristics, drainage condition of the area may be collected in order to assess possibility of water logging and salination so that decision regarding lining can be taken.

The decision to line the canal should be taken at the time of planning any project. In the early stages of a river development, water may be plentiful, and the immediate need for saving percolation losses not so urgent. But it must be borne in mind that it is very difficult to provide lining later on. Once an unlined canal has been designed and constructed, it comes in a regime. If the decision to line is taken afterwards, the unlined section is much greater than that required for a lined canal. If the lining is carried out on the same section then it will be difficult to maintain the required water levels (FSLs) for the same discharge. It is therefore recommended that the desirability of lining a canal should be carefully studied, when an irrigation scheme is formulated.

2.5 Alignment of canals

(i) Watershed Canal or Ridge Canal
(ii) Contour Canal
(iii) Side-slope Canal

(i) Watershed canal or ridge canal

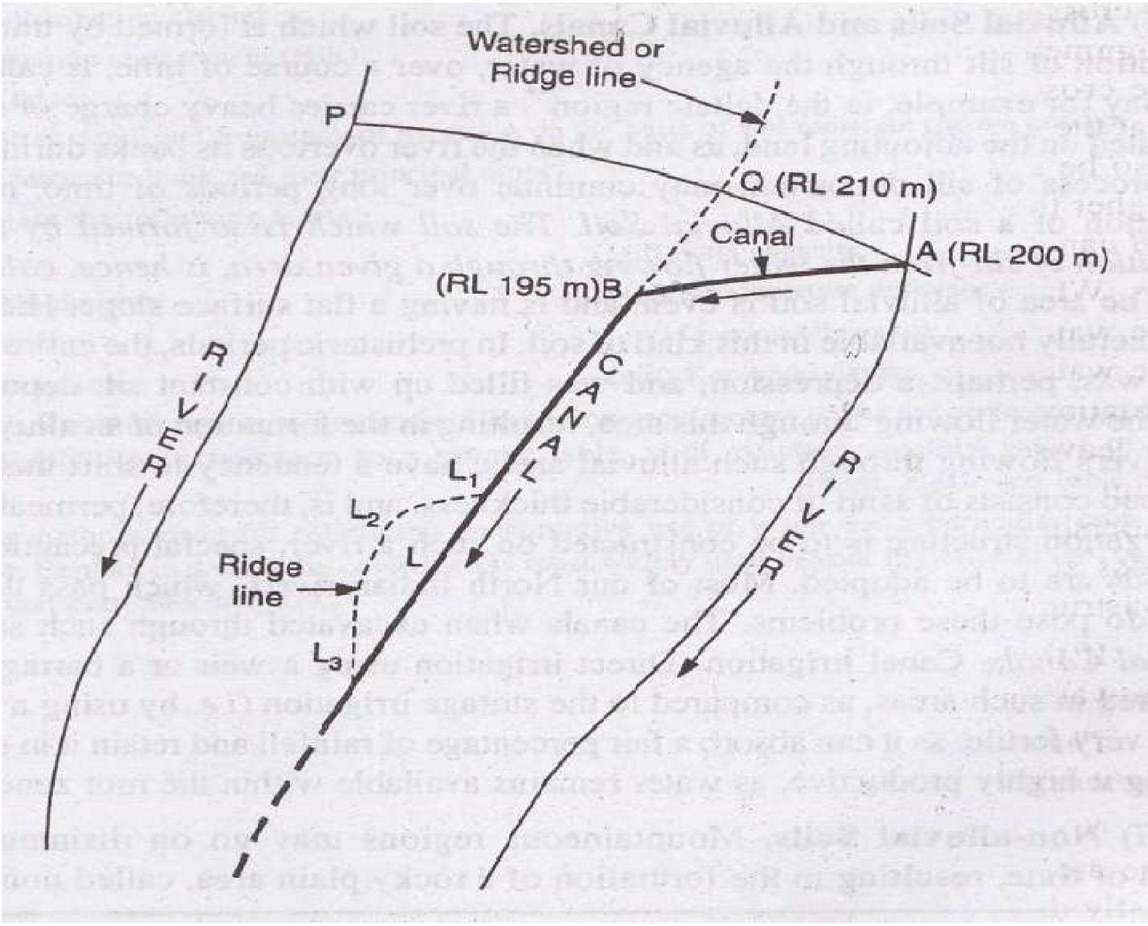


Fig: Alignment of a ridge or watershed canal
(Head reach of main canal in plains)

- The dividing ridge line between the catchment areas of two streams (drains) is called the watershed or ridge canal.
- Thus between two major streams, there is the main watershed (ridge line), which divides the drainage area of the two streams. Similarly, between a main stream and any of its tributary, there are subsidiary watersheds (ridge lines), dividing the drainage between the two streams on either side.
- The canal which is aligned along any natural watershed (ridge line) is called a watershed canal, or a ridge canal. Aligning a canal (main canal or branch canal or distributary) on the ridge ensures gravity irrigation on both sides of the canal.
- Since the drainage flows away from the ridge, no drainage can cross a canal aligned on the ridge. Thus, a canal aligned on the watershed saves the cost of construction of cross-drainage works.

(ii) Contour Canals

Watershed canal along the ridge line are, however, not found economical in hill areas, since the conditions in hills are vastly different compared to those of plains. In hills, the river flows in the valley well below the watershed. In fact, the ridge line (watershed) may be hundreds of meters above the river. It therefore becomes virtually impossible to take the canal on top of such a higher ridge line. In such conditions, contour canals are usually constructed.

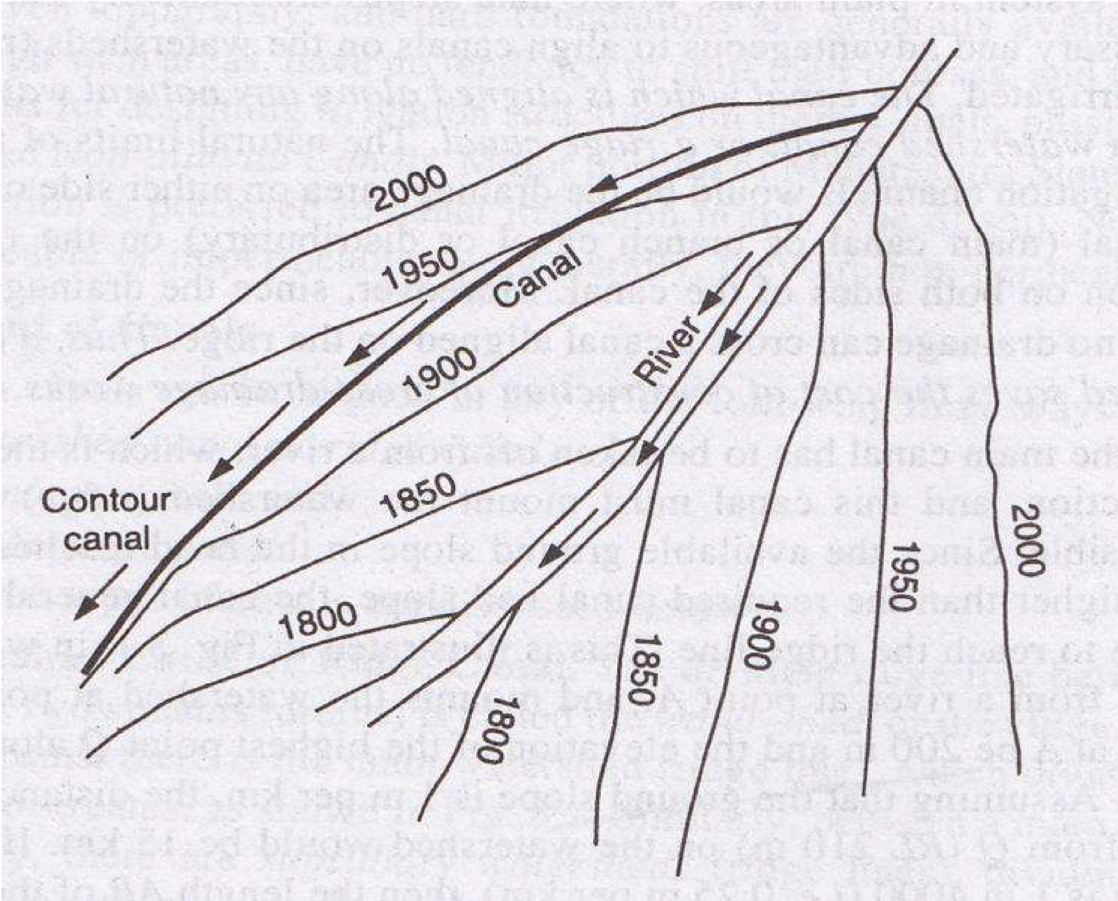


Fig: Alignment of a contour canal
(Head reach of main canal in hills)

Contour channels follow a contour, except for giving the required longitudinal slope to the canal. Since the river slope is much steeper than the canal bed slope, the canal encompasses more and more area between itself and the river. A contour canal irrigates only on one side because the area on the other side is higher.

(iii) Side-slope canal

Side slope canal is that which is aligned at right angles to the contours; i.e. along the side slopes, as shown in figure. Since such a canal runs parallel to the natural drainage flow, it usually does not intercept drainage channels, thus avoiding the construction of cross-drainage structures.

It is a canal which is aligned roughly at right angle to contours of the country but not on watershed or valley. The canal thus runs roughly parallel to the natural drainage of the country and as such cross drainage works are avoided. The side slope channel has the advantage of not intercepting cross drainage works but its course must follow the shortest route the nearest valley and such channel shall be along a line of steepest possible slope except in very flat areas.

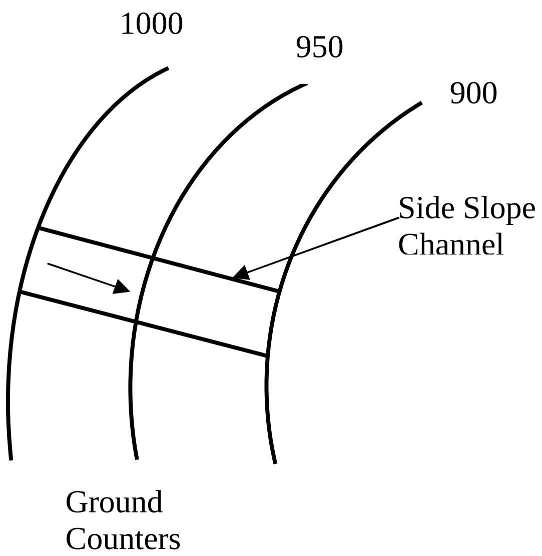


Fig: Alignment of a Side Slope Canal

2.5(A) Procedure of Alignment of Canal

Command area maps shall be prepared preferably to a scale of 1:10000 or 1:15000 showing the contours, spot levels and important land features for the whole area to be developed. Alignment of the main canals, branches and distributaries shall be tentatively marked on the map. A typical canal system may be generally represented as a main canal aligned as a contour canal and branches and distributaries aligned as watersheds or side slope canals.

The main canal should be generally carried out on a contour alignment until it attains the top of a watershed. From such a point, it should be aligned down to the watershed ceasing to be a contour canal. After reaching the watershed the main canal should be located along the main watershed and branch canals along secondary watersheds since it is generally observed that the slope of main watersheds is less than slope of secondary watersheds and the branches are required for irrigating the area upto the adjacent drainages on either side of the watershed crest.

The alignment of contour canals, especially in the upper reaches shall be decided after careful consideration of economy. Alternative alignments, their benefits and costs shall be compared. Contour canals may irrigate only on one side and they themselves from the upper boundaries of the irrigated area and cut across the natural drainage lines of the country. Watershed and side slope canals have the advantage of non interception of any cross drainage, but since they lie along the line of steepest possible slope, except in a very flay area, only the smaller of distributary canals may be so located.

While selecting the alignment, consideration of economy shall be borne in mind. Deep cuttings or high embankments should be generally avoided by suitable detouring, after comparing the overall costs of the alternative alignments. Carrying of a canal in high embankment involves the risk of branches from percolation. Careful judgment shall be exercised in fixing the points of crossing of drainage.

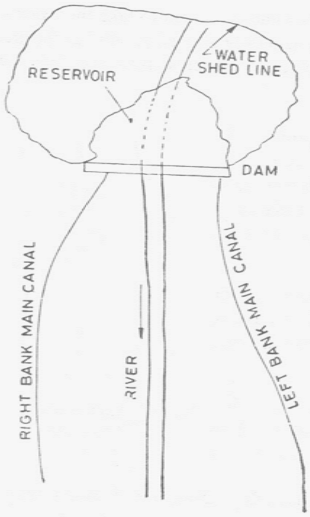


Fig. 3.4. Storage Irrigation Scheme.

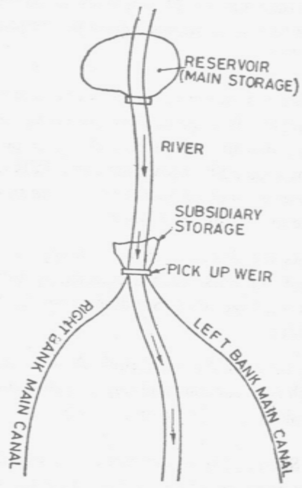


Fig. 3.5. Storage Irrigation Scheme with Pick up weir.

2.5 (B) Curves

The alignments of the canals shall consist of straight lines with circular curves. Radii of the curves should be usually 3 to 7 times the water surface width subject to minimum given below:

Radii of curves for canals			
Unlined Canals		Lined Canals	
Discharge (cumecs)	Radius Minimum (m)	Discharge (cumecs)	Radius Minimum (m)
80 and above	1500	280 and above	900
Less than 80 to 30	1000	Less than 280 to 200	750
Less than 30 to 15	600	Less than 200 to 140	600
Less than 15 to 3	300	Less than 140 to 70	450
Less than 3 to 0.3	150	Less than 70 to 40	300
Less than 0.3	90	Less than 40 to 10	200
		Less than 10 to 3	150
		Less than 3 to 0.3	100
		Less than 0.3	50
1. The above radii are not applicable to unlined canals located in hilly reaches and highly permeable soils			
2. On lined canals where the above radii may not be provided proper superelevation shall be provided			

2.5 (C) Spacing

Distributaries may be spaced suitably depending upon the configuration of the area.

2.5 (D) Crossing

It is desirable that alignment of a canal crosses least number of drainages.

2.6 Surface Area

After the alignment of canals and drains is drawn in plan, areas served by various canals shall be calculated. The area depending on a branch/ distributary canal shall generally be limited by the nearest drain.

2.7 Evaluation of Full Supply Levels of Canal

After the tentative alignments of the canal system are marked, full supply levels shall be decided beginning with outlets, minors, distributaries, branch canal and then obtaining the full supply levels in main canal. This may be done by drawing longitudinal sections of the main canals and its distributary canals. Longitudinal scale of about 1:10000 to 1:20000 and vertical scale of about 1:100 is recommended. The following information shall be added below the datum line on the longitudinal sections

a) Natural surface level
b) Full Supply Level
c) Bed Level
d) Subsoil Water levels
e) Water surface Slope
f) Bed widths; Value of 'N' side slope, F.S.D, F.S.Q. velocity
g) Free Board
h) Broad details of hydraulic data of outlets, regulators, bridges, drainage crossing, offtaking channels etc.

Other Data to be incorporated in the longitudinal section

a) Test pit / auger bore hole data at every 500 m
b) Location and data of offtaking channels / outlets
c) Location and type of C.D. works along with hydraulic data of the drain namely catchment area, H.F.L. Design discharge and foundation data. Loss of head shall be indicated
d) Location of railway crossing with rail levels
e) Location of other structures such as road bridges, cross regulators, escapes, falls etc.

2.8 Section and Slope

The following principals should be kept in view for designing a canal system

a) The cross sectional area of a canal should generally not increase from upstream to downstream
b) Cross section area of lined canals shall be designed in accordance with IS 10430-2000 "Criteria for design of lined canals and guidelines for selection of type of lining" and IS 7112-1973 "Criteria for design of cross section for unlined canals in alluvial soil".
c) Balancing depth shall be adopted wherever possible

2.9 Cross drainage structures

In working out the longitudinal section, the provision of regulators, falls, escapes, cross drainage works etc shall be considered. The losses shall be fully accounted for. A list of all cross drainage structures, its location and other necessary details like HFL of the drain, bed level/ bank level of the drain / nallah, FSL of canal at that location etc. is to be prepared.

3.0 REGIME THEORY

Whenever water flows through a channel (artificial or natural), it tries to scour its surface. Silt or gravel or even larger boulders are detached from the channel bed or banks. These particles are swept downstream by the flowing water. This phenomenon is called sediment transport.

It is desirable that a canal is non silting non scouring i.e. it should neither erode its channel nor silt it up. Then it is in true regime. Various theories have been proposed for the design of such channels. The theory mostly used in India for the design of canal cross section in alluvial soils is Lacey's regime theory.

According to Lacey, a canal is said to have attained regime condition when a balance between silting and scouring and dynamic equilibrium in the forces generating and maintaining the canal cross section and gradient are obtained. If a canal runs indefinitely with constant discharge and sediment charge rate, it will attain a definite stable section having a definite slope. If a canal is designed with a section too small for the given discharge and its slope is kept steeper than required, scour will occur till final regime is achieved. On the other hand if the section is too large for the discharge and the slope is flatter than required, silting will occur till the true regime is achieved. In practice true regime conditions do not develop because of variation in discharge and sediment rates.

On the analysis of data from a large number of natural drainage and canals running for long, Lacey developed relations for determining regime slope and channel dimensions. He postulated, firstly that the required slope and channel dimensions are dependent on the characteristics of the boundary materials which he quantified in terms of silt factor (f) defined as:

$$f = \frac{2.39v^2}{R} \qquad \text{----} \qquad (1)$$

or

$$f = 1.76\sqrt{D_{50}} \qquad \text{----} \qquad (2)$$

Where

v	=	the mean velocity of flow in m/s
R	=	the hydraulic mean depth of an existing stable channel
D_{50}	=	the average particle size of the boundary material in mm.

Thus, in case, the conditions on a canal to be designed are similar to those on an existing stable canal, the value of silt factor (f) may be determined by the use of formula (1) using the observed value of v and R on the existing stable canal. Alternatively, the value of (f) may be determined by use of formula (2) after determining the D_{50} size of the boundary material.

Having determined the value of (f) the following three relationships may be used for determining the required slope and canal dimensions:

$$S = \frac{0.0003 f^{\frac{5}{3}}}{Q^{\frac{1}{6}}}$$

$$P = 4.75\sqrt{Q}$$

$$R = 0.47\left[\frac{Q}{f}\right]^{\frac{1}{3}}$$

$$V = \left[\frac{Qf^2}{140}\right]^{\frac{1}{6}}$$

Where

S	=	Slope of the canal
Q	=	discharge in m^3/s
P	=	wetted perimeter of the section in m
R	=	hydraulic mean depth in m

Having known the desirable values of P, R the curves given in fig 2 may be used for determining the corresponding canal bed width (B) and Depth (D) for a canal having internal side slopes of ½: 1 (it is assumed that the canal attains a slope of ½ : 1 after running in regime).

Alternatively cross section can be designed from Regime type fitted equations evolved on the basis of data collected from various states in India or by modified Lacey's equation as given in IS 7112 -: 2002 " CRITERIA FOR DESIGN OF CROSS SECTION FOR LINED CANALS IN ALLUVIAL SOIL".

Problem

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

Solution:

$Q = 50 m^3/s$ And Silt factor (f)= 1.1

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

$V = 0.896$ m/s

$A = Q/V = 50/0.896 = 56.3\ m^2$

$R = (5/2)(V^2/f) = (5/2)(0.896^2/1.1) = 1.675$m

$P = 4.75 (Q)^{1/2} = 4.75 (50)^{1/2} = 33.56$m

For a trapezoidal channel with 0.5H : 1V slopes. Therefore, Perimeter of channel

$P = b + (5)^{1/2} y$ and Area of trapezoidal channel

$A = (b + y/2) y$

Where,

b = width of channel

y = depth of channel

Therefore,

$33.56 = b + (5)^{1/2} y$ ------------(1)

$56.30 = (b + y/2) y$ ------------(2)

Solving the equations 1 & 2, we get

$b = 33.56 - 2.24 y$

Put this value of b in equation (2)

$$56.30 = \{33.56-2.24y\}y+(y^2/2)$$
$$56.30 = 33.56 - 1.74\ y^2$$

Solve the above equation for the value of (y), we get

Depth $y = 1.85m$
Width $b = 29.77m$

Slope,

$$S = f^{5/3} / 3340\ Q^{1/6} = (1.1)^{5/3} / 3340\ (50)^{1/6}$$
$$S = 5420$$

Use a bed slope of 1 in 5420.

3.1 DESIGN OF CROSS SECTION

3.1.1 Data Required

The following data shall be collected for design of canal section:

- Topographical map of area to the scale of 1:10000 showing alignment of canal communication lines (roads, railways etc) and other features. A contour interval of 2 m in hilly area and 0.3 m in plains may be adopted in the preparation of this map.
- Longitudinal section of the ground along the proposed alignment to a horizontal scale of 1:10000 and vertical scale of 1:100, showing the upstream water level at the point of offtake, bed slope, Lacey's silt factor for manning's Rugosity Coefficient n, side slopes assumed, velocity and depth, the discharge for which the canal is to be designed in various reaches, sub soil characteristics at every 5 km and also wherever marked changed are noticed, premonsoon and post monsoon ground water levels, position of crossing (roads, railways, drainages etc.) and positions of curves
- Cross section of the ground at every km
- Transmission losses
- Having determined the canal capacity in various reaches as outlined before, the section required to carry the design discharge shall be worked out. A trapezoidal section is recommended for the canal. From the longitudinal section of the ground along the proposed alignment, the average slope of the ground shall be determined. This shall be maximum average slope that can be provided on the canal.

3.1.2 Side Slope

This shall depend on the local soil characteristics and shall be designed to withstand the following conditions during the operation of the canal

1) The sudden drawdown condition for the inner slope
2) The canal running full with banks saturated due to rainfall

Canal in filling will generally have side slopes of 1.5:1, for canals in cutting the side slopes can be between 1:1 to 1.5:1 depending upon the type of soil.

3.1.3 Freeboard

Freeboard in the canal is governed by the consideration of canal size and location, rain water inflow, water surface fluctuation caused by the regulators, wind action, soil characteristics, hydraulic gradient, service road requirement and availability of excavated material. A minimum freeboard of 0.5 m for discharge less than 10 cumecs and 0.75 m for discharge greater than 10 cumecs is recommended

For the design of other cross section elements for example Bank top width, radii of curvature, Berms, dowels , fall etc, guidelines given in IS 7112 -: 2002 " CRITERIA FOR DESIGN OF CROSS SECTION FOR LINED CANALS IN ALLUVIAL SOIL" may be followed.

A typical section of canal is given below as figure -1:

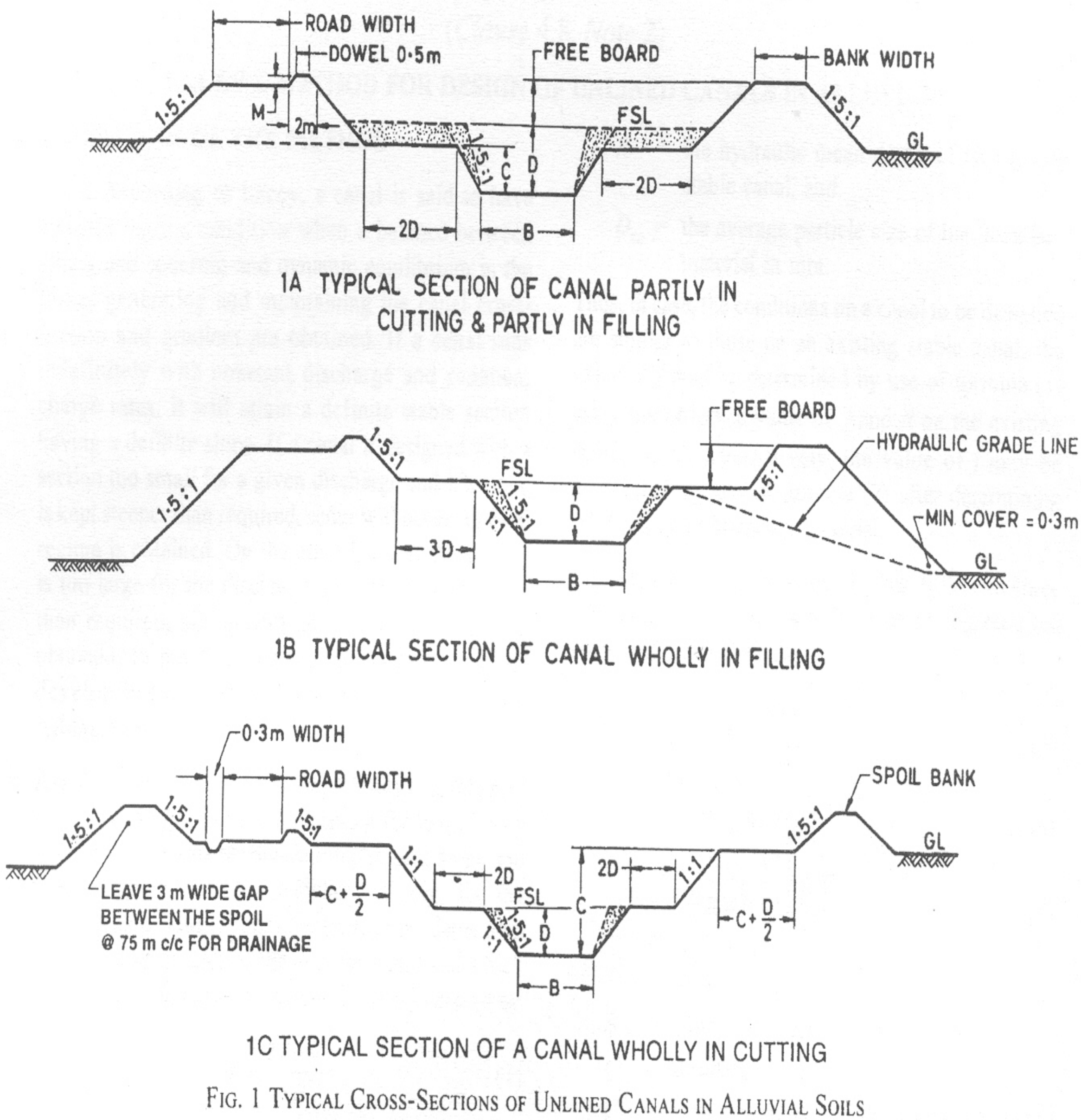


FIG. 1 TYPICAL CROSS-SECTIONS OF UNLINED CANALS IN ALLUVIAL SOILS

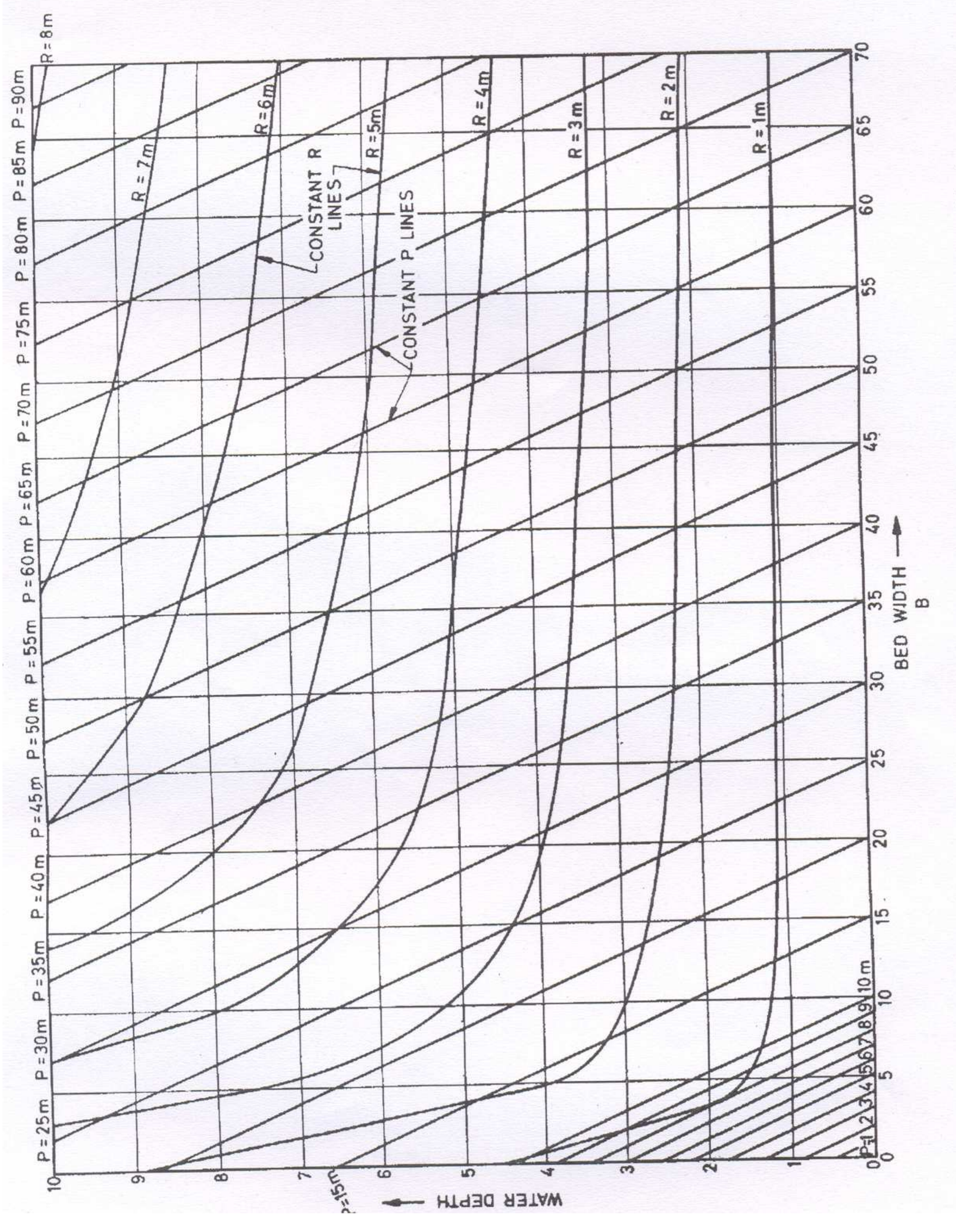


FIG. 2 HYDRAULIC CHART OF RELATIONSHIP BETWEEN B, D, R AND P FOR A CHANNEL HAVING INSIDE SLOPE ½ : 1

Terminology

1) Branch Canal

A canal receiving its supply from the main canal and acting as feeder for distributaries. It is also called lateral.

2) Contour Canal

Canal conforming generally to the contours of the country traversed having, however, such slopes as necessary to produce required velocity of flow.

3) Crop Ratio

The crop ratio is defined as the ratio of areas under different crops to be irrigated during a year.

4) Culturable commanded Area

The gross area commanded minus the area of unculturable land included in the gross area.

5) Distributary

A channel receiving its supply from the main canal or branch canal. It supplies water to the minors and water courses. It is called major distributary when it supplies water to another distributary called a minor distributary.

6) Duty or Duty of Water

The relation between the area irrigated, or to be irrigated, and the quantity of water used, or required, to irrigate it for the purpose of maturing its crop. Duty is stated in reference to base period and the place of its reckoning or measurement.

7) Delta of Water (Δ)

Total depth of water (usually in cm) required by a crop to come to its maturity is called its delta

8) Intensity of Irrigation

The percentage of culturable commanded area proposed to be irrigated annually.

9) Inundation Canal

A canal taking off from a river in flood without a permanent diversion work.

9) Main Canal

The principal channel of a canal system offtaking from a river or a reservoir or tail reach of a feeder. Also called “Main Line”.

10) Permanent Canal

A canal having a regular channel and masonry works for regulation and distribution and with an assured source of supply.

11) Side Slope canal

Canal aligned at right angles to the contour but not on a watershed or valley.

12) Water Allowance

The outlet capacity in m^3/s authorized per thousand hectares of culturable commanded area.

The water allowance therefore is the outcome of all consideration of duty of water, intensity, proposed crop ratio, water availability etc. and not only defines the size of outlet for each outlet area but also form the basis for the design of the distributing canals in successive stages.

13) Watershed Canal

Canal along any natural watershed is called Watershed Canal.

Basic Relationship

1) $\Delta = 864B/D$ cm

Where

Δ = Delta of crop is in cm

B= Base period is in days

D = duty in hectare / cumecs

ANNEXURE – B

TYPICAL PROFORMA FOR CAPACITY STAEMENT OF A CANAL

(1)	S. no.
(2)	Reach from
(3)	Reach to
(4)	Length of reach , m (3)-(2)
(5)	Name of offtake
(6)	Reduced distance of offtake and side (L or R), m
(7)	Gross Commanded Area (ha)
(8)	Culturable commanded area (C.G..A.) (ha)
(9)	Basic discharge per ha of C.C.A.
(10)	Discharge at offtake head m^3/s (8)x(9)
(11)	Discharge in the reach m^3/s ∑(10)
(12)	Wetted Perimeter m
(13)	Wetted area m^2
(14)	Rate of transmission losses* m^3/s /10^6 m^2
(15)	Transmission Losses (13) x (14)
(16)	Total Discharge in the reach m^3/s (11) + (15)
(17)	Discharge at head of the reach m^3/s (Discharge at the tail of the reach + (16))
(18)	Sloe of the reach m/1000 m
(19)	Vale of "n" (Rugosity coefficient)
(20)	Bed width m
(21)	Full supply depth m
(22)	V (Velocity) m/s
(23)	Calculated Discharge m^3/s
(24)	Remarks

* Transmission Losses for lined canal shall be assumed as given in IS 10430 2000 "Criteria for design of lined canals and guidelines for selection of type of lining"

ANNEXURE – C

REFERENCE AND APPLICABLE BIS CODES

- IS 4410:1967 (Part I) Glossary of terms relating to river valley projects: Part I Irrigation Practices
- IS 4410:1968 (Part V) Glossary of terms relating to river valley projects: Part 5 Canals
- IS 5968 : 1987 Guide for planning and layout of canal system for irrigation
- IS 7112 : 2002 Criteria for Design of Cross-Section for Unlined Canals in Alluvial Soil
- IS 4701 : 1982 Code of practice for earthwork on canals
- IS 12331 : 1988 General Requirements for Canal Outlets
- IS 1498 : 1970 Classification and identification of soils for general engineering purposes
- IS 7986 : 1976 Code of practice for canal outlets
- IS 4839 : Part 1 : 1992 Maintenance of canals - Code of practice : Part 1 Unlined canals
- IS 4839 : Part 3 : 1992 Code of Practice for Maintenance of Canals - Part 3 : Canal Structures, Drains, Outlets, Jungle, Clearance, Plantation and Regulation
- IS 8835 : 1978 Guidelines for planning and design of surface drains

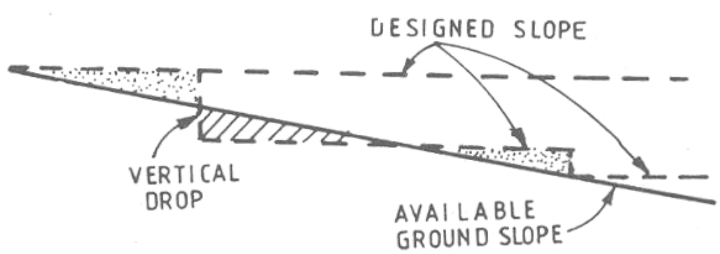
Chapter-8

Canal Falls - Selection of the type & Design principles.

Whenever natural ground slope is steeper than the designed bed slope of the canal, the difference is adjusted by constructing vertical 'drops' in the canal bed at suitable interval. **A canal fall is a structure designed to secure lowering of the water surface in a canal and to dissipate safely the surplus energy so liberated**.

Necessity of Fall:

It is a well known fact that the velocity in a canal is a function of slope and that there is a certain limiting velocity which can be allowed in a canal, depending upon the nature of the soil through which it passes, without causing it to erode its bed and sides. It follows therefore that there is a limiting water surface slope which cannot he exceeded without detriment to the canal. The slope of country -which a canal has to irrigate, is visually steeper than the water surface slope required for the canal as shown in figure below.



Introduction of falls at intervals becomes, therefore, a necessity, to absorb the differential head, and keep the canal fairly in balancing depth.The introduction of falls can, however, be avoided by skirting around the contours as was practised in the construction of some earlier canals but it introduces many curves and increases the length of the canal which besides being uneconomical, results in loss of command and increase in the absorption losses and creates silting and scouring troubles.

Development of fall:

The necessity of providing falls in our country became apparent early in the 19th century when big irrigation projects like Western and the Eastern Yamuna, the Cauvery, the Ganges, the Bari Doab canals etc. were constructed which called for economy and efficiency in the working of irrigation systems.

The introduction of falls, however, required the devising of ways and means to counteract its harmful effects, briefly described below, before they do damage to the bed and sides and endanger the safety of structure.

(i) When water passes over a fall, the water surface begins to drop down, starting from a small distance upstream of it so that the depth of water at the fall with its crest at bed level may have a minimum value of 2/3 of the full supply depth in a canal. This local drop increases the velocity upstream causing erosion in bed and sides. This action may extend upstream from a few metres to a considerable distance depending upon the crest length and its height above bed.

(ii) Considerable impact force is produced by a large mass of failing water at the foot of fall.

(iii) The flow immediately downstream of the fall is highly turbulent and the velocity distribution down-stream along a vertical plane, is not normal for a long distance lower down. This causes eddies, irregular currents, scour on the bed and the erosion on the sides of a canal, thus endangering the safety of the structure due to steepening of the exit gradient. Various types of falls, with different shapes, length and height of crest have been tried to ensure that for all discharges in the canal, the water surface level at the crest is the same as in the canal upstream. The vertical trapezoidal notch fall proved a success in maintaining depth discharge relation and was very widely used in India and even in other parts of the world. But on account of the greater difficulty,

cost of construction arid unfavourable conditions for regulation, it was discarded gradually in favour of the glacis fall.

To safeguard against the destructive effects at *(ii)* and *(iii)* above, vertical falls with deep water cushion on downstream, have been found quite successful for low drops. Occasionally grating were used to divide the flow into a large number of smaller stream and reduce the force of impact of falling water. Their action was similar to that of a large number of notch openings in a trapezoidal notch fall adopted later on. The main objection to this type of fall arose out of the difficulty of construction and the narrow openings which caused obstruction to the free passage of detritus. The dissipation of energy downstream, was also not complete for large discharges and high drops inspite of deep cistern and the addition of semi-circular lip to notches which helped to spread out the water in a fan like shape to reduce the force of impact.

With the phenomenon of standing wave or hydraulic jump being more clearly understood the glacis type of fall came for adoption, the glacis being straight or half parabolic and half straight with baffle wall placed across the path of flowing water in certain cases. The use of standing wave along with various types of energy dissipators has proved a great success in the safe dissipation of surplus energy with the result that glacis type falls have come into use extensively, particularly for large discharges and high drops. With the development of irrigation and ever increasing necessity to bring more and more area under irrigation, requiring economical use of water, it became extremely necessary to ensure accurate measurement of discharge, involving comparatively small time, labour and cost. The falls provided the most convenient points for measuring discharges along the canal. With this object in view, broad crest which .ensures constant coefficient of discharge under varying conditions of flow, came in use. The consequent slight increase in cost was proposed to be offset by reducing the crest width i.e. the idea of fluming or constricting laterally came to be adopted when a fall was combined with a bridge. The fluming does create severe conditions downstream, but proper downstream expansion and provision of energy dissipators can effectively prevent scouring on bed and sides.

Location of fall:

The location of fall in a canal depends upon the topography of the country through which the canal is passing. In case of the main canal which does not directly irrigate any area, the site of a fall is determined by considerations of economy in 'cost of excavation and filling' versus 'cost of falls'. The excavation and filling on two sides of a fall should be tried to be balanced. By providing a larger drop in one stop, the quantity of unbalanced earthwork increases. But the number of fall reduces. An economy between these two factors has to be worked out before deciding the location and extent of falls. The location of falls may also be influenced by the possibility of combining it with a bridge, regulator or some other masonry work for economy and better regulation.

To fix the site of falls on the canal full supply levels required at the heads of all off take canal and outlets of the canal are determined from the contour plan of area to be irrigated and plotted on the L-Section of the canal. The full supply level in the canal is then so marked that it covers all the command points and allows for a minimum working head of 0-30 metre for the off-take regulators and 0-15 metre for the outlets. The approximate position of the falls thus determine on the L-Section, may then be verified at site to see that it does not lie in a local depression or at a high land and that the soil is suitable for the foundations.

It is economical to flume a fall an combine it with a bridge, regulator or some other masonry work.. It may sometimes be found desirable apart from economic reasons, to provide a fewer number of falls of higher drops spaced far apart as con pared with a greater number of falls of lesser drop located closely, so as to enable installation of hydro power units.

Different types of Falls :

Various types of falls have been designed and tried since the inception of idea '***fall construction'*** came into being. The important types of such falls which were used/are being used in modern days are as below :-

Type 1 --The Vertical drop type:

In this type of fall, the nappe impinges clear into the water cushion below. There is no standing wave and the dissipation of energy is effected by the turbulent diffusion as the high velocity jet enters the deep pool of water downstream.

Type 2 --The Glacis type:

This type takes full advantage of the fact that a- standing wave is an effective natural means of energy dissipation. This type may be divided into three classes:

(a) *Straight glacis with baffle platform and baffle wall:* provided at a calculated height and distance from the toe of glacis, commonly known as Poona type or Baffle fall.

(b) Straight glacis without baffle platform and baffle wall: The straight glacis may also be replaced by half gravity parabolic glacis commonly known as Montague profile, in the case of bigger discharges.

(c) Modified Glacis Type: The modifications are in respect of glacis, pavement length and spacing, location and design of friction and toe blocks. This works satisfactorily under drowned conditions in unflumed small falls.

Type3-- Trapezoidal Notch Fall :

It consists of a number of trapezoidal notches constructed in a high crested wall across the channel with a smooth entrance and a flat circular lip projecting downstream from each notch to spread out the falling jet. The characteristics of a notch type fall are as follows :

i) It maintains the depth-discharge relationship to a fair degree.

ii) It can be calibrated as meter for free overflow conditions. It is however unreliable for drowned conditions.

iii) As it provides opening for flows down to the bed level, there is no obstruction to the flow of silt in the canal.

iv) When an off-taking distributory takes-off just upstream, the supplies to the distributory are automatically regulated as the depth-discharge relationship of the main canal is maintained by the fall.

Suitability of notch type fall as an alternative to glacis type fall may be studied and adopted if found economical.

Type 4--Chute Type Fall :

One way of negotiating steep fall in a terrain is by providing a chute. A chute is a special type of inclined fall where the lowering of the water surface is achieved over a longer length of canal where the slope that is flatter but steep enough to develop high velocities. Chutes are generally longer and larger than other falls.

A Chute structure usually consists of an inlet section, a chute canal in which the excess energy is dissipated and an outlet section through which water is discharged to the downstream canal. Inlet and outlet transitions are generally special type of transition.

Type 5 ---Well Type falls or siphon well drops :

This type of fall consists of an inlet well with a pipe at its bottom, carrying water from the inlet well to the downstream well or a cistern. The water falls into the inlet well through a trapezoidal notch constructed in the steining of the well, from where it emerges near the bottom, dissipating its energy in turbulence inside the well.

Types based on functions :

Flumed or unflumed falls: Depending upon the crest width being smaller or equal to bed width of the canal.

Meter or non-meter fall: Depending upon the purpose, if the fall, is also to serve as a discharge meter or not.

Selection of type of fall:

In selecting a type of fall most suitable for particular site, the main consideration is the height of drop and the discharge passing over the fall or in other words, the amount of energy to be dissipated downstream of the fall. The type which dissipates this energy most satisfactorily is to be preferred.

In cases, where full dissipation of energy does not take place on the masonry structure of the fall, the issuing jet has still got higher bed velocities than what the soil can withstand, in such cases a baffle design, though costly in construction may prove economical in the long run, from the maintenance point of view. Where the bed material is hard enough to withstand scouring action of the strong current, the type of design that is just. enough to dissipate the surplus energy should be adopted.

Design of a fall envisages the following aspects:-

(i} t should be cheap in initial cost.

(ii} Recurring expenditure on maintenance and repairs should be minimum.

(iii) limited amount of harmless scour may be tolerated.

(iv) The design should work under 10 percent retrogression, if it were to occur subsequently.

(v It should be capable of being used as a meter.

When a fall is to be used as a meter, a standard baffle design is most satisfactory. But in actual practice, it is neither necessary nor economical to design every fall as a meter flume. A modified design of crest and approaches of the baffle fall reduces its cost but the coefficient of discharge for a short crest does not remain constant. In order that it may be adopted for metering, the fall has to be fully calibrated. This involves considerable research work. The calibration curves will vary with different factors and as they are to be used by the field staff, it is feared they may cause confusion and serious errors in metering may result. From this point of view alone, the standard broad crested meters are to be preferred. They will not add much to the cost as they will be few in number.

The fall should also ensure a constant coefficient of discharge under all conditions of working. Generally, a vertical fall is not suitable as a meter due to the formation of partial vacuum under the nappe and glacis falls may be preferred in which case the minimum working head should not be less than 0.2 D, where D is depth of crest below total energy line upstream in metres. It has, however, been found that in the case of vertical falls, critical section is adherent and stable up to minimum modular head of 0.33 D provided there is not high submergence. A vertical fall may, therefore, be used as discharge meter when working head is more than 0.33 D and the downstream full supply level is up to or lower than, the crest level. The Glacis Falls are however more reliable as discharge meters even at low working heads.

For Unflumed Designs : The Baffle design is well suited under clear overfall conditions specially where the soil is easily erodable. Under drowned conditions, the design may be. adopted on the lines suggested in the accompanying table. For discharge between 15 to 8 cumecs, whenever soil is erodable, baffle type design, may be adopted due to its efficiency in energy dissipation. Below 8 cumecs, the choice may be based on cost considerations alone.

For Flumed Designs: The vertical type is not suitable as effective dissipation of energy is not possible and results in. harmful scouring. The choice of design is limited only to baffle type and sloping glacis type. Under drowned conditions, baffle type may be rejected in view of the experiments conducted at Central Water and Power Research Station, Poona, which have revealed its unsuitability under drowned conditions, and the straight glacis type with three to four rows of friction blocks and a deflector may be adopted.

Baffle type design, is also recommended for retrogressed conditions, when there is a likelihood of formation of hydraulic jump due to lowering of the tail water levels, both in flumed and un-flumed designs.

The following Table serves as a guideline for selecting the type of fall to be adopted.

S. No	Discharge in cumecs M/s	Drops (H) in m.	Recommended Type
1.	High Discharge $Q \geq 15$	$H_L \geq 1.5m$	Baffle type
2.	High Discharge $Q \geq 15$	$H_L < 1.5m$	Baffle/sloping glacis type
3.	Low Discharge $Q \leq 15$	$H_L \geq 1.5m$	Modified glacis type
4.	$Q \leq 15$	$H_L \leq 1.5m$	sloping glacis type
5.	$Q \leq 8$	$H_L \leq 1.5m$	Vertical drop fall /Trapezoidal notch fall type
6.	$Q \leq 1.5$	$H_L \leq 3.0m$	Well drop fall

Design Principle :

A fall consists of the following main parts as shown in figure below :

i) Upstream approach
ii) Throat
iii) Downstream Glacis
iv) D/s. Expansion
v) Energy dissipater.

Upstream Approach:

The design requirements of the upstream approach depend upon the functions of fall. If it combines with it the function of discharge meter as well, the side and bed approaches to the crest have necessarily to be gradual and smooth so as to avoid eddies and impact losses and reduce concentration of flow. A bell-mouth approach such that the change of velocity is smooth throughout the transition, is desirable.

Throat:

Clear width of Throat (Bt) : In the case of vertical falls ,it has been found that increased intensity of discharge due to fluming creates unfavourable scour on the downstream side. The vertical falls should be full width fall *i.e.* The width of the crest should be equal to the bed width of the canal. If the canal widths upstream and downstream of the fall are different, the crest width adopted should be the smaller of the two bed widths. This , however is not the case in the glacis falls which may be flumed when combined with bridge so as to economise in the cost. The fluming should be kept within given ranges as determined from experience, so as to keep the scouring action downstream within limits; otherwise the very object of fluming may be defeated due to longer expansions, thicker floor and heavier protection needed on the downstream side.

It is quite rational to design such discharges per metre run of crest width which with the height of drop available, gives a value of total energy (E_{f2}) on the downstream side equal to the full supply depth of canal. It does not then require deep cistern downstream and avoids difficult construction in foundations when the sub-soil water level is high.

Length of the Throat (Lt): A narrow crest gives a higher value of co-efficient of discharge and consequently a higher discharging capacity than a broad crest. It is therefore economical to keep a short crest as it reduces the cost of the work both transversely and longitudinally. A narrow crest does not however ensure a constant co-efficient of discharge for all conditions of working. If therefore a fall is to be used as a meter, broad crest should be used. The length of crest should then be such that the not only the convex flow dies out on the crest, followed by critical conditions of flow, but it ensures hypercritical flow below the critical section for a sufficient length. To ensure this the length of crest should be kept equal to a little more than 2D, the minimum required for broad crest.

For non-metered falls, narrow crest may be adopted for the sake of economy. Experiments on model has shown that a crest length of 2/3 D for glacis fall and $0.55\sqrt{D}$ for vertical falls is quite suitable and may be adopted.

Downstream Glacis:

In the vertical fall, since, there is no glacis or sloping floor downstream of the crest, the deep cushion of water is depended upon for dissipation of energy due to falling water. This method cannot however be extended to larger falls. The use of glacis which ensures the formation of standing wave or hydraulic jump at its toe, is then, resorted to. The surplus energy is destroyed by the impact between the hypercritical flow below the critical section and the sub-critical flow in the channel downstream. The slope of glacis should however be such that it imparts maximum horizontal acceleration and thus ensures optimum dissipation of energy.

To ensure this, Montague developed a half gravity parabola glacis known as Montague type and it can be deduced from the following equation:—

$$X = U\sqrt{\frac{4Y}{g}} + Y$$

Where,

X = The horizontal ordinate measured from the downstream edge of crest.

Y = The vertical ordinate measured below crest level.

U = Critical Velocity = $(q^2/g)^{1/3}$

g = Acceleration due to gravity.

Downstream Expansion:

It is a sound practice confirmed by experiments to extend the parallel and vertical side down to the toe of the glacis in the case of glacis fall and to the end of baffle platform for baffle fall *i.e.* the point of formation of standing wave. In the case of vertical fall, however, these shall extend only up to the downstream end of crest. The expansion afterwards should be gradual so that expanding flow adheres to the sides. This prevents formation of back rollers on sides, which cause harmful scour on the downstream side.

Energy Dissipators:

Followings are the broad catagorisation of types of phenomenon used for carrying out energy dissipations in canal falls:

I. Interaction and Impact:

In the case of vertical falls, the energy is dissipated by means of impact and sudden deflection of velocity from vertical to horizontal direction. A water cushion is provided at the toe of drop to protect the floor against the impact of falling water and to lessen down the velocity before it leaves the structure. The water cushion is formed by suitably depressing the floor below the downstream bed of canal.

Hydraulic jump or the standing wave, is considered to be the most effective means of dissipating the energy and reducing the hypercritical velocity to normal subcritical velocity in the canal downstream. This phenomenon is utilized in Glacis Fall.

II. Mechanical Energy Dissipators:

Various types of energy dissipators have been used from time to time. Each has its usefulness to a varying degree depending upon its shape and location in stabilizing the flow, reducing its turbulent action; forming positive horizontal vortices and thus preventing scour in bed and on sides.

. ***Friction blocks:***

—These are found most effective and serve the following purposes:—-

(a) to fan out the flow and thus prevent return fow at sides causing eddies and scour.
(b) to deflect upward the high speed flow above the bed which reduces scour action below the work.
(c) to withstand impact of high speed jet and thus dissipate energy.
(d) to assist in the formation of standing wave in the case of glacis fall without baffle.

Deflector Wall::

In glacis falls a deflector wall of height $d_3/10$ (d_3 = full supply depth of canal downstream of the fall) is provided at downstream end of cistern. This helps in the formation of horizontal positive vortices. The result is piling up of the bed material against the curtain wall which ensures the latter's safety. Minimum height should be 15 cm..

Chapter – 9

Canal Design: Lined and Unlined Canals

Design of a Canal consists of determination of the cross-sectional area, depth of flow, bed width, side slopes and longitudinal slope etc for a given design discharge and boundary surface. The design of canal is mainly governed by the quantity of silt in the water and the type of wetted boundary surface. Depending upon these factors, the irrigation channels can be broadly classified into the following types:

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

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

In rigid boundary channels, the surface of the channels is lined. The quantity of silt transported by such channels remains more or less the same as that entered the channel at its head.

In the case of alluvial channels, the quantity of silt may vary from section to section along reach. The silt quantity may increase due to scouring bed and sides of channel or may decrease at some sections due to silting. Scouring occurs when the velocity is high and silting takes place when the velocity is low. Both scouring and silting will lead to loss of command due to reduced FSL and carrying capacity. Channels in alluvium are to be designed for a non-scouring & non-silting velocity called the *critical velocity*.

DESIGN OF RIGID BOUNDARY CHANNELS (DESIGN OF LINED CANALS)

Before getting into design of lined canals it is necessary discuss the type of linings available, the need for lining and economics of lining.

Lining is a system of reducing seepage losses and improving conveyance efficiency of the canal channel section material by improving its properties of impermeability, Rugosity coefficient, strength etc with introduction of a rigid surface or flexible layer to the channel section.

Typical cross section of the lined canal in cutting and filling are given in fig 1 and 2. Typical cross section of the lined canal in stone cutting is shown in fig 3

Types of lining

Lining can be classified into following categories:

1. Exposed and Hard surface lining
   a) Cement Concrete Lining
      i. Un-reinforced
      ii. Reinforced

b) Shotcrete lining
c) Precast concrete lining
d) Brick Tile lining
e) Stone lining (Stone masonry, stone pitching)
f) Asphaltic Concrete lining
g) Soil cement lining
h) Earth lining

2. Buried Membrane lining
a) Sprayed in place asphaltic membrane lining
b) Ready made asphaltic membrane lining
c) Plastic film and synthetic rubber membrane lining
d) Betonite and clay membrane lining

Need of lining / Advantages of lining

1) Water Conservation

Chief advantage of lining is reduction in seepage and thereby conservation of water which can be put to some other beneficial use. Aquatic weeds transpire a lot of water, with the provision of lining, such weed growth is reduced. This also helps in water conservation.

Lining of canal does not altogether eliminate seepage. Reduction in seepage depends upon the type of lining, surface area of lining, surrounding water table, sub grade material, operation characteristics of canal system etc. As a rough estimate seepage from a properly lined canal does not exceed 0.03 m^3/m^2/day.

2) Prevention of water logging

Seepage water causes water logging in the nearby low lying areas especially if the drainage of that area is not proper. Vast tracts of lands have been rendered uncultivable due to water logging. Lowering of water table in the surrounding area of unlined canal increases seepage. Lining reduces seepage from the canal and thereby helps in preventing waterlogging.

However to prevent waterlogging lining of entire canal system falling in the problem area may be necessary. Besides, water should not be excessively applied to the fields and proper drainage arrangement should be made.

3) Reduced water way and right of way costs

By provision of lining, much higher velocities can be adopted in the canal. Therefore the section of the canal is reduced. Also with lining, steeper slopes can be provided thereby further reducing the land required for canals. Narrower water way implies lower cost on account of cross shorter drainage works and bridges

4) Reduced operation and maintenance costs

Aquatic weed growth is discouraged in the lined channels thereby reducing its maintenance and operation costs. Silt removal is also less and with improved Rugositycoeff, the removal of silt is also easier. Lined channels are more stable than unlined channels and require less maintenance. Overall as a rough estimate it may be said that in Indian conditions, the maintenance costs can be reduced by about 50% by provision of lining.

5) Protection against erosion, structural safety and other benefits

A lined channel withstands erosion in a much better way than unlined channel. Steeper slopes are also more stable in lined channels. Flatter slopes can be provided in power channels and available generation head can be increased. Sometimes lining is necessitated by the poor stability of the subgrades.

Economics of lining

Provision of lining and type of lining depends upon relative economics of the alternatives. A lining is justified only when the benefits accrued from providing the lining outweigh the cost of lining. When soil is very impermeable and its stability is also good, the lining may not be justified.

While deciding a particular type of lining it may also be considered whether the entire canal system is to be lined or a part can serve the purpose. Lining of the distribution system can save more water as the percentage losses are more in the smaller channels, because the perimeter of the smaller channels is more in proportion to the area of the channel and smaller channels may be running intermittently. Therefore techo-economic studies should be carried out for each case in detail and system approach using acquifer modeling and optimization techniques may be adopted to study the complete water balance of the area. As all the seepage water is not lost in Besides it may also be studied if the water saved from lining the canal can be put to beneficial use.

Lining of the canals at the time of original construction itself is more advantageous than at a later stage because the size of associated structures such as cross drainage works, bridges, pumping stations and distributaries necessary to provide given amount of water is reduced. Major economic in construction is possible when concrete lining is planned and placed at the time of original canal construction. Unlined or earth lined canal require much larger cross section area than lined canals to avoid damaging high velocities which may be safely provided in lined canals. Hence at the time of providing lining to existing canals, unnecessary large lining shall have to provided or backfilling and reshaping of canal shall have to be done. Both of these alternatives are costlier than properly designed lined canals.

The calculations needed for economic study of canal are given as under:

Sl. No.	Details of work	Cost
	Land saved	
1	Right of waterway, unlined canal in acres	
2	Right of waterway, lined canal in acres	
3	Right of waterway saved (1)-(2) in acres =(3)	
4	Reclaimed waterlogged land in acres	
5	Total land saved in acres (3) +(4) = (5)	
6	Annual value of total land saved in acres at net crop value in acres	
	Water saved	
7	Flow when in use, in cusecs	
8	Hours in use(in whole year 365X24)	
9	Total flow per year = (7) X (8) X(.0826) = (9) in acres ft	
10	Estimated loss from unlined canal(% of (9)) in acre ft = (10)	
11	Annual value of water saved in acre ft = (10 X rate per acre ft =(11)	
Savings in maintenance (including maintenance of necessary drainage facilities)		
12	Annual maintenance of unlined canals	
13	Annual maintenance of lined canals	
14	Annual savings in maintenance (12)-(13) =(14)	
Labour saved		
15	Labour. Irrigation from unlined canal in man days per year (if any)	
16	Labour. Irrigation from lined canal in man days per year (if any)	
17	Annual savings in labour in man days per year (15)-(16) =(17)	
18	Total annual tangible benefits = (6) +(11) + (14) + (17) = (18)	
19	Estimated annual intangible benefits (socio economic uplifts etc) including pride of the nation)	
20	Total annual benefits (18) + (19) = (20)	

Criteria for selection of type of lining

IS 10430- 2000 "CRITERIA FOR DESIGN OF LINED CANALS AND GUIDANCE FOR SELECTION OF TYPE OF LINING" gives the essential factor to be considered for selection of type of lining.

General guidelines for selection of type of lining:

a. Channels with bed widths upto 3.0 m
   1. single burnt clay tiles or brick lining where seepage considerations are important
   2. P.C.C slab lining and
   3. Flexible membrane lining with adequate tile / earth cover

b. Channels with bed width 3.0 m to 8.0 m
   1. lining of single burnt clay tile
   2. P.C.C slab lining and
   3. Combination lining (flexible membrane lining in the bed and rigid lining on the sides). This may be adopted where the channels have become stable and no danger of scour is expected.

c. Channels with bed width greater than 8.0 m
   1. Insitu cement concrete lining in bed and sides in accordance with IS 3873-1978
   2. Insitu cement concrete lining in bed and P.C.C. slab lining on sides and
   3. Burnt clay tile lining in accordance with Is 3872-1966 9double on the sides and single on the bed) where aggregate for manufacture of concrete are not available economically.

For lining of canals in expansive soils refer IS 9451-1980.

However for making a choice between various types of lining following factors should be considered carefully:

1) Availability of construction material

A lot of construction materials are required for the construction of lining. Therefore, generally the most economic lining is the one which makes the best use of locally available materials. But usually thesematerials have to be brought from some considerable distance to the work site. In such case the transportation issues and economics may be considered carefully.

2) Availability of labour and material

Some linings are labour intensive e.g Tile lining or brick lining. Whereas some linings can be speedily constructed with the help of machines. The relative economics of each alternative may be considered before selecting a type of lining.

3) Durability and Reparability of lining.

The canal lining should be able to with stand the effects of flowing water, rain, sunshine, thermal and moisture changes, chemical action etc. It should also be able to withstand damaging effects caused by cattle movement, rodents and weed growth.

The durability of lining depends on the type of lining, the quality of construction and canal operation and maintenance. However some types of lining e.g. brick or tile linings are more easily repairable than cement concrete lining. Therefore before selecting any type of lining cost benefit calculation in this regard is also needed to be carried out.

4) Structural stability

Although the lining is supported by the subgrade and it transmits all the load to subgrade. But it must be structurally strong enough to withstand the subsoil pressure from behind when the sub soil gets saturated through rains or in case of sudden draw down. It must be structurally strong enough to withstand any local cavity formed due to incomplete compaction of sub grade.

Pressure relief valves and drains are also provided behind the lining to dissipate the excess differential pressure due to saturation of soil.

5) *Hydraulic Efficiency*

Discharge carrying capacity of the canal increases with decrease in the value of Rugositycoeff. The Rugositycoeff of a lining depends upon its material and it increases with deterioration and time. A lining with lower Rugositycoeff during its lifetime will be more efficient and economical.

6) *Water tightness*

The seepage losses from the canal depends upon type of lining, water table of the surrounding area, type of sub grade, continuous or intermittent canal operation etc. Relative economics of conserving water and extra expenditure on account of lining may be undertaken.

7) *Type of Sub grade*

Type of Lining selected depends upon the type of subgrade on which it is supported. For example concrete or some other type of rigid lining if laid over subgrades containing swelling type clays or gypsum are likely to get damaged. Sometimes it is advantageous to remove unsuitable soils in a short reach.. Suitability of various types of subgrades as classified under IS 1498- 1970 "Classification and identification of soils for general engineering purposes" has been given below.

Sl No and Relative Rating	Soil Classification	Soil Description	Remarks
1	GP	Poorly graded gravel or gravel sand mixture	Extremely permeable. Needs lining
2	GW	Well graded gravel, gravel sand mixtures, little or no fines	Extremely permeable. Needs lining
3	SP	Poorly graded sans or gravely sands; little or no fines	Moderate to highly permeable, usually requires lining
4	SW	Well graded sands, gravelly sands, little or no fines	Moderately permeable. Usually requires lining
5	CH	Inorganic clays of high plasticity, fat clays	Very impermeable when wet or extremely permeable after drying, needs special consideration
6	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with none to low plasticity	Fairly impervious but bank erosion difficult to hold. Needs special attention

7	MH	Inorganic silts of high compressibility, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Fairly impervious but bank erosion difficult to hold. Needs special attention
8	GC	Clayey gravels, poorly graded gravel-sand-clay mixture	May range from moderate to very low permeability
9	SC	Clayey sands, poorly graded sand –clay mixture	Usually impermeable. Good stability
10	GM	Silty gravel, poorly graded gravel-sand-silt mixture	Usually fairly impermeable but hard to hold on bank. Lining requirement is minimal
11	CL	Inorganic clays, gravelly / sandy / silty clays, lean clay of low plasticity	Usually very impermeable. Lining requirement is minimal
12	OL	Organic silts and silty clays of low plasticity	Permeability fairly low but stability is questionable. Provision of lining needs judicious consideration
13	OH	Organic clays of medium to high plasticity	Low permeability if soil is kept wet but stability is questionable and shrinkage cracks are probable. Provision of lining needs judicious considerations.

8) Operation and Maintenance:

If the operation of the canal requires frequent running and closing or large water level fluctuations, a hard surface lining will be preferred. In case of earth linings or earth covered membrane linings, such conditions would speed up the deterioration process. As regard to weed control, small repairs and silt removal etc. hard surface lining s have greater advantage over the latter type of the linings. Ability of the lining to withstand weed growth and burrowing by the rodent should also be considered as these affect the maintenance.

If water table is above the canal bed, closing the canal will subject the lining to external hydrostatic pressure. Some pressure relief arrangement may have to be incorporated as in this case no lining can resist such pressures

9) Land Value

Where the land value is high, providing lining with lower Rugosity coefficient, higher velocities can be allowed and land required for canal can be reduced to achieve overall economy.

Design of Lined Cross Section:

The discharge that can pass through a canal section is given by
Q= A x V where

A = Area of cross section
V = Mean velocity

Mean velocity of the lined canal can be calculated by Manning's formula as

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

Where

V = Mean velocity in m/s
R = hydraulic mean depth
S = Slope of canal
n = Manning's Rugosity Coefficient

Design of lining involves the following components:

1) Cross Section
2) Inner Slope of lined canal cross section
3) Rugosity Coefficient
4) Free Board
5) Velocity
6) Under Drainage
7) Other Cross Section elements like Berm, Bank top Width, Dowla, Roadway etc

1) Cross Section

The cross section of the lined canal may be:

a) Trapezoidal with or without rounded corners . This section can be used for all types of lined canals

b) Cup shaped. It may be used for distributaries or minors for discharge up to 3 m^3/s

2) Inner Slope of lined canal cross section

Lining is usually made to rest on stable slopes of the natural soil; So slopes should be such that no earth pressure or any other external pressure is exerted over the back of the lining. Sudden drawdown in the lined canal should be controlled by strict operation rules and regulations to avoid external pressure on the lining. However where the chances of sudden drawdown in the channel is considerable., the canal slopes should be checked for stability using slip circle analysis as given in IS 7894. In addition proper drainage behind he lining should also be ensured.

Steeper slopes are economical but stable slopes depending on the type of soil are preferred. The following slopes depending upon the soil are recommended as per IS 10430-2000:

S. No	Type of soil	Side Slope (Horzontal : Vertical)
1	Very light loose sand to average sandy soil	2:1 to 3:1
2	Sandy Loam	1.5 : 1 to 2 : 1 (in cutting) 2 : 1 (in embankment)
3	Sandy gravel / muram	1.5 : 1 (in cutting) 1.5 : 1 to 2 : 1 (in embankment)
4	Black cottom	1.5 : 1 to 2.5 : 1 (in cutting) 2 : 1 to 3.5 : 1(in embankment)
5	Clayey Soil	1:5 to 1 to 2:1 (in cutting) 1.5 :1 to 2.5:1 (in embankment)
6	Rock	0.25 : 1 to 0.5:1

The above slopes are recommended for depths of cutting / height of embankment upto 6.0 m . For depth / height in excess of the above, special studies for the stability of slopes are recommended

The outer slopes of the lined sections should be based on the engineering properties of the soil. Where the fill height is more than 6.0 m need for introduction of berms should be kept in view.

3) Rugosity Coefficient

As per IS 10430-200, the rugosity coefficient values for different types of linings are as follows:

S. No.	Surface Characteristics	Value of n
i)	Concrete with surface as indicated below	
	a) Formed, no finish / P.C.C tiles or slabs	0.018 to 0.020
	b) trowel float finish	0.015 to 0.018
	c) gunited finish	0.018 to 0.022
ii)	Brick / tile lining	0.018 to 0.020
iii)	U.C. R. / Random rubble masonry with pointing	0.024 to 0.026
iv)	Asphalt	
	a) Smooth	0.013 to 0.015
	b) Rough	0.016 to 0.018
v)	Concrete bed trowel / float finish and slopes as indicated below:	
	a) Hammer dressed stone masonry	0.019 to 0.021
	b) Coursed Rubble masonry	0.018 to 0.020
	b) Random Rubble Masonry	0.020 to 0.025
	c) Masonry plastered	0.015 to 0.017
	d) Stone pitched lining	0.020 to 0.030
vi)	Gravel bed with side slope characteristics as given below:	
	a) Formed Concrete	0.02 to 0.022
	b) Random rubble in mortar	0.07 to 0.023
	c) Dry rubble (Rip rap)	0.023 to 0.033

1. For canals with an alignment other than straight, a small increase in the value of n may be made or alternatively bend losses may be accounted for. In cases of canals with relatively higher values of discharge in straight reaches, lower value of n as indicated above may be adopted
2. the n value shall be decided in view of the age of lining, surface roughness, weed growth, channel irregularities, canal alignment, silting, suspended material and bed load etc.

Equivalent roughness;
Where side and bed slopes have different types of linings, the equivalent rugosity coefficient can be calculated by the following formula:

$$n = \frac{\sum_{i=1}^{N}\left[n_i^{3/2} P_i\right]^{2/3}}{P^{2/3}}$$

where

P_i	=	lengths of different portions of perimeter with corresponding roughness
N_i	=	roughness of portion Pi

$$P = \sum_{i=1}^{N} P_i$$

4) Free Board

Free Board is measured from the full supply level to the top of the lining. It depends upon size of canal, velocity of canal, curvature, wind and wave action and method of operation. USBR recommends the following values of free board:

Is 10430-2000 recommends the following values of freeboard:

Discharge (cumecs)	> 10	3-10	1-3	<1	<0.1(watercourses)
Freeboard (meter)	0.75	0.60	0.50	0.30	0.15

However as given in Manual on Canal Lining published by INCID, the Central Water Commission recommends the following values of freeboard

Discharge (cumecs)	upto 0.7	0.7 to 1.4	1.4 to 8.5	over 8.5
Freeboard (meter)	0.46	0.61	0.76	0.92

However in case of power channels and in lengths in the vicinity of intake and forebay area, consideration needs to be given to the surge height due to sudden closure or start of the power house.

5) Velocity

Maximum permissible velocities for guidance for some types of linings are as follows:

a)	Stone pitched lining	1.5 m/s
b)	Burnt Clay tile or brick lining	1.8 m/s
c)	Cement Concrete lining	2.7 m/s

however to ensure that silting does not take place in the lined canals, the critical velocity ratio should be as higher than unity.

6) Other Cross Section elements like Berm, Bank top Width, Dowla, Roadway etc

These cross section elements can be taken as per guidelines given in IS 10430-200 "CRITERIA FOR DESIGN OF LINED CANALS AND GUIDANCE FOR SELECTION OF TYPE OF LINING"

<u>Provision of Drainage Valves and Under Drainage</u>

Embankments of relatively permeable soils (e.g gravels with sandy soil having permeability greater than 10^{-4} cm/s) do not need drainage measures behind the lining. However, the following conditions require suitable drainage measures to be provided to protect the lining in accordance with IS 4558-1983.

i) Where the lined canals passes through an area with seasonal ground water level likely to be higher than water level inside the canal
ii) Where sub grade is sufficiently impermeable to prevent the free drainage of seepage or leakage from the canal
iii) Where there is built up pressure due to time lag between the drainage of the sub grade following draw down of canal.

Under the above conditions, stability of the lining is threatened. In such cases it becomes essential to design the drainage arrangement behind the lining. The drainage consists of following arrangements:

a) Longitudinal Drains
b) Transverse Drains
c) Pressure Relief Valves
d) Dwarf Regulators

a) Longitudinal Drains

The number of longitudinal drains in a canal depends upon the bed width. In the bed of the canal generally one drain every 10 m should be provided. For bed width minimum one drain and for bed width more than 10 m minimum two drains should be provided. The drainage pipe may be asbestos cement or PVC. Usually 150 mm diameter pipes are used. The perforations should be 12 mm in diameter and should be done by drilling. On the average there should be a minimum of 100 perforations / holes per meter length of the pipe. The drains should be placed symmetrical with reference to centre line of the canal. The drains should be carefully filled up to the bottom of the bottom of the lining with graded filter and compacted to form an even bedding for canal

b) Transverse Drains

Transverse drains should be provided, where necessary in the bed and on side slopes upto free board level. Generally transverse drains are provided on 10 m interval. However their spacing depends upon the size, location and efficiency of pressure relief valves.

c) Pressure Relief Valves (PVR)

Pressure relief valves which open into the canal are provided to relieve excessive hydrostatic pressure behind lining. These are to be provided in pockets of suitable sizes and shapes and filled with graded filter underneath the lining. The PVR's can be classified according to their material of construction e.g metallic or non-metallic. These can be further termed according to their mode of placement such as horizontal or vertical. Still further they can be classified according to their closing member such as flap type or ball type. At present usually non-metallic PVR's are used as they have clear advantages over metallic PVR's. It is suggested that before finally selecting any particular type of PVR, their suitability in field conditions should be ascertained by getting them tested from any Lab. U.P. Research Institute, Roorkeehas such testing facility available.

Generally PVR's are installed in horizontal and vertical position and their size is generally 150 mm for bed and 75 mm for side slopes. The PVR's should be provided on the longitudinal / transverse drains. If there are no transverse drains, the PVR's can be provided in pockets filled with graded filter underneath the lining. Pockets may be 600 mm square or cylindrical with diameter 600 mm. Pockets on the side slopes should be excavated with their sides at right angle to the slope.

The spacing of PVR should be decided as per site conditions. However one PVR for every 100 square meter area and in canal bed one PVR for every 40 square meter in side should be provided. On the sides in general one row at every 4 meter spacing should be provided. The first row should be provided about 50 cm above bottom and top row at 50 cm to 100 cm below full supply level. Valves in adjacent rows should be staggered. Model studies or semi field studies are sometimes adopted to optimize the spacing and other features of PVR's. Performance of any particular PVR installed earlier is also helps in selecting any particular type / spacing etc.

c) Dwarf Regulators

Where water table remains continuously high and the conventional pressure release arrangement as mentioned above are not likely to prove adequate, dwarf regulators re provided. These are regulators of part height and constructed across lined canal at suitable intervals for ponding up water to counteract excess uplift pressure below the lining. The height and spacing of Dwarf Regulators will depend on the minimum depth of water required to be maintained inside the canal for counterbalancing the residual pressure on the slope of canal. These also help in maintaining the minimum water depth in the canal even when the canal runs on part capacity. However their disadvantages are head loss, siltation and difficulty in empting the canal for repairs etc.

In Main Western Gandak canal which is designed for head discharge of 442 m3/s with double brick tile lining on side and single brick lining in the bed has been provided with dwarf regulators at spacing of about 3 km in reaches where uplift pressure is likely to pose problems.

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DESIGN OF UNLINED CANALS IN NON-ALLUVIAL SOILS

The design is same as that of the lined canals except appropriate change in maximum permitted velocity and rugosity coefficient as given below

coefficient of rugosity 'n' (Irrespective of the discharge): IS-7112-1973 and IS-10430-2000
The following values are recommended :
Normal alluvial soils- 0.0225
Murum- 0.025
Rocky strata- 0.03-0.035

Allowable velocities:-		C.B.I.P 171
Unlined canals - Recommended Velocities :		
All Soils	0.6 to 1.1m/sec	Technical Report No.7 of C.B.I.P
Hard clay or grit	1.0 to 1.5m/sec	
Gravel and shingle	1.5 to 1.8m/sec	
Cemented gravel conglomite,hard pan	1.8 m/sec	
Soft Rock	1.4 m/sec	
Hard rock	2.4 m/sec	
Very Hard Rock	4.5m/sec.	

DESIGN OF UNLINED ALLUVIAL CANALS

IS 7112- 2002 "CRITERIA FOR DESIGN OF CROSS-SECTION FOR UNLINED CANALS IN ALLUVIAL SOILS" gives methods for design of unlined canals in alluvial soils

LACEY'S METHOD FOR DESIGN OF UNLINED CANALS IN ALLUVIUM
According to Lacey, a canal is said to haveattained regime condition when a balance betweensilting and scouring and dynamic equilibrium in theforces generating and maintaining the canal cross-section and gradient are obtained. If a canal runsindefinitely with constant discharge and sedimentcharge rates, it will attain a definite stable sectionhaving a definite slope. If a canal is designed with asection too small for a given discharge and it's slopeis kept steeper than required, scour will occur till finalregime is obtained. On the other hand, if the sectionis too large for the discharge and the slope is (flatterthan required, silting will occur till true regime isobtained. In practice true regime conditions do notdevelop because of variations in discharge andsediment rates.

Lacey's design equations are in terms of two known quantities namely the design discharge Q and the mean diameter d, these are:

$P = 4.75\sqrt{Q}$
$S = 0.0003f^{5/3}/Q^{1/6}$
$R = 0.47Q^{1/3}/f^{1/3}$
Where f is Lacey's silt factor and is given by
$f = 1.76\sqrt{d}$ where d is mean particle size in mm

A channel can be designed for known Q, d and assumed side slope of a trapezoidal channel Z:1.

TRACTIVE FORCE APPROACH FOR DESIGN OF UNLINED CANALS

The unit tractive force exerted on bed of a running canal can be calculated from the formula:

$$\tau = \gamma RS$$

where

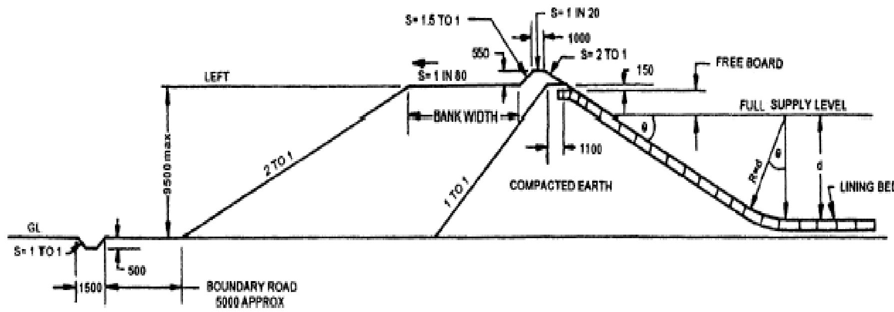
τ = unit tractive force in kg/m^2,
γ = the unit weight of water in kg/m^3 (usually 1000 kg/m^3),
R - the hydraulic mean radius in m, and
S = the canal slope.

The permissible tractive force may be defined as the maximum tractive force that will not cause serious erosion of the material forming the canal bed on a level surface. The permissible tractive force is afunction of average particle size (d_{50}) of canal bed in case of canals in sandy soils and void ratio in case ofcanals in clayey soils and sediment concentration. Thevalues of permissible tractive force for straight canal have been given by some authors on the basis of laboratory experiments but the same can better be determined by analysis of observed data on existing canals. Once this is done this would provide a rationalapproach to the design of section of regime channels.The values of permissible tractive force for sinuouscanals may be reduced by 10 percent for slightly sinuous ones, by 25 percent for moderately sinuousones and by 40 percent for very sinuous ones.

A suitable bed slope is to be selected either with referenceto average ground slope along the canal alignment oron the basis of experience and the value of R shall heobtained from above equation. Knowing the value of Rand assuming a suitable value of n for the canal,referring to Table 3(IS 7112:2002) as a guide, the average desirablevelocity of flow in the canal may be determined by using the Manning's formula given below

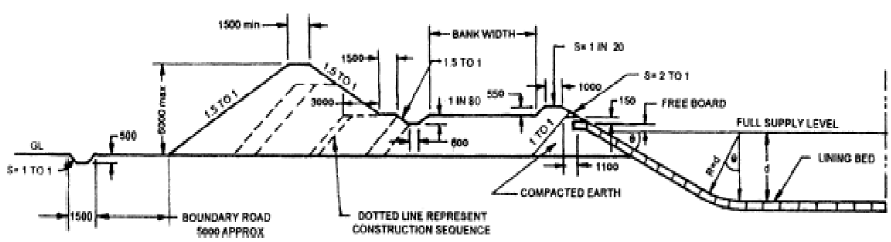
$$V = 1/n\, R^{2/3} S^{1/2}$$

The area of cross-section, bed width and depth of flow can be determined by assuming suitable side slopes.



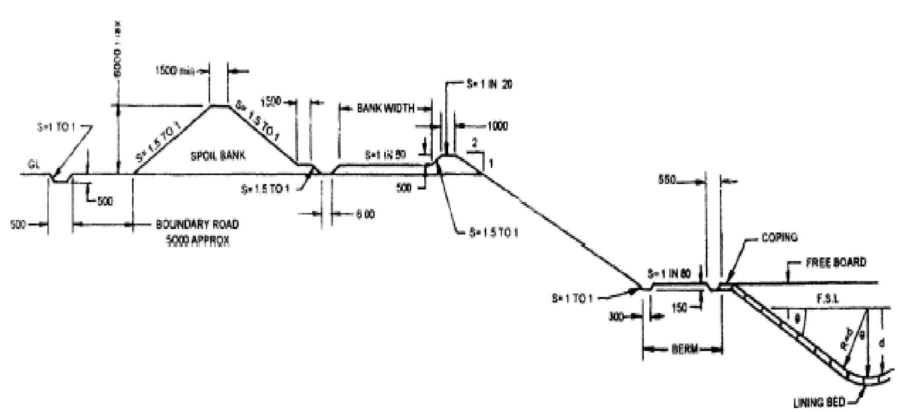
All dimensions in millimetres.

FIG. 1A NATURAL GROUND BELOW BED



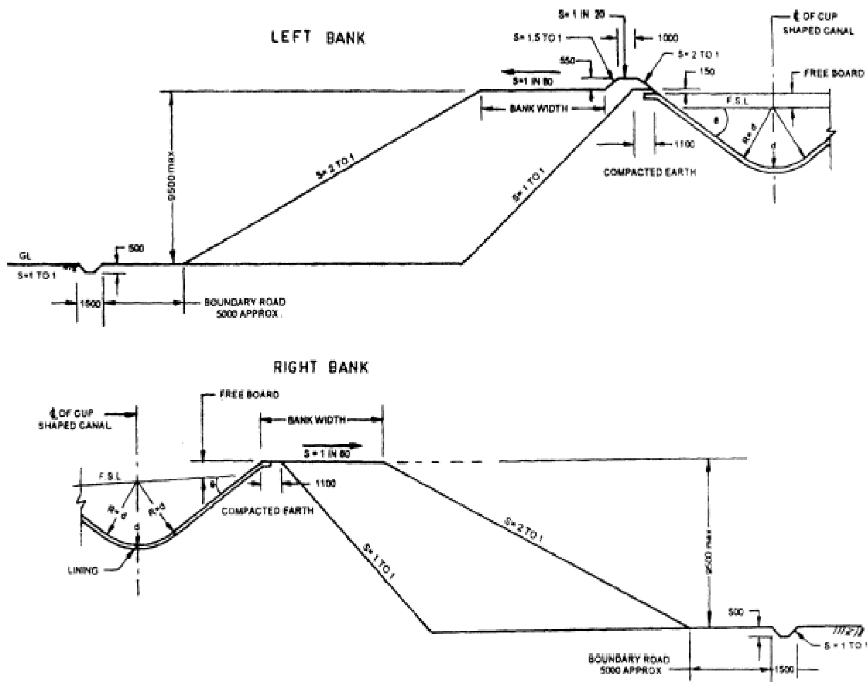
All dimensions in millimetres.

FIG. 1B NATURAL GROUND BETWEEN BED AND FULL SUPPLY LEVEL



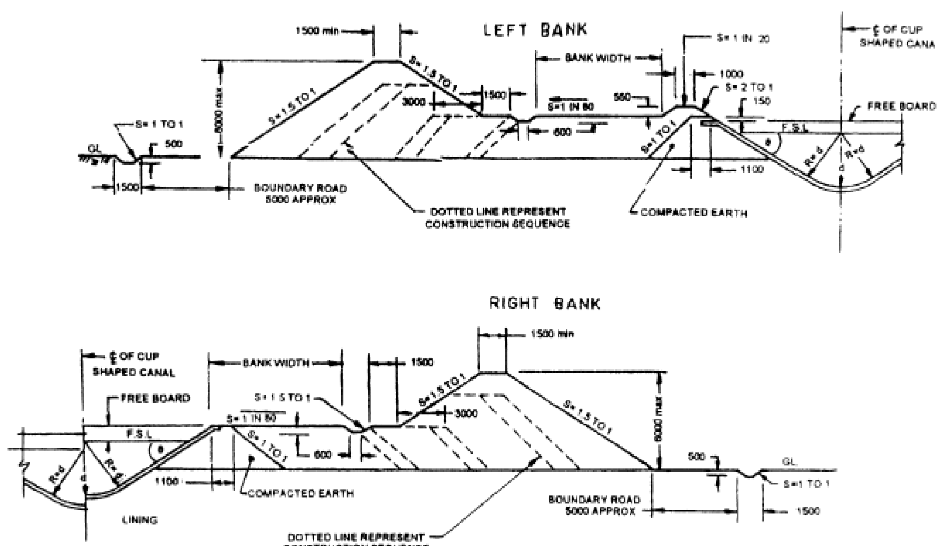
All dimensions in millimetres.

FIG. 1C NATURAL GROUND IS AVOVE TOP OF LINING



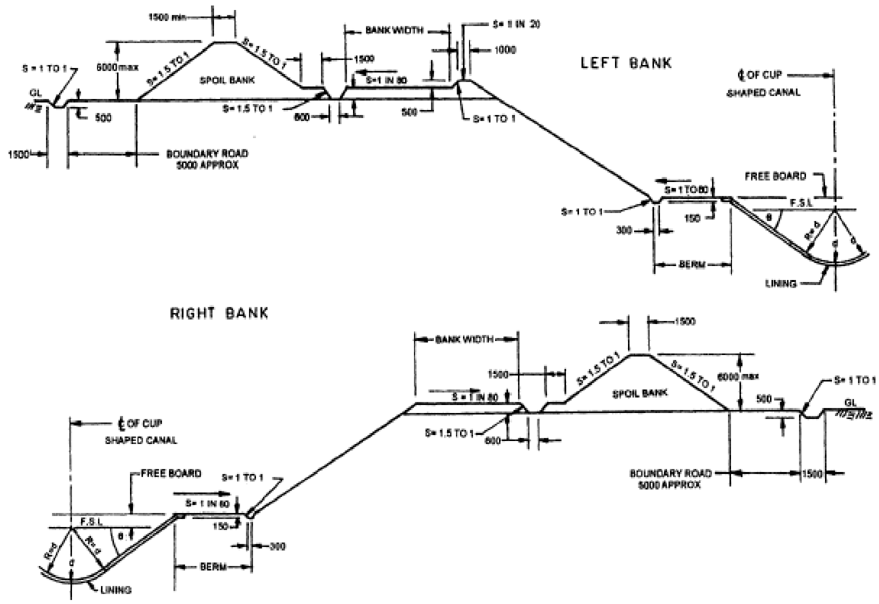
All dimensions in millimetres.

FIG. 2A NATURAL GROUND BELOW BED



All dimensions in millimetres.

FIG. 2B NATURAL GROUND BETWEEN BED AND FULL SUPPLY LEVEL



All dimension in millimetres.

FIG. 2C NATURAL GROUND IS AVOVE TOP OF LINING

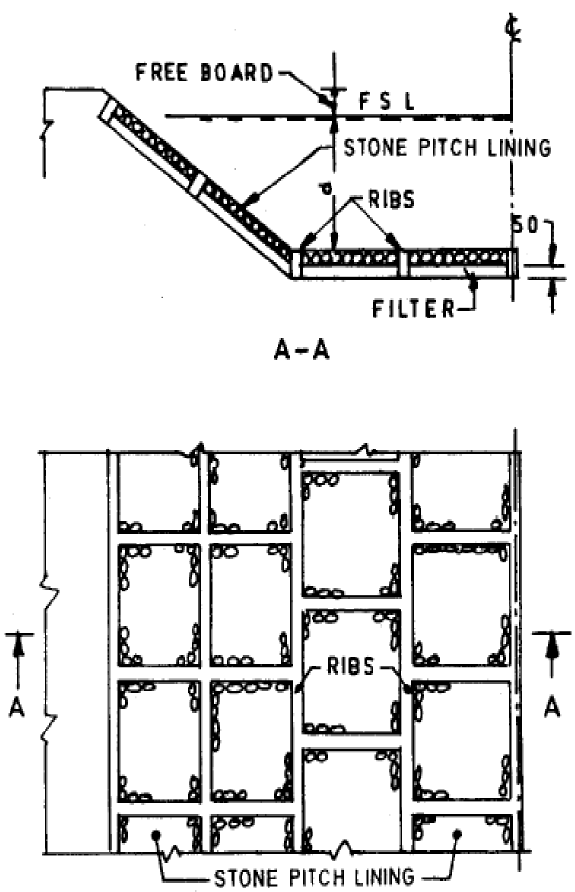
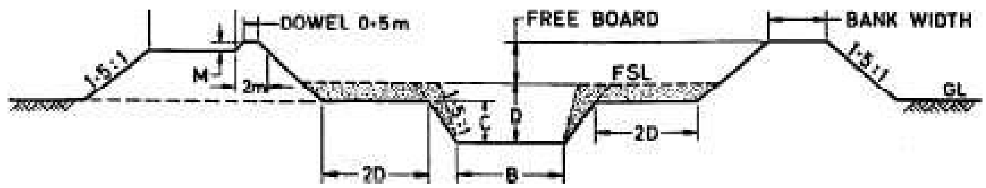
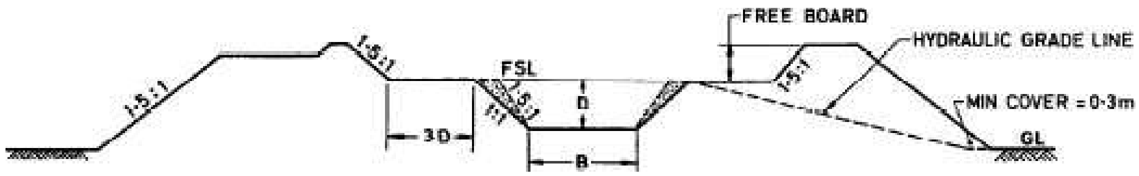


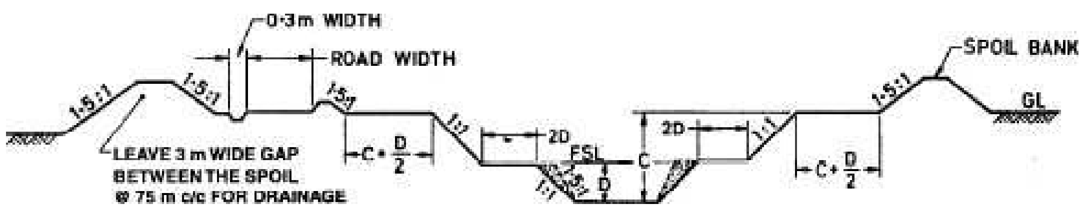
Fig 3 Stone pitched lining



4A TYPICAL SECTION OF CANAL PARTLY IN
CUTTING & PARTLY IN FILLING



4B TYPICAL SECTION OF CANAL WHOLLY IN FILLING

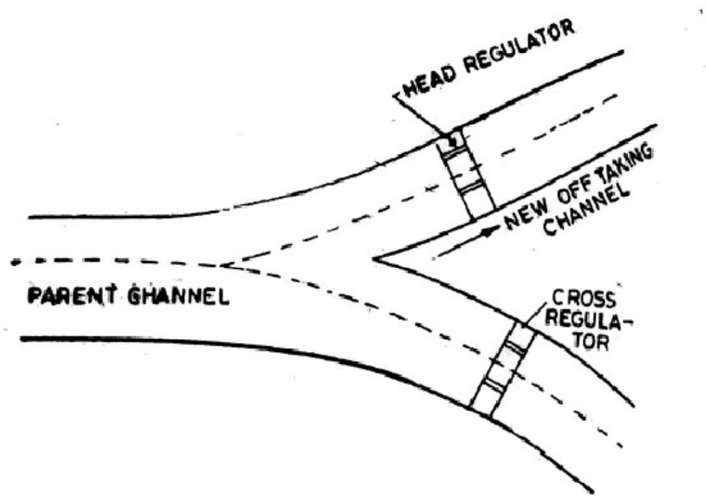


4C TYPICAL SECTION OF A CANAL WHOLLY IN CUTTING

Fig. 4 Typical Cross-Sections of Unlined Canals in Alluvial Soils

Design of Head and Cross Regulator

Head Regulator controls the supply of the off-taking channel while **Cross Regulator** controls the supply of the parent channel.



The **Head Regulator** or intake provided at the diversion structure/ at the head of the off-taking channel has the following functions to fulfill :

(i) Regulation of the supply of water into the canal or water conductor system for purposes of irrigation, hydel power generation, industrial or domestic water supply etc.
(ii) Prevention of high floods from entering into the canal
(iii) Control of the entry of silt into the canal.

A **Cross Regulator** in the main parent channel has the following main functions to perform:

i) To effectively control the entire canal irrigation system
ii) During the period of low flow in the main channel, it helps in the heading of water in the upstream of main channel and to feed the off-take channel to their full demand in rotation.
iii) Helps in absorbing fluctuations in various sections of the canal system and in preventing possibilities of breaches in the tail reaches
iv) Cross regulator is often combined with a road bridge and canal fall structure.

Hence, it is very important that the head/cross regulator is designed carefully for satisfactory hydraulic and structural performances since the success or otherwise of the project would depend on it also as it is the source at the head. The various aspects regarding the same are discussed below.

Layout:

From the point of view of sediment exclusion, the head regulator is generally aligned with its axis perpendicular to that of the diversion structure and located near the abutment. However, in some cased, it has been observed that keeping the axis of the head regulator oblique to the barrage/weir axis at an angle varying upto 110° or so gives better performance from sediment exclusion and smooth entry point of view. Hence aid of hydraulic model studies is generally taken for location and alignment of head regulator axis. The head regulator may be located on one or both the banks as per requirements.

While the location of the head regulator adjacent to the abutment of the diversion structure is preferred, it may not sometimes be possible to locate it there due to topographical feature such as hills, etc. In that case the head regulator may have to be sited upstream near the periphery of the pond, but very far from the main structure. If the discharge requirements are small, sometimes the head regulator or intake is provided in the form of an opening in the wing wall of the abutment, constructed at a suitable angle to the latter.

The head regulator could be constructed independent of the abutment separated from it by suitable joints and seals or it can be monolithic with it, say in the case of gravity type of abutments of the main structure and the head regulator, with a valley line forming in between. The abutments of the head regulator themselves can be separated from its floor by longitudinal joints and seals or they can be made monolithic with the raft floor of the head regulator and the whole structure designed as a trough section.

The regulation of the head regulator is provided usually by vertical lift gates. The gates can be a single or double or double or triple tiered one depending on the silt content of the river water and the purpose for which water is required to be diverted. The required discharge into the canal including about 10 to 15% of discharge for sediment extractors can be passed at pond level for which a gate controlled opening from the crest level to the pond level only is required to be provided. To reduce the uneconomical height of the gate right upto the high flood level due to higher flood level due to higher cost of the gates, heavier machinery to operate them and high level of operating platform required, breast walls are provided instead above pond level and upto the high flood level plus a little free board or upto the top of the abutments.

Usually a road bridge is provided across the head regulator for vehicular traffic or for inspection and would be suitably connected to the bridge across the main structure. For the operation of the gates, a working platform across the head regulator would also be provided. In the case of head regulators feeding hydel channels, usually trash racks are provided in front of the gates and stop logs to prevent floating debris from getting past the head regulators. Trash racks can be provided even in the case of irrigation canals if the floating debris are to be eliminated as much as possible.

Often times, the waterway of the head regulator may be more than the bed width of the canal downstream. In such cases, suitable transition would need to be provided beyond the downstream end of the head regulator to reach the normal section of the canal.

Sometimes, water may have to be diverted from the river to serve two main canals in different territories. It has to be analysed in such cases to determine whether a single head regulator with the combined discharging capacity and the two canals branching off after some common length could be had or two independent head regulators with two separate canals can be had. Apart from economical considerations the operating conveniences including disputes likely to arise should also be considered. If two separate head regulators are to be provided, it would be desirable to determine their best locations and alignments with the help of model studies so that their hydraulic performances are not impaired.

Hydraulic Design:

In the hydraulic design of the head regulator, the following items are decided : *Crest level, waterway, profile of the floor & energy dissipation arrangement are decided.*

Crest level : The crest level and waterway of a head regulator are inter-related. For prevention of silt entry into the canal, the crest level should normally be kept about 1.2 to 1.5 meter above the crest level of undersluices if silt excluders are not provided and it would be about 1.75 to 2.5 m above, if silt excluders are provided. The crest level could be kept at the top level of the silt excluder tunnels or about 0.5 m higher depending on the silt charge.

Waterway *:* The waterway should be adequate to pass the required discharge through the head regulator without difficulty. In important structures, a stand bye bay also could be provided to take care of any bay under repairs. The required waterway is calculated by using

$$Q = C^d L_o H^3/^2$$

Where, C_d = Co-efficient of discharge

L_o = Effective Width of waterway

H = Head over the crest

In case of rectangular openings of intake, sometimes, submerged orifice formula could be used.

The spans of head regulators usually vary from 5 to 10 m. Hence as per the required waterway calculated, the number of spans and their widths can be decided. As already mentioned under layout, stand bye bays could be provided wherever needed.

Profile of the Floor : While finalizing the profile of the floor of the head regulator, the upstream and downstream cut off levels upstream floor level and length, downstream glacis, cistern level and length, and end sill level and length would be determined.

When the head regulator is located close to the under sluice bays of the main structure, the upstream cut off/sheet pile would be taken down to the same level as that of the under sluice bays. If it is located away from the main structure, it has to be taken down to the level governed by the usual considerations of scour on the upstream. The downstream cut off/sheet pile level would be governed by the consideration of exit gradient and scour.

The upstream floor is usually kept horizontal at the crest level. The upstream floor would have to accommodate the grooves for trash rack bottom, stoplog and gates. It should also be fixed from the overall length required for a safe exit gradient which should be checked with canal closed and high flood on the upstream. The downstream glacis is usually kept as 3(H) : 1 (V) as for the main structure.

Energy Dissipation arrangement : Energy Dissipation is achieved through formation of hydraulic jump under different discharge conditions. For various gate openings with Pond level on the upstream, the discharge through the head regulator and the corresponding water level in the canal have to be worked out. From these values, the cistern levels and lengths would have to be calculated and the governing values adopted for the profile. Additional energy dissipating devices such as chute blocks, friction blocks, end sill or dentate end still, etc., could also be provided wherever necessary. For head regulators with small discharging capacities, additional energy dissipating devices except an end still may not be necessary.

Structural Designs:

The structural designs of a head regulator are more or less similar to those of the main structure. As the canal is usually completely closed when the highest flood is passing in the river, it provides the worst static condition and the floor should be able to resist uplift pressures under this condition. These pressures are usually high and a gravity type of floor may not be economical. A concrete raft floor may prove to be economical in such places.

The breast wall of the head regulator generally consists of two parts viz., vertical stem and horizontal beam. The two parts can be constructed monolithic or separated by a joint and provided with a seal in between. The vertical stem would be designed as a slab spanning between the piers or pier and abutment, fixed at the two ends and loaded by the horizontal water thrust on the upstream side. The horizontal beam would be designed for bending in both the horizontal and vertical directions. The loads to be taken care of in the analysis include the self weight, horizontal water thrust on the upstream, uplift acting below the beam and self weight of the vertical stem transferred at the beam eccentrically. Since the vertical stem is fixed to the piers and abutments, full weight of the stem may not transferred to the beam. A fair proportion at the discretion of the designer may be taken as the transferred load. When the stem and beam are monolithic, the breast wall has to be checked for torsion also caused by the water trust. The area of the second stage concrete for the embedded parts should be neglected while analyzing the beams.

Regulation of Gates:

Wherever two tier or three tier gates are provided, for normal conditions when the silt charge is not much, the lower most tier of gates would be in operation. When the silt charge of water increases, the lower most tier would be kept in closed position and the other tiers of gates operated.

Wherever trash racks are provided to prevent the floating debris from entering through the head regulator, it is necessary that they are cleaned regularly so that the openings are not allowed to be chocked. Apart from increasing the differential head over the trash rack, the required discharge through the head regulator cannot be fed if they are allowed to get clogged.

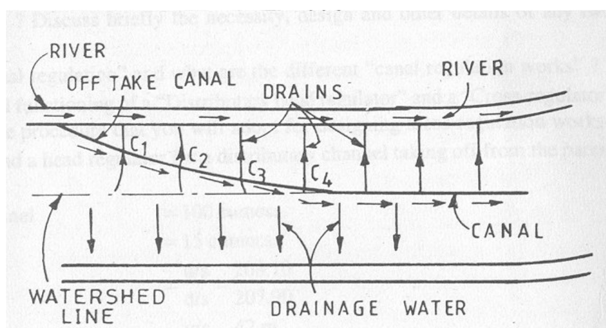
Instructions regarding the closure of head regulator gates beyond a permissible silt charge of diverted water should be strictly followed.

Chapter - 11

Cross Drainage Works
– types & suitability criteria &and Design considerations

A cross drainage work is a structure which is constructed at the crossing of a canal and a natural drain, so as to dispose of drainage water without interrupting the continuous canal supplies.

In whatever way the canal is aligned, such cross drainage works generally become unavoidable. In order to reduce the cross drainage works, the artificial canals are generally aligned along the ridge line called water-shed. When once the canal reaches the watershed line, cross drainage works are generally not required, unless the canal alignment is deviated from the watershed line. However, before the watershed is reached, the canal which takes off from the river has to cross a number of drains, which move from the watershed towards the river, as shown in Fig. below. At all such crossings c_1, c_2, c_3, c_4, etc. cross drainage works are required.



A cross drainage work is generally a costly construction and must be avoided as far as possible. Since a watershed canal crosses minimum number of drains, such an alignment is preferred to a contour canal which crosses maximum number of drains. The number of cross drainage works may also be reduced by diverting one drain into another and by changing the alignment of the canal such that it crosses below the junction of two drains.

Cross drainage works can be classified under the three broad categories, based on the type of the structure to negotiate a canal over, below or at the same level of the drainage channel.

1. Structures for Canal Over a Natural Drainage Channel

The structures falling under this category are aqueducts, syphon aqueducts and culverts. Maintenance of structures in this category is relatively more convenient, as these are generally above the ground and hence open for inspection.

2. Structures for Canal Underneath a Natural Drainage Channel

The structures falling under this category are superpas-sages and syphons including well syphons. In case of syphons the maintenance is difficult as these run below the natural drainage channel and are, therefore, not easily accessible to inspection.

3. Structures for Canal Crossing a Natural Drainage Channel at the Same Level

Structures falling under this category are level crossings and inlets, with or without escapes.

SELECTION OF THE TYPE OF CROSS DRAINAGE WORK:

While aligning the canal, the type of cross drainage work envisaged should always be kept in view. The economics of various types of cross drainage works *vis-a-vis* alternative alignments should be considered before deciding upon the site and type of crossing. As a general guide, for deciding upon the type of the cross drainage work, important considerations are as given below:

a) Full supply level and functions of canal — *vis-a-vis* high flood level of the drainage channel,
b) Topography of terrain,
c) Regime of the stream,
d) Foundation strata,
e) Dewatering requirements,
f) Ratio of design flood to be provided in drainage channel to the discharge in the canal, and
g) Envisaged head loss.

Full Supply Levels of Canal vis-a-vis High Flood Level (HFL) of Draiiiage Channel:

The choice of any particular type of cross drainage work is dependent on the high flood level (HFL) in the drainage channel to be negotiated. Aqueducts are generally proposed when the bed level of canal is well above the HFL of the drainage channel. Superpassages are generally proposed when the full supply level (FSL) of the canal is well below the bed level of the drainage channel. When the bed level of the canal is at, or below, the HFL of the drainage channel, the depression of the bed of the drainage channel is often a more economical proposal and in such cases syphon aqueducts may be considered.

Topography of Terrain:

Detailed examination of the topography of the terrain is essential to locate a stable reach of the drainage channel with good foundations permitting, preferably, a right-angle crossing. Topography of the terrain may also permit diversion of one channel into another and locating the cross drainage work below the confluence of the two channel for greater economy.

Regime of Drainage Channel:

The regime of a drainage channel requires careful examination. For drainage channel carrying high sediment charges or drift materials, the possibility of choking up of the syphon and the effect of fluming of the drainage channel should be kept in view.

Foundation Strata:

The selection of the most suitable site and a good design, for any cross drainage work is intimately related to the engineering properties of the foundation sub-strata at various alternative sites. These properties have, therefore, to be determined by site explorations. Where an alternative site, meeting other criteria, is available, the final choice would obviously depend on the location where the sub-strata available close to the bed of the stream is firm.

Dewatering Requirements:

In the execution of foundation works for cross drainage structures dewatering of foundations may pose serious problems. An accurate estimate of the cost and procedure of dewatering requires to be carefully worked out when designs involve laying of foundations below the ground water table.

Ratio of Design Flood in Drainage Channel to the Discharge in Canal:

Negotiating a canal below the drainage channel is generally more difficult and involves more head loss. However, if the topography and other features warrant a choice to be made between canal syphon and syphon aqueduct, then canal syphon may be preferred, only if the ratio of canal discharge to the design flood is substantially low.

Envisaged Head Loss:

The choice of any particular type of cross drainage work is also dependent on the head loss that can be permitted in the canal. Whereas higher head loss can throw some area out of command, restriction on head loss may necessitate provision of wider sections making tie structure costly.

DATA REQUIREMENT

General:

a) A location map for the work with results of subsurface exploration conducted at site, cross sections of the stream, upstream and downstream of the proposed site, should be prepared.

b) An index map to a suitable scale showing the recommended location of the cross drainage structure, the alternative sites of crossings investigated and rejected, the existing communications, the general topography of the country and the important habitations in the vicinity.

c) A catchment area map to a suitable scale, with contour markings at suitable intervals showing the main drainage channel from its sources together with all its tributaries.

d) A detailed survey plan of the drainage channel to suitable scale showing important topographical features extending considerable distances, downstream and upstream, of the proposed site of crossing and on either of its banks.

e) The other requirements in plan are:
   - i) Reference to the position of the bench-mark used as datum and reduced level.
   - ii) The locations of the various trial pits and/or borings with their identification numbers;
   - iv) The contour of the drainage channel at intervals between 0.5 m to 1.5 m depending upon the terrain. This interval may be greater in mountainous regions;
   - v) The direction of flow of water.
   - vi) The angle of direction of crossing.

f) A cross section of the drainage channel at the proposed site of the crossing to appropriate vertical and horizontal scales indicating the following information:
   - i) Cross section covering the bed and banks of the channel portion and the ground levels beyond the banks covering the entire flood plane
   - ii) Nature of the soil in bed, banks and approaches, with trial pit or bore-hole sections showing the levels and natures of the various strata down to stratum suitable from foundation considerations.
   - iii) Low-water level.
   - iv) Maximum flood level.

g) Longitudinal section of the drainage channel covering a reasonable reach to suitable scale, showing the location of the cross drainage work, with levels of the observed flood, the low water and the bed evels at suitably spaced intervals along the line of the deep water channel.

h) A note giving the salient design features of structures existing upstream or downstream of the proposed site.

Hydraulic data:

<u>Canal:</u>

1. Full supply discharge, *Q;*
2. Bed width;
3. Full supply depth;
4. Water surface slope;
5. Bed level;
6. Bed slope;
7. Full supply level;
8. Top of bank level;
9. Cross section of canal showing Natural Ground Level;

10. Subsoil water level; and
11. Nature pf bed material and value of 'n' (rugosity coefficient in Manning's formula).

Drainage Channel:

1. Extent and nature of drainage area (catchment area);
2. Maximum annual rainfall and the period (years) of data;
3. Maximum intensity of rainfall with year;
4. Maximum observed flood discharge at the site;
5. Maximum flood level;
6. Water surface slope;
7. Site plan of proposed crossing including contours;
8. Log of borehole or trial pit data;
9. Type of bed load of drainage channel;
10. Longitudinal section of the stream for suitable distance upstream and downstream of the canal depending upon site conditions;
11. Cross section of the drainage channel for a distance 100 m to 300 m upstream and downstream, at intervals of 10 m to 50 m;
12. Waterways provided in road and railway bridges or other hydraulic structures on the drainage channel;
13. Spring water level at the crossing site in May and October; and
14. Silt factor.

DESIGN FLOOD FOR DRAINAGE CHANNEL:

Design flood for drainage channel to be adopted for cross drainage works should depend upon the size of the canal, size of the drainage channel and location of the cross drainage. A very long canal, crossing a drainage channel in the initial reach, damage to which is likely to affect the canal supplies over a large area and for a long period, should be given proper weightage.

To safeguard against unforeseen nature of flood intensities the foundation of the cross drainage structure should be checked for a check flood discharge of value twenty percent higher than the design flood.

HYDRAULIC DESIGN ASPECTS:

Waterway:

Waterway-for a cross drainage work is fixed from hydraulic and economic considerations with particular reference to:

a) design flood,
b) topography of the site,
c) existing and proposed section and slope of the drainage channel in the vicinity of the crossing,
d) permissible afflux, and
e) construction and maintenance aspects.

In plains, the drainage channels are generally in alluvium and the waterway usually provided in works without rigid floor is about sixty to eighty percent of the perimeter, given by Lacey's formula:

The value of wetted perimeter obtained is the total waterway between the two faces of the abutments.
In works with rigid floors, however, waterway' can be further flumed within the permissible limits of velocity negotiated through the available vents.

For mountainous terrains with flashy flows, the waterway is provided within the width of the existing stream. Where the slope of the natural drainage channel is quite steep suitable methods may be adopted to bring the velocity within the desired limits.

The minimum dimension of openings should be such as to permit, as far as possible, manual clearing of deposits therein.

Clearance for Aqueducts:

The clearance will depend upon the relative levels of the canal bed and high flood level of the drainage channel. Values given in IS 7784 (Part 1) : 1993 are suggested as suitable minimum clearances (taking into account allowable afflux) for purposes of design, where available.

In the case of drainage channels, where a bed rise due to progressive silting is anticipated, the permissible clearance should be increased to allow for such aggradations depending upon the extent of silting.

Free Board:

On aqueduct structures, the free board is reckoned from the high Hood level (including afflux) in case of drainage channel and from the full supply level in case of canals, to the formation level of guide bank or canal embankment. The free board should not be less than 900 mm. Wherever heavy wave actions are anticipated, the free board should be suitably increased.

Afflux:

The afflux to be adopted in the design should be that which would correspond to the design flood.
The afflux should be restricted to such a value that the resulting velocity does not cause serious bed scour in the drainage or does not create submergence which cannot be permitted.

Depth of Scour:

The mean depth of scour (dm) in metres below the check/high flood level may be calculated. The value obtained should take into account any concentration of flow through a portion of the waterway assessed from the study of the cross section of the drainage channel. The maximum depth of scour below the Highest Flood Level (H.F.L.)

and at obstructions and configurations of the channel should be estimated from the value of 'dm' as laid down in IS 7784 (Part I) : 1993.

Loss of Head (Energy Loss):

When water flows through any structure there are head losses due to various factors.
The total loss of head occurring for a flow is represented as the sum of these losses as applicable. Thus, if the total loss of head is denoted by *H* then :

$H = h_1 + h_2 + h_3 + h_4$

Where,
h_1 = losses at the inlet and outlet.
h_2 = losses at elbows or bends.
h_3 = losses due to transitions.
h_4 = losses due to skin friction

Transition Walls:

Transition walls as seen in plan, should at their ends, turn nearly at right angles to the flow in the channel and should extend for a minimum length of 0.6 in into the earth bank. Suitable pitching may be provided to the slopes, beyond the transition end.
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Flumiug Ratio:

Except when dictated by conditions particular to a specific structure, a fluming ratio less than seventy percent may not be adopted. For the purpose of com puting the fluming ratio of canal, the width at mid depth may be taken as one hundred percent In drainage channel. When the course is undefined, a fluming ratio from seventy to ninety percent of the Lacey's waterway may be adopted.

Structure and Earth Work Connection:

The earth mass in vicinity of the rigid structure is the connection between rigid structure and flexible earthwork. The rigid structure is mm settling, relative to the earthwork. The deflections, settlements and other movements in the rigid structure are comparatively very small. The rigid structure may consist of masonry, PCC, RCC, etc. The connection between rigid structures and earthwork is to be designed so as to reduce the differential settlement, and to avoid the possibility of formation of a separation (cleavage) between the two. The condition of connection between tbe rigid structure and the earth work affects the seepage, creep coefficient and piping and thus affects the stability of the earthwork.

***Exit gradient* :**

Exit gradient of seepage water should be limited within the permissible limit.Adequate foundation depth or cut-off or curtain walls maybe provided of suitable depth so as to get safe exit gradient, which may be worked out in accordance with Khosla’s theory.

Foundation:

Foundations of a cross drainage work should be designed to satisfy the requirements of allowable bearing capacity of the foundation strata under critical toads including positive pressure conditions (i.e. no uplift or tension), seismic effects, anticipated scour and settlement. As far as possible, the foundation should bear on homogeneous, undisturbed and uniform sub-grade of fairly dense type. Where foundations have to be provided on sub-grade of different types suitable joints should be provided to avoid cracks due to differential settlement.

The foundations should be taken sufficiently deep lo secure firm strata from considerations of settlement, overall stability and avoidance of undermining due to erosion. The depth of foundation of various members should be such that these are safe against scour or are protected against it.

Miscellaneous details:

Waterstops:

Waterstops, also referred to as water seals, are generally of three types, namely :
(a) rubber water seals.
(b) metal water seals and
(c) synthetic material seals.

The waterstops are used in and across all joints where leakages are detrimental to structural safety or the water needs are to be conserved. The locations where waterstops are provided in various types of cross drainage works are as below:

Aqueduct — In R.C.C. through side walls and bottom slab over each pier in a continuous length and at the junction of transition and R.C.C. trough, both in the floor and wing walls.
Syphon—At expansion joints and at the junction of each of the sloping limbs in a continuous form and at the junction of the transition walls and floors with the barrel, both at the entry and exit in a continuous form.
Superpassage — At the junction between the drainage trough wing walls, namely, trough wall of R.C.C. and wing wall of masonry and all the expansion joints in a continuous length.

Weep Holes:

Weep holes are small openings in the retaining walls, like wings (i.e. transitions of natural stream). These are to facilitate the drainage of backfills and avoid build up of pressure. Weep holes may be provided above the flow net line of zero water pressure, under the condition of canal flowing full and natural stream with lowest annual flow. Weep holes should have filters with graded material suitably provided to avoid piping of earth fill behind the wall and also to avoid choking of the holes.

Typical plan and elevation of three different types of cross-drainage structure are given below.

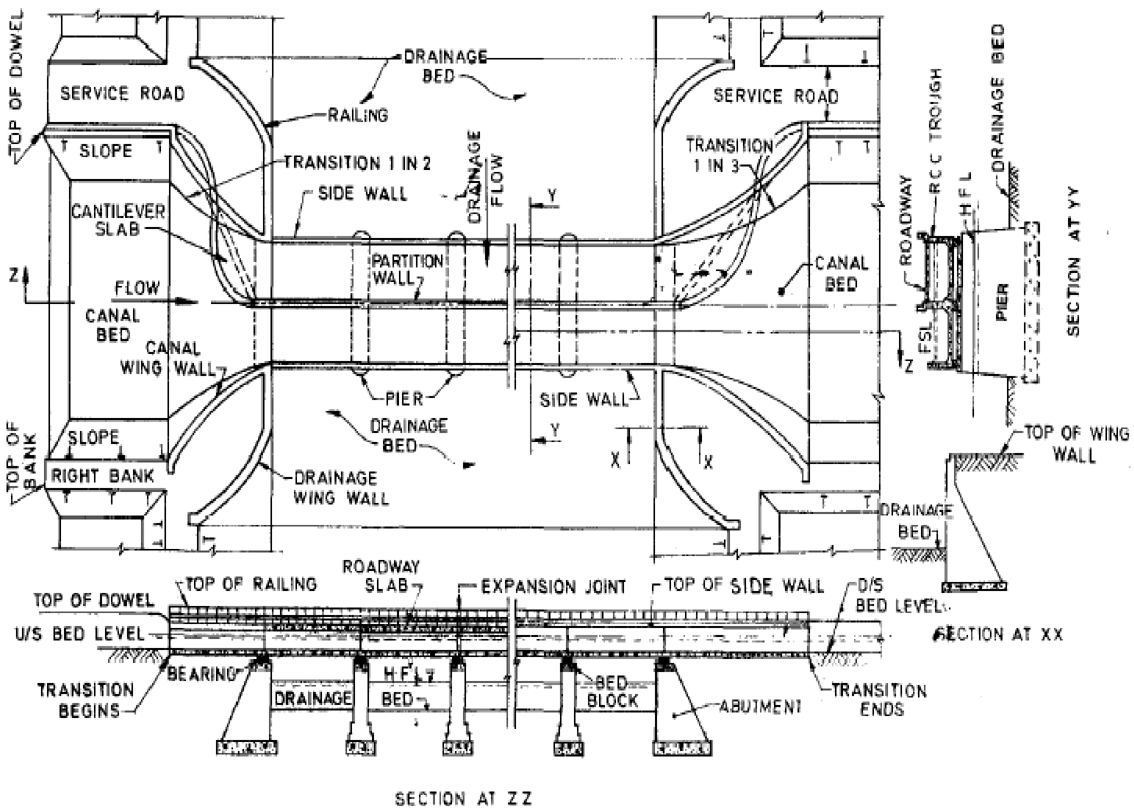


FIG. 1 TYPICAL PLAN AND SECTION OF AQUEDUCT

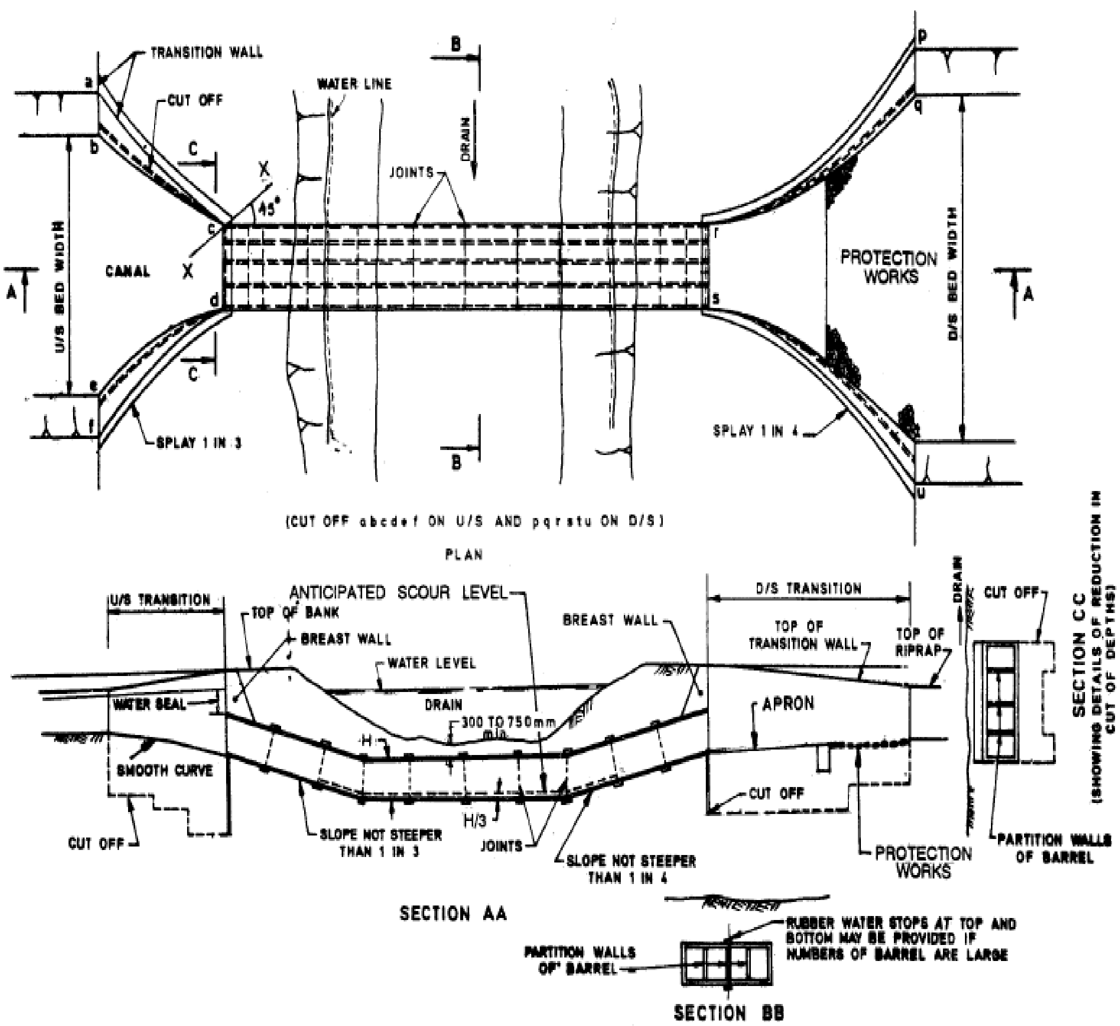


FIG. 1 PROFILE OF TYPICAL CANAL SYPHON

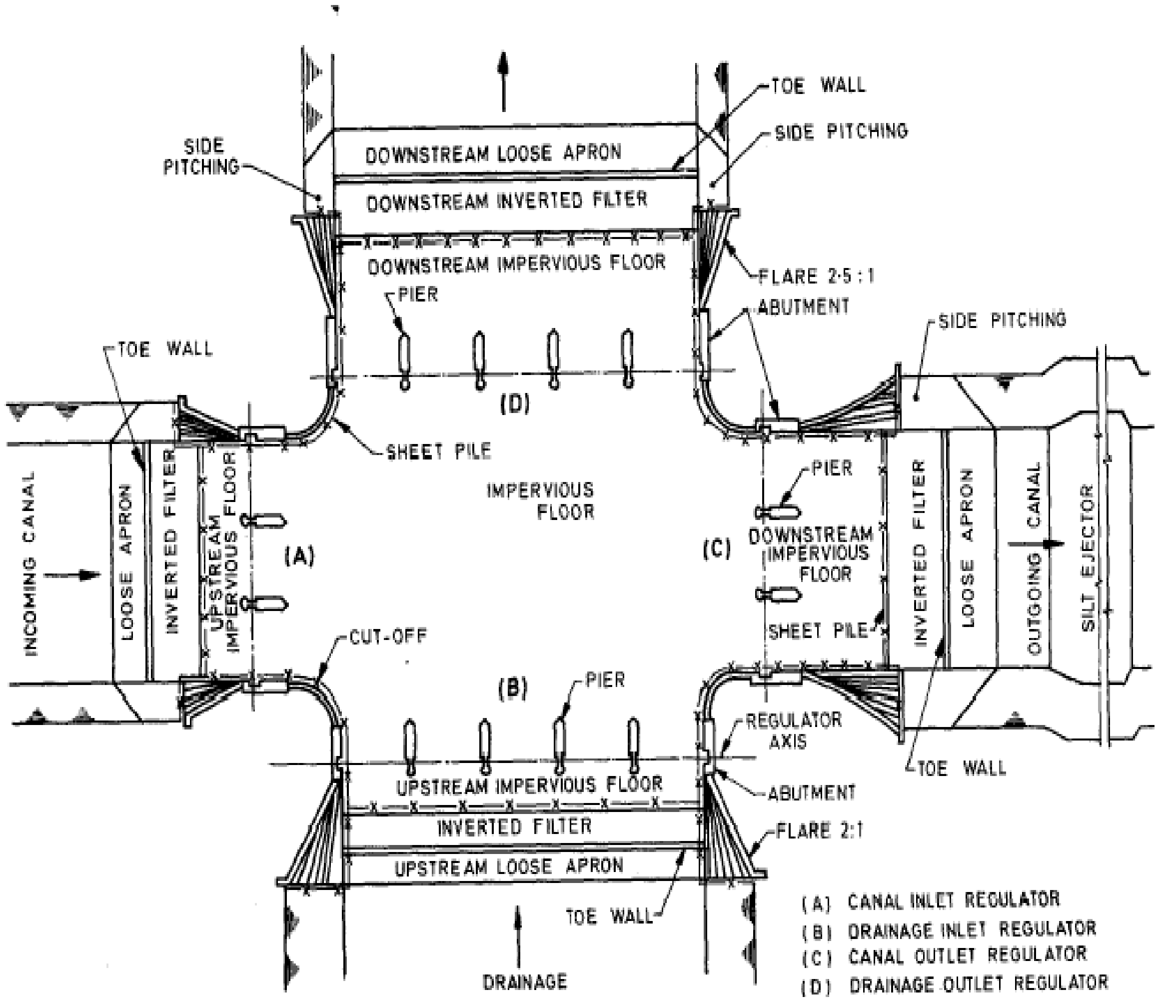


FIG. 1 TYPICAL LAYOUT PLAN OF LEVEL CROSSING

Chapter - 12

Design of Canal Automation System

ABSTRACT

Canal automation is the application of automatic devices or logic to assist in the operation of canal systems. It represents a wide array of technologies, ranging from simple mechanical devices to complete computerized control. Because the technology is so diverse, canal automation means different things to different people. It is applied to different aspects of the overall operation of a canal system, using different hardware software and with different control objectives.

1. Introduction:

Canal automation is the application of automatic devices or logic to assist in the operation of canal systems. It represents a wide array of technologies, ranging from simple mechanical devices to complete computerized control. Because the technology is so diverse, canal automation means different things to different people. It is applied to different aspects of the overall operation of a canal system, using different hardware/software and with different control.

But why do we need to consider automation of canals? Many irrigation schemes are not producing expected returns, are experiencing water quality problems, or are under pressure to reduce irrigation diversions. Many of the problems associated with these irrigation schemes are linked, at least partially, to ineffective operation of the water distribution system, typically a canal network. Rehabilitation of such systems suggests the need not to rebuild them as originally designed, but to rebuild them so that they can be operated effectively.

In the future, canal distribution systems will have to allow more flexibility in operations to meet the needs of users. This often implies more flexible delivery rules, which some systems might have difficulty satisfying. In many cases, improvements can be made with simple structural changes or with improved manual operating procedures, and these should be examined first. In other cases, one of a variety of canal automation technologies can be useful for improving delivery performance, provided they are appropriately applied – that is, with the goal of improved operations.

Unfortunately, defining the benefits of improved flexibility has been an elusive target – it is hard to quantify. Thus, it has been difficult to determine the level of flexibility that is economically justified, thereby making the cost/benefit justification for canal automation technology uncertain. Justification for automation seems to require a shift in philosophy from a least-cost-delivery to a customer-service orientation. For a given level of flexibility, there may be situations where automation replaces labor and is therefore cost effective. There may be other situations where automation actually improves performance, but these have not been well documented.

A canal's response to changes in flow is influenced by both the properties of the hydraulic structures in use and by the hydraulic properties of the canal pool. The controllability of a canal is influenced by both. The properties of hydraulic structures can often be modified relatively easily. However, the hydraulics of the pools are often be modified relatively easily. However, the hydraulics of the pools are often very difficult to alter, and thus they impose a greater limitation on the performance of some control schemes. Some canals are inherently difficult to control, whereas others might be controllable with nearly any method.

2. MODERNIZATION:

Definition of modernization given by FAO is that - Modernization is a process of technical and managerial upgrading (as opposed to mere rehabilitation) of irrigation schemes combined with institutional reforms, with the objective to improve resource utilisation (labour, water, economic, environmental) and water delivery service to farmers. At the farm level, there is a great value and a need for a dependable water supply that is flexible in frequency, rate, and duration, and that will permit the use of state-of-the–art equipment and methods.

Modernization differs from rehabilitation, which simply returns a deteriorated project or structures to their original new state. It is common to misunderstand modernisation as consisting of simple actions like lining of canals, establishing WUAs and experimenting with computer programs rather than a whole new integrated thought/design/operation process which targets good water delivery service and good water management throughout the project. A modern irrigation design is the result of a thought process that selects the configuration and the physical components in light of a well-defined and realistic operation plan that is based on service concept. A modern irrigation design is not defined by specific hardware components and control logic. Advanced concepts of hydraulic engineering, irrigation engineering, agronomy and social science should be used to reach at the most simple and workable solution. Modernization – which is custom-designed to fit local needs and circumstances is essential if irrigation systems and irrigated agriculture are to be sustainable.

CANAL MODERNIZATION NEED:

The need for modernization of canals has developed because of a number of factors, including the updating of systems objectives and purposes, changes in the demand patterns, increased costs of operation and maintenance, the need for reduced operational spillage and mismatches between demand and delivery. Most importantly, there is an increased awareness of the need for delivery flexibility in order to maximize on-farm yield, and for water conservation through more effective on-farm use.

From the perspective of the on-farm water users, these primary criteria judge the flexibility of water supply systems, namely frequency, rate, and duration. The frequency at which water is available at the farm needs to be related to many items, each often resulting in a different answer and affecting yield and financial return. Rate of flow available at the farm affects labour and irrigation efficiency, irrigation methods adaptability, and economics of operation. The upgrading of the scheduling capability through the use of techniques such as pipelines, increased capacities, or automation, can almost always permit improved on-farm management with increased net returns capable of repaying both the on-farm and project costs.

3. SCADA:

Supervisory control and data acquisition (SCADA) is a system that allows an operator to monitor and control processes that are remotely located. There are many processes that use SCADA systems: hydroelectric, water distribution and treatment utilities, natural gas, etc. SCADA systems allow remote sites to communicate with a control facility and provide the necessary data to control processes. For many of its uses, SCADA provides an economic advantage. As the distance and inaccessibility to remote sites increases, SCADA becomes a better alternative than an operator or repairman's visiting the site for adjustments and inspections. Distance and accessibility are two major factors for implementing SCADA systems.

Evolution - SCADA vendors release one major version and one to two additional minor versions once per year. These products evolve thus very rapidly so as to take advantage of new market opportunities, to meet new requirements of their customers and to take advantage of new technologies.

3.1 NEED FOR SCADA

Canal water distribution utilities use some type of remote monitoring and/or control system to aid in proper and efficient operation. Control systems designed to monitor a process are referred to as data acquisition systems (DAS). If the system also allows remote control to occur based upon the acquired data, it is referred to as a supervisory control and data acquisition system, or SCADA system.

Dynamic processes like canal water distribution require continuous process monitoring and control to achieve minimum standards of cost, flow regulations and safety. Majority of irrigation departments still rely heavily on expensive periodic visits by maintenance people to check on the status of various cross-regulators at canal flow distribution points. A SCADA remote monitoring and control system can reduce the frequency of these visits substantially. Typically configured to consolidate operations at a central office, personnel can monitor and control virtually any aspect of the operations. A SCADA system can provide information in a real-time environment that identifies problems as they occur and can take corrective action when assistance is needed. Proper monitoring of canal hydraulic structures can maintain operations at an optimal level by identifying and correcting problems before they turn into significant system failures. Avoiding major problems are more important as both central and state governments increase the adverse economic consequences of improper discharges.

SCADA operates on the premise that better control and distribution of water in a canal network results in improved performance by providing water in a more adequate, timely and reliable manner to suit the needs and expectations of the water consuming community. For a water distribution network, the common objectives of a SCADA system are to do the following:

- Monitor the system
- Obtain control over the system and ensure that required performance is always achieved
- Reduce operational staffing levels through automation or by operating a system from a single central location
- Store data on the behavior of a system and therefore achieve full compliance with mandatory reporting requirements for any regulatory agency
- Provide information on the performance of the system and establish effective asset management procedures for the system
- Establish efficient operation of the system by minimizing the need for routine visits to remote sites and potentially reduce power consumption during pumping operations through operational optimization
- Provide a control system that will enable operating objectives to be set and achieved
- Provide an alarm system that will allow faults to be diagnosed from a central point, thus allowing field repair trips to be made by suitably qualified staff to correct the given fault condition and to avoid incidents that may be damaging to the environment

4. NUMERICAL SIMULATION OF CANAL

Unsteady-flow mathematical models for irrigation canals networks are needed to examine the transient nature and their control during operations of canal. These models can be applied to canal networks for real-time control, gate scheduling analysis of operations and training. The mathematical model can simulate the system under different schemes of operation and provide information regarding water level fluctuations.

The mathematical model consists of set of instructions following the algorithm of governing equations and with appropriate boundary conditions. Though simulation models are an abstraction of reality nevertheless it gives a realistic solution with reasonable accuracy. The adequacy of a model must therefore be seen in relation to its purpose and not from an absolute point of view. In the light of this, it is desirable to verify the results obtained from simulation model either with the field data of prototype operation or with the results obtained from other proven model.

Use of the mathematical model permits application of various principles of control theory to automation of water systems. With the help of mathematical model various methods of canal operation can be evaluated, which will reflect the conditions endangering the structural stability of the canal system.

The use of simulation models to improve canal control strategies is the motivation behind studies on canal control. Mathematical models for unsteady open channel flows are commercially available for around three decades, which can be used to study the control of irrigation canals. There are a number of tradeoffs between simplicity and functionality. All these models present difficulties and have limitations. The hope is to provide an introduction on the next generation of unsteady-flow canal simulation models.

BASIC EQUATIONS

Unsteady flow in open channels is based upon one-dimensional Saint Venant equations for conservation of mass and momentum. The equations have five key fundamental assumptions:

(i) The flow is one dimensional – i.e. the velocity is uniform in a cross-section and the free surface profile is horizontal.
(ii) The streamline curvature is very small and the vertical fluid accelerations are very negligible; as a result the pressure distribution is hydrostatic.
(iii) The flow resistance and turbulent losses are same as for a steady uniform flow for the same depth and velocity, regardless of trends of depths.
(iv) The bed slope is small enough to satisfy following approximations: $\cos\theta \approx 1$ and $\sin\theta \approx \tan\theta$.
(v) The water density is constant.

These assumptions are valid for any channel cross-section shape. The conservation of momentum and conservation of energy are equivalent if the two relevant variables (e.g. velocity and water level) are continuous functions. At a discontinuity (e.g. hydraulic jump), the equivalence turns invalid. Unsteady flow equations must be based upon the continuity and momentum principles, which are applicable to both the continuous and discontinuous flow situations.

The Saint Venant equation:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad \text{Continuity equation}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}(V^2 A) + gA\frac{\partial d}{\partial x} = gA(S_0 - S_f) \qquad \text{Momentum equation}$$

Where $Q = VA$ = Discharge
V = Flow velocity & A = Flow cross-sectional area
d = Water depth
S_o = Bed slope
S_f = Friction slope

These are hyperbolic nonlinear partial differential equations that cannot be solved analytically, except for a few simplified conditions. Therefore all unsteady simulation programs solve the governing equations numerically. Early models of unsteady canal flow used the method of characteristics.

5. Control Systems vs Control Algorithms:

Many canal control methods and algorithms have been developed, but only some of them are being used on operating canal projects. These algorithms are categorized as implicit algorithms in self-regulating gates, local automatic feedback controllers, and supervisory control algorithms.

Facing the conflict between water supply and demand, proper water management is absolutely necessary. Hence, more accurate and flexible irrigation systems are required i.e. the actual water supply matches the desired supply. In order to fulfill the two functions of an irrigation canal, all the infrastructures, like check and lateral gates or weirs, pumps and power stations, should be manipulated properly to efficiently transport large amount of water. In practice, considerable amount of water is wasted due to the inaccurate manual operation or the lack of control. Therefore, it is useful to apply automatic control to those check structures to control the water level or flow and efficiently distribute the incoming water, finally, minimize the water losses. Several control methods can be applied to the canal, like classic Feedback Control, Feedforward Control and more advanced Model Predictive Control (MPC). They have different applications to different systems depending on the actual situations.

An algorithm is a step-by-step procedure for solving a problem or accomplishing some end. A canal control algorithm is the logical procedure that processes input, such as water levels, and outputs a control action, such as gate movement. Typically, control algorithms are expressed as a series of mathematical equations that are incorporated into software and implemented using computers. A control system can include both hardware and software. Canal control systems may include sensors, communication equipment, power supply, electromechanical devices, and human interface equipment. Many existing control systems do not employ a control algorithm, because human operators provide the logic and decisions required for control actions (e.g., gate movements and flow changes).

Classic control types:

Classic control types are feed forward control, feedback control and their combination. Control variables, like water level, discharge and volume, are commonly used in the irrigation canal control. The following descriptions will present all these different methods and their comparisons.

Feedforward control:

Feedforward control is also called open-loop control. The control action is independent to the output, but it is calculated from the disturbances, the target variables and the process simulation. In the irrigation canals, disturbances can be the incoming flow from the most upstream, and the farmer offtakes along the canal. If these disturbances are already known in advance or can be anticipated, the feedforward control can improve the control performance considerably. It can compensate for the time delay, especially in those long canals. But in reality, the single feedforward control is not so practical, because of the change in the irrigation system or the inaccuracy of anticipations. In summary, the limitations of the feedforward control are:

- the accurate control is difficult due to inaccurate anticipation
- the performance of feedforward control is sensitive to the system changes
- it reacts only on the known disturbances

Therefore, feedforward control always combines with the feedback control, in order to overcome its limitations.

Feedback control

Feedback control is a closed-loop control which takes the output variables into account. It is thought to be a very robust control algorithm. In the feedback control, the controlled variables, water level and discharge, are obtained from the measurements. Those deviations from the set points are calculated and sent back into the controller which will compute the control actions to correct the deviations. After the control actions are executed, the output variables are measured again and the new control actions are generated. This creates a closed loop. In summary, advantages of feedback control are:

- the accurate control is possible due to the consecutive correctness of deviations
- the performance is not sensitive to the system changes
- the deviations caused by any factors (known or unknown) can be compensated

But the feedback control has one big limitation that it does not anticipate the disturbance, thus, it can not provide a relatively quick response. Only after the deviation occurs, the control actions are triggered. In practice, most of the feedback control is indispensable, but if the irrigation system needs the anticipation of disturbances, a combination of feedback and feedforward control is useful.

Combination of Feedback and Feedforward control

Because of both the advantages and the disadvantages of feedback and feedforward controllers, a combination is often used to control the irrigation canal system. The advantages are added together and compensate for the disadvantages each other. If the disturbances are already known, it is recommended to use the information by applying the feedforward controller in order to make a quick response. The controller does not need to be very accurate, because the feedback can compensate for the deviation. The properties of the combined feedforward and feedback control system are:

- the disturbances can be anticipated due to feedforward control
- the accurate control is possible due to the feedback control
- the performance is not sensitive to the system changes
- the deviations caused by both known and unknown disturbances are corrected

5. CANAL FLOW MEASUREMENT:

Public concepts of how to share and manage the finite supplies of water are changing. Increasing competition exists between power, irrigation, municipal, industrial, recreation, aesthetic, and fish and wildlife uses. Best management measures and practices without exception depend upon conservation of water. The key to conservation is good water measurement practices.

Benefits of Better Water Measurement

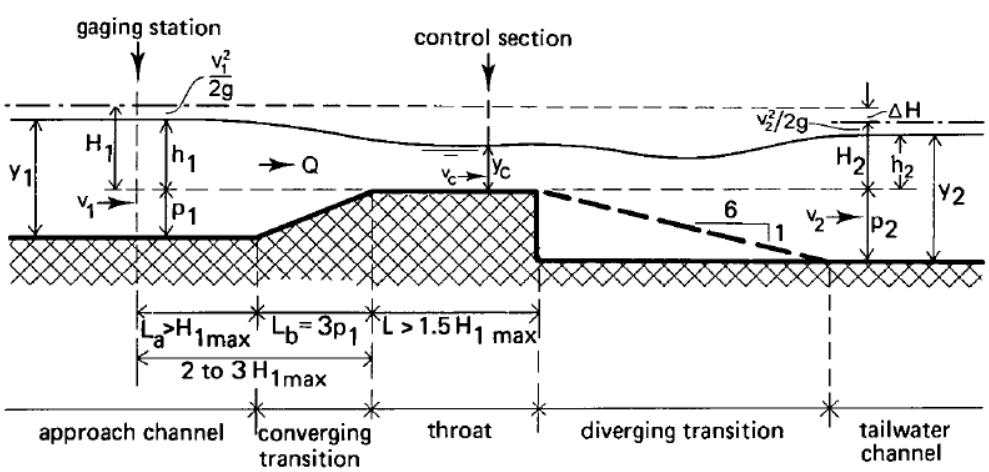
Besides proper billing for water usage, many benefits are derived by upgrading water measurement programs and systems. Although some of the benefits are intangible, they should be considered during system design or when planning a water measurement upgrade. Good water management requires accurate water measurement. Some benefits of water measurement are:

- Accurate accounting and good records help allocate equitable shares of water between competitive uses both on and off the farm.
- Good water measurement practices facilitate accurate and equitable distribution of water within district or farm, resulting in fewer problems and easier operation.
- Accurate water measurement provides the on-farm irrigation decision-maker with the information needed to achieve the best use of the irrigation water applied while typically minimizing negative environmental impacts.

- Installing canal flow measuring structures reduces the need for time-consuming current metering. Without these structures, current metering is frequently needed after making changes of delivery and to make seasonal corrections for changes of boundary resistance caused by weed growths or changes of sectional shape by bank slumping and sediment deposits.
- Instituting accurate and convenient water measurement methods improves the evaluation of seepage losses in unlined channels. Thus, better determinations of the cost benefits of proposed canal and ditch improvements are possible.
- Permanent water measurement devices can also form the basis for future improvements, such as remote flow monitoring and canal operation automation.
- Good water measurement and management practice prevents excess runoff and deep percolation, which can damage crops, pollute ground water with chemicals and pesticides, and result in project farm drainage flows containing contaminants.
- Accounting for individual water use combined with pricing policies that penalize excessive use.

Long Throated Flume:

Long-throated flumes and broad-crested weirs provide a practical, low-cost, flexible means of measuring open-channel flows in new and existing irrigation systems, with distinct advantages over other flume and weir devices. A primary advantage is the fact that these structures can be custom-designed and calibrated with a computer program based on well-established hydraulic theory. This allows the design of structures that meet unique operational and site requirements, and eliminates the need for laboratory calibration. To facilitate future use of these devices, the Bureau of Reclamation and the Agricultural Research Service have developed the software – Winflume.



Definition sketch of Long Throated Flume

Significant advantages of long-throated flumes include:

- Rating table uncertainty of ±2% or better in the computed discharge.
- Choice of throat shapes allows a wide range of discharges to be measured with good precision.
- Minimal head loss needed to maintain critical flow conditions in the throat of the flume.
- Ability to make field modifications and perform computer calibrations using as-built dimensions.
- Economical construction and adaptability to varying site conditions.

6. LONG CRESTED WEIRS

As the name implies, a long crested weir has the length of its crest perpendicular to the flow direction elongated to minimize the effect of the flow change on the variation of the head on the weir and hence the variation in its upstream water level. To study the effect of increasing the length of the weir crest we follow the same procedure utilised above in making the comparison between weirs and orifices. Here we assume that the flow is constant while the length of the weir crest (b) varies. If we consider a case of making the weir crest ten times longer then:

$$Q = Cd\ b\ \sqrt{g}\ (2/3\ h)^{3/2}$$

$$\text{Const1} = \text{Const2} * 10 * h^{3/2} \text{ so, } h = 0.215$$

i.e. when the flow over the weir is constant the head on the weir of length 10*x* will be 22% of the head on the weir of length *x*. This further enhancement in the weir design promotes the idea of using long crested weirs instead of undershot gates as upstream water level control structures.

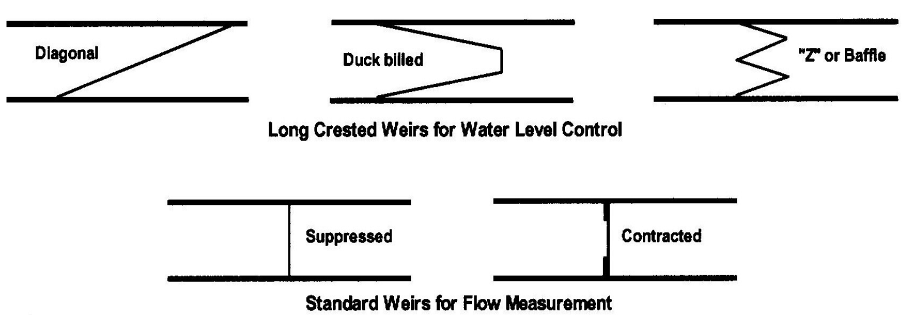


Fig.8 - Various weir configurations – plan view

The concept of a long crested weir is simple: provide more weir length than is possible with typical weirs, which are installed across the canal with the crest perpendicular to the centerline of the canal. The additional weir length makes it possible to pass the design flow rate with smaller heads. From an operations point of view this means that large changes in flow rate over the long crested weir will result in smaller changes in head and small changes in flow into the lateral or farm turnouts upstream of the weir. Long crested weirs are used to control the water surface elevation and are not intended to be used for flow measurement.

7. CONCLUSION

The technology for sensors, communication and soft computing is changing rapidly and is likely to continue to change and become more flexible, more intuitive and available at lower cost. Automation will be implemented with greater regularity as water districts transform themselves into modern service utilities. Knowledge of irrigation canal control has improved substantially over the last decade. Supervisory Control And Data Acquisition (SCADA) systems are now affordable and cost effective for irrigation distribution systems, large and small. With the advances in electronics and communications and in canal control theory and methods, significant improvements in canal control are now possible with minimal cost and infrastructure changes. It is time for irrigation to join the information age in a big way.

Chapter - 13

Maintenance of Canals and Civil Structures

After the construction of irrigation system, it becomes essential to maintain it for its proper and efficient functioning. There are various reasons due to which a canal may cease to function efficiently:

(A) Silting of canal
(B) Failure of Weaker Banks
(C) Weed growth
(D) Canal Breaches

(A) Silting of Canals

When the silt is deposited on the bed and sides, the capacity of the canal reduces. It is better to exclude silt by providing silt excluder and silt ejector. However, none of the methods can put a complete check over the entry of silt into the channel even after the best care as certain amount of silt can definitely enter into the canal. Other sources of silt deposition might be canal regulation, scouring/ erosion within the channel etc.

However, if the canal functions properly and is in regime and taking its full supply, it is not necessary to clear the silt to the theoretical cross section of the canal.

But if the canal is not functioning properly, desilting is required but only to the extent of clearing a portion of it so as to get the canal back into efficient working order.

The following methods are adopted to remove the silt:

(a) *Flushing*

Flushing of the canal with clear water will lift up deposited silt. Absolutely clear water should be used for flushing but if this is not available, then, the water which contains minimum quantity of silt should be used for flushing. Flushing should generally be over done to cause some scour in case of unlined canal. This will create room for further silting and thus reduce the frequency of flushing.

(b) *Silt scouring fleet*

The method consists of having three barges connected to the upper barge by a cable operated by winch. The lower barges have moveable shutters. The silt is kept agitated by maneuvering the barge up and down. *The method was used in Punjab but was unsuccessful.*

(c) *Bundle of thorny bushes* tied together and pressed down by weight of stones are pulled inside the channel by animals. They are quite useful in dislodging the fine muddy silt.

(d) ***Iron Rakes*** are also dragged in the channel to dislodge silt.

(e) ***Reduction of area of flow***

Loaded boats are put across the section to reduce the area of flow and increase the velocity of flow.

(f) ***Stirring of silt by water jet***

A pump fitted with a pipe and nozzle is placed on a barge . The high velocity jet is directed to the bed to stir the silt and prevent silting.

(g) ***Dredging***

A dredger is very rarely employed for removing silt from canal as it is a very costly method.

(h) ***Excavation***

The silt deposited in a channel is cleared off by manual labour. The method is quite costly as it requires recurring expenditure. This method is generally adopted for silt clearance in distributaries and minors. The silt must be deposited clear off the channel so that it does not find its way back to the channel.

(B) Failure of Weaker Banks (Strengthening of Canal Banks)

- The banks of the canals should be made in full designed width and should be maintained as such.
- Due to constant use by men and animals, the canal banks get eroded at various places. They must be repaired and all cuts and breaches filled up with suitable soil and proper tamping.
- Grass or turfing should not be scrapped while smoothening the bank surfaces, as it helps in stabilizing the soil and thus prevent the erosion. However, long grasses should be cut as far as necessary to smoothen the surface of bank and to avoid holes being hidden under long grass.
- To prevent breaching of the canal bank, it should be strengthened properly so that valuable loss of irrigation and property is prevented due to breaching of a canal section. Following are the methods of strengthening a canal bank.

  (a) External silting system
  (b) Internal silting system

. ***External silting system***

In this method subsidiary banks are constructed which run parallel to the main banks. Cross bunds are constructed between the subsidiary bank and main bank at a distance of 150m to 1500m to form compartment of silting or silting tanks.

Water is allowed to get into the compartment from upstream side and is held there for some time before discharging it back to the canal from outlet end.

When the cross bunds are separated by a distance of 150 to 300 m, the capacity of the tank is small and only a portion of full supply discharge is taken into the compartment. It is then known as into out system.

But when the length of the compartment is large say 1200 to 1500m, full discharge of the canal can be taken inside the compartment and it is then known as long reach system. This method is a costly method and may not be recommendable in actual practice.

- ***(b) Internal silting system***

In this system the canal banks are set back away from their original positions. The section of the canal provided is larger than required and therefore, its velocity is low. The section, therefore, gets silted up very quickly. To induce silting and accelerate the process, low submersible spurs may be constructed.

(C) Weed Growth

- Water weeds are unwanted plants that grow profusely in water under certain favourable conditions. They tend to reduce the discharging capacity of channel by reducing the area of the channel section and velocity of flow.
- The problem of weed growth is more marked in Deccan where the heavy weed growth may reduce the channel discharge to even less than 15%. The nuisance has, therefore, to be checked to make the channel to function efficiently.
- There are a variety of weeds growing on canal bed, water surface and water marks. They tend to thrive better in a range of 20^0 to 30^0 C.
- Weed growth is not possible in channels having high velocity of flow. But when the channel has a velocity less than 0.6 m per second, weed growth is generally possible.
- The deposition of silt has no direct effect on weed growth yet profuse weed growth is known to take place where silt is deposited.
- Light has a considerable effect on weed growth. Weed growth is accelerated in the presence of light.
- The weed growth can be checked by passing higher velocity than regime velocity in the channel. This will keep the silt in suspension and will make water turbid. Thus the light rays are cut off and silt is not deposited on channel bed.
- Yet another way of weed control is rush rotation. In the process of rush rotation, the channel is run with full supply discharge for some time and, then, it is left completely dry for some time. This helps in excluding more light when higher depth is flowing, thus reducing the weed growth. During closure, weed is unable to resist scorching rays of sun. Long duration closure has a killing effect on the weed growth.
- Weed removal may be done by plucking them by hand and burning them when channel is dry.
- In case of newly constructed canals, regular inspections should be undertaken to locate the spots where the weed growth has set in. Weeds from such spots should be removed completely so that infestation does not spread.
- In the case of old canals where aquatic weed growth is profuse, suitable mechanical or chemical methods may be employed at a stage as early as possible.

(D) <u>Canal Breaches</u>

Canal breaches are the openings or gaps created in the canal banks due to breaking up of the banks. The breaches in canal banks may be caused due to various reasons such as:

- Breach due to faulty design or construction of banks
- Breach due to overflow of the canal

- Breach due to piping
- Breach due to intentional cuts by cultivators

(a) Breach due to faulty design or construction of banks

If,

- the width of the bank is not kept sufficient;
- height is not as per correct design requirements;
- the soil material used at site is poor and is not up to the specifications; or
- The rolling, watering etc. has not been properly done at the time of construction;

the banks may not be so strong as were expected to be. Hence banks fail to hold the canal water without showing any sign of damage. This may ultimately lead to full fledged cut and breach within the length.

(b) Breach due to overflow of the canal

- Sometimes due to increased discharge in the canal, the banks may be overtopped. This overflow will definitely damage the banks.

(c) Breach due to piping

(c1) On account of exceeding FSL

Sometimes the banks may not be overtopped but the water depth may go above full supply depth due to:

- Increased discharge or
- Reduced section of the silted canal.

In such a case, the hydraulic gradient line within the filled canal bank section will also rise. If the provided cover over hydraulic gradient line is not sufficient, it may pass out of the banks.

In this way dislodgement of the soil particles will take place from the outer slopes of the banks.

This will consequently weaken and erode the banks which may ultimately lead to complete breach and water may rush out of the canal.

If the events of exceeding the full supply level are dominant and the HGL crosses the outer slopes owing to certain circumstances, the banks may be strengthened by providing counter berms.

(c2) Due to insects or burrowing animals

Water may start leaking through the holes created within the body of the bank by insects or burrowing animals.

- The leaking water will go on removing the sand particles with it.
- If this goes on unnoticed, the holes may go on increasing in size.
- Ultimately the canal bank may breach with a rush of water.
- Normally these holes are smaller in size and as such do not pose a serious problem if timely inspection and remedial plugging is done.
- As a preventive measure, a sand core is sometimes provided within the bank embankment, which will settle and fill the holes. In such cases the banks will

certainly settle down but the breach will be avoided. The settlement of the banks should be attended immediately and the damage made good.

(d) Breach due to intentional cuts by cultivators

- When the country area gets flooded due to excessive rains or poor drainage, the cultivators cut the canal banks in order to pass the drainage water to the other side.
- This sometimes rather proves detrimental to their expectations as the canal supplies may also enter the low lying area, thus it may further aggravate the water congestion in the area.
- Similarly, sometimes, the cultivators cut the banks to obtain illegal water supply from the canal. These small cuts may widen and take the shape of a big breach in a short time consequently damaging the crops.
- In order to avoid such occurrence, a strict vigil should be kept on the points of possible cuts.

(D) Canal Breaches

CLOSURE OF BREACHES

- If the breach of the bank takes place inspite of all the measures taken or otherwise, it has to be closed and the canal restored to its original shape and size.

Minor or Distributary

- As the water of the breach spreads on the adjoining land, there is usually no outside nearby place from where earth may be taken for closing the breach.
  The earth has, therefore, to be collected either
  - (i) by cutting the outer slope of the existing bank or
  - (ii) from the spoil banks (if existing) or
  - (iii) from berms of the canal
- It is essential to store huge quantities of earth on both sides of the gap before closing the breach.

- The closing process should be started from both ends.
- A breach is closed by dumping huge quantities of earth instantaneously from both sides of the gap.

- The spots from where the earth has been borrowed should be made good after the closure of the breach.

Bigger Canals, Branch Canals or Major Distributaries

The huge discharge from the canals/ branch canals/ major distributaries may completely wash away the earth dumped in the gap. Therefore simply dumping earth in the gap does not provide a solution to breach problem. The following procedure is adopted in such cases:

- Huge quantities of earth is collected on either sides of the breach from the sources as stated in case of minors and distributaries.

- Reduce the flow through the breach by
  - Driving double lines of stakes or balla piles.
  - Fill up the space between the pile lines with planks or bushes.
  - Secure the filler material (i.e. planks or bushes) by placing sand bags on the top.
- If the breach is very wide, another line of defence may be provided.

- No dumping of earth should be started before the flow through the gap has reduced considerably to the satisfaction.
- The deposited earth should now be dumped simultaneously from both the ends to form a ring bund on outer side of the breach.
- The opening is then filled with suitable earth in layers, each layer being compacted as per specifications.
- (i) All jungle from the ring bund site should be removed before earth work progresses.
  (ii) The dumped earth should be freed from grasses or bushes.

MAINTENANCE OF SERVICE ROADS

- The canal service road is unmettalled and therefore in monsoon season grass and small bushes grow on the road surface. The surface of road also wears and tears off due to some traffic over it. Some time unauthorized traffic also passes over canal roads which make the condition still worse.
- The maintenance of service roads therefore consists of:
  (i) Removal of grass and small bushes
  (ii) Levelling of road surface
  (iii) Ramming and watering of top of wearing surface.
- Maintenance is usually carried out after the monsoon season.
- To check unauthorized traffic of carts, sometimes a check barrier is constructed at every crossing. The check barrier consists of a small earth mound with a slope of 1 : 4 on upstream and downstream side with some top width. Jeeps can easily be crossed over these mounds but for trucks and bullock carts it becomes a real barrier.

Maintenance of Banks

- Regular cross-sections should be surveyed to see settlement pattern of banks. Banks shall be brought up and maintained to full section. The minimum width and free board of the bank should be in accordance original design.
- Before continuous bank repairs are started, profiles should be made 30m apart. These should be at the correct height and width of the bank repaired and should be checked before work is started.
- All the holes and rain cuts should be fully opened up to the bottom by digging steps not more than 0.5m deep in the sides and removing all the fallen or loose lumps of earth, bushes, grass roots etc. Filling and repairing should be done by placing level layers of earth (not more than 15 cm deep).
- The earth in each layer should be free from clods, roots, grass, brickbats and other debris and it should be compacted at adequate moisture content.
- Leaks should be stopped from the upstream side by cutting off the penetrating water.

If practicable, cracks should have good earth worked into them by chisel pointed poles. But if the presence of water against the bank prevents this, the leakage should be stopped by a cover of good earth thrown over it. Subsequently in dry season the defective part should be opened up and properly remade.

- Top of bank should be smooth and free from clods and silt mounds. They should be given a slight outward cross slope of about 1 in 80 in order to take the rain water away from the canal.
- Both edges of banks especially the inner ones should be neatly aligned parallel to the canal. They should be absolutely straight in straight reaches and have regular curve in curved reaches.
- Both inner and outer slopes and toes of banks should be free from irregularities. Only projections shall be cut down and earth thus obtained should be utilized in filling hollows
- The bank slopes should not be scrapped or cut as a general rule.
- Loose earth should not be left lying on top of a bank. Wherever filling is necessary, it should be well compacted.
- Grass or turfing should not be scrapped. It should only be cut as far as necessary to show the surface of the bank and to avoid the holes being hidden under long grass.
- Scrapping the top edges of banks for appearance should not be permitted.

MAINTENANCE OF CIVIL WORKS

- **Inspection and Maintenance of Civil Works**

Regular and careful inspection and maintenance of civil works must be carried out for all the components, both under water and above. Necessary repair should be carried out in time before damages are extended.

<u>Upstream and Downstream Aprons</u>

After the monsoon, sounding and probing should be done both on the upstream and downstream sides. These would be carried out by sounding rods or echo sounders over the cc blocks and loose stone protections.

- Contour maps and sections should be prepared to determine the places of scour, its extent, launching etc.
- The effectiveness of the down stream inverted filter may also be determined.
- Wherever scours have taken place extensively, the apron should be replenished to the designed values.

<u>Pucca Floor</u>

- The pucca floor, both upstream and downstream should be thoroughly inspected for cracks, wear and tear, cavitations etc.
- Energy dissipating devices like chute blocks, friction blocks, sills etc. should also be checked for damages.
- While dewatering deep downstream basins for inspection, it should be ensured that the design uplift values for the pond level condition are not exceeded. Necessary repairs should be carried out immediately.

<u>Sediment Excluding Devices</u>

Sediments excluder tunnels and the deflector should be thoroughly inspected for cracks, choking etc with the help of divers and under water lamps. Wherever necessary, desilting of chocked tunnels and channel should be carried out and other repairs attended to.

Piers

The piers should be inspected carefully for settlement,cracks, tilting etc. The noses of the piers should be checked for damages if any due to boulders, floating debris etc. Necessary repair must be attended to immediately for proper functioning of the gate, stop logs, bridges etc.

Abutments

Inspection of abutments should be carried out similar to the piers. In addition, the backfill should also be examined for settlement and if need be, it should be made up and properly compacted.

Flankwalls

Inspection and maintenance of flank walls are similar to those for the abutments. Damaged CC blocks of the flared out walls, if any , should be replaced so that hydraulic performance of the structure is not impaired.

Divide Walls

In addition to the inspection like that for the piers, the effectiveness of the divide walls for satisfactory hydraulic performance should also be observed and necessary corrective measures adopted. The protection works around the noses and shank of the divide walls should be checked up by sounding and replenished to designed values wherever necessary.

Bridges

The beams , slabs and the wearing coats of the bridges should all be checked for cracks, wear and tear, joints, etc. and repairs carried out so that smooth traffic is ensured.

Fish pass

Apart from the usual inspection necessary for the concrete structure of the fishpass, its effectiveness for the smooth movement of fish from upstream to downstream and vice versa should be studied and corrective measures adopted wherever necessary.

Navigation lock

- The concrete structure of the navigation lock should be thoroughly inspected for cracks, settlements etc. and repairs carried out.
- If the lock chamber and also the navigation channel are silted up, necessary desilting operation should be carried out. It must be ensured that the filling and empting times are maintained as per designed values. Any difficulties in the same should be removed so that free flow of navigation is maintained.

- Approaches to both upstream and downstream navigation locks should be maintained properly to avoid any congestion and damages.

<u>Head Regulators</u>

The inspection and maintenance of the head regulator are similar to those of the main barrage/ weir. The discharging capacity of the head regulator should be measured and if difficulties are experienced in passing the required discharge, necessary corrective measure should be adopted, if need be, through model studies.

<u>Instruments</u>

It is necessary that every year, performance reports are prepared,based on the different instrument observations and other observations. The observations could be broadly classified under:

- Uplift pressure
- Suspended sediment
- Settlement
- Retrogression
- Upstream aggradation
- Discharge distribution and Cross flow

Maintenance- **Instruments**

1) **<u>Uplift pressure</u>**

It may be adequate to take observations once a month during non-monsoon period and more frequently during monsoon period. However, it should be ensured that:

- The mouths of all pipes are kept closed by caps to avoid the chances of extraneous materials getting into pipes and clogging them.
- The pipes are properly and clearly numbered for identification.
- The pipes are frequently tested to make sure that the strainers are not choked.

The quantity and quality of the sediment content of the water coming out of pressure release pipes on the downstream floor should be tested during dry season. This would indicate their efficiency and also the event of undermining the foundations, so that necessary remedial measures can be adopted in time.

2) <u>Suspended Sediment</u>

The silt charge of the water upstream and downstream of the under sluices and in the canal below the head regulator should be observed frequently so that the efficiency of the silt excluding devices can be assessed and the gate regulation can be modified suitably to improve the efficiency.

3) <u>Retrogression</u>

It is necessary to measure the retrogression of the downstream river bed so that if the limit over the assumed values are exceeded, necessary remedial measures can be taken in time to ensure the safety of the structure.

4) **<u>Upstream aggradations</u>**

Upstream aggradations of the river bed increases the afflux and consequent encroachment into the free board in the components of the diversion structure including the river training works.

Hence it is necessary to know the aggradations taking place so that the top level of the components can be raised wherever necessary.

5) <u>Discharge Distribution and Cross Flow</u>

Designs are carried out assuming certain concentration of flow through the different bays of the diversion structure. Hence it is necessary to know the discharge distribution through the bays so that the designs could be checked up for the safety of the structure. Similarly cross flow should also be observed and proper remedial measures taken for ensuring the safety of the structure. This could be done by suitable modification of the gate regulation pattern.

<u>River Training Works</u>

It is necessary to have maps of the river showing the configuration of the river every year so that the effectiveness of the river training works could be studied and modified if necessary.The point of attack of the flowing water should be observed and strengthened. For this purpose, a good stock of loose stone boulders should be kept near vulnerable spots.

<u>Inspection and maintenance of mechanical and electrical works</u>

In addition to the inspection and maintenance of the civil works, the mechanical and electrical works also should be inspected and maintained regularly. If they are not kept clean, tidy and in proper working order, they will fail at the time of emergency leading to damage of the entire structure. Under the mechanical and electrical works the following could be included
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(i) Gates and falling shutter (ii) gate grooves and seals (iii) Steel wire ropes(iv) roller trains and fixed roller (v) Winches /hoists (vi) Flood Lighting (vii) Bridge bearings and super structures.

Maintenance mechanical and electrical works

<u>Gates and Falling Shutters</u>

The gates and falling shutters should be kept clear of the debris and silt accumulations and

राष्ट्रीय जल अकादमी

पुणे स्थित राष्ट्रीय जल अकादमी, केन्द्रीय जल आयोग की एक विशिष्ट संस्था है। जल संसाधन क्षेत्र से जुडे राज्य तथा केन्द्र सरकार में विविध स्तर पर कार्यरत अभियंताओं के प्रशिक्षण के क्षेत्र में राष्ट्रीय जल अकादमी एक "उत्कृष्ट केन्द्र" के रूप में कार्य कर रही है। राष्ट्रीय जल अकादमी जल संसाधन के विकास एवं प्रबन्धन के क्षेत्र में अल्प एवं मध्यम अवधि के पाठ्यक्रमों के नियमित आयोजन के साथ-साथ केन्द्रीय जल अभियंत्रण (वर्ग 'क') सेवा के अंतर्गत चयनित अधिकारियों के लिए लम्बी अवधि का प्रवेशन कार्यक्रम भी आयोजित करता है।

राष्ट्रीय जल अकादमी की वेबसाइट http://nwa.mah.nic.in से इस संबंध में अधिक जानकारी प्राप्त की जा सकती है ।

they should not be allowed to rust due to improper drainage.The upstream face of the skin plate coming into the contact with water should preferably be painted with suitable primer and subsequently with sanded aluminum paint for long life. While painting the surfaces, all necessary precautions should be observed.

Gate Grooves and Seals

Gate grooves and particularly the machined faces should be kept clean and lubricated well and their sticky deposits should be scrapped off before applying the lubricant. The efficiency of the rubber seal should be tested and also examined for wear & tear and deterioration. Their replacement whenever necessary should be done immediately.

Steel Wire Ropes

Steel wire ropes should be cleaned to remove all the dust accumulation and lubricated with suitable greases at least once a year. After lubrication, the portion of the wire ropes submerged in water should be wrapped with gunny bags which should be securely fastened to the ropes. The clamping devices also should be inspected for their efficacy.

Roller Trains and Fixed Rollers

The roller trains and fixed rollers should be clean, free movement ensured and greased for smooth operation. Worn out rollers and pins should be replaced and bolts etc. tightened.

Winches/ Hoists

Winches and lifting drums should be kept clean and greased properly for smooth and easy operation. Alignments of shafts should be checked and coupling bolts tightened.

In the case of electrically operated hoists, all the electrical wirings, switches, bearings ,reducing gears etc. should be checked for a safe and trouble free operation. The platform should also be examined and properly protected.

Flood Lighting

During the flood season, all flood lightings and site illumination should be checked daily. During non-monsoon season, it may be done once a week.

Bridge Bearings and Super Structures

The bridge bearings over the piers and abutment should be cleaned and greased once a year after the monsoon season. The painting of superstructure should be done once in two years.