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FALL STRUCTURES ON OPEN FLOW CANALS (DESIGN CONCEPT AND METHODOLOGY)

B. A. Shakir

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B.Sc. Engg. (Pb.) F.I.E. (Pak)

D.I.C. (Conc. Struct.) London

I started my engineering career in 1950 as an engineer in the Irrigation Department Punjab, where I worked on construction and operation of canals for about 8 years. During this period, I never cared to learn and understand about the theory and design of the system, as was predominantly the case with most of my colleagues. Then I shifted, to my good luck and my choice, to Central Design Office of the Irrigation Department which in fact was an Alma-mater for irrigation engineers. There my horizon developed to a wide spectrum of knowledge and I wondered as to why the career engineers usually avoid this chance of life time and grudge even a short stint of being posted on the design job.

At the time I wrote a paper on the 'Development of design of canal fall structures' over the last century. It was published in 1962 Golden Jubilee Publication of Pakistan Engineering Congress. Now after about half a century, while working as a Consultant, I realized that the younger engineers do need an introduction to design methodology of a canal drop structure, which is the basic and most important component in the system. This paper, I hope and believe, will help many a practicing engineers.

1.0 Though this paper primarily deals with the design of a fall structure, still I consider it necessary to give a peripheral introduction also to an open flow canal design.

1.1 Conveyance of water for irrigation or other utilities is done through earthen or lined channels, which flow under gravity, from main source of supply i.e the river to the large tracts of agricultural lands. The flow is maintained under gentle gradients so that a low or optimum velocity can be sustained, which does neither scour nor erode the bed or sides of the channels and neither prompts silting so that the system needs least maintenance. The slope for the specific conditions is calculated as per standard formulae which also cater for specific soil conditions. The most popular formula or the one which is well known to the engineers is Manning's as reproduced below:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \text{ (foot pound system)}$$

Where,

V = Velocity in feet per second which the designer considers, will neither scour the specific natural soil strata nor unload its burden of sediments carried from the river or picked up en-route. Explained in subsequent Para 1.2 (i).

n = Co-efficient of rugosity of the soil strata through which the channel passes. Values recommended by VEN-TI-CHOW are given in Annexure-A.

R = Hydraulic mean depth of the channel section selected.

= Area of X-Section

Wetted perimeter

S = Slope per foot length of channel

In case of metric system the formula is changed slightly as below:

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

here n remains the same, while velocity is in meters per second and other dimensions in meters.

R = Remains same but is calculated in metric system

S = Slope per meter length of channel.

1.2 For the solution of design problems following procedure is followed;

i) Selection of a velocity of flow which is non-silting and non-scouring for the particular strata and expected channel bed load. This selection could be based on studies carried out by **T.R. Camp and Kennedy**.

(a) **T.R. Camp** established a relationship between critical velocity and the particle diameter being transported in the flowing water at which the forward movement will stop and the particle will start sinking. According to his formula 0.51m/s is the critical velocity at which the load in suspension below 2mm diameter will continue to be transported and all the heavier particles will settle down, while 0.197 m/s is the critical velocity which can only transport particle sizes less than 0.2 mm while the higher grade particles will be shed off. This helps to choose a minimum velocity of 0.51 m/s (2'/sec) for canals in Punjab to avoid siltation.

(b) **Kennedy** studied data on Upper Bari Doab canal system in the Punjab and recommended a non-scouring-non silting critical velocity relationship as below:

$V_o = 0.84\, D^{.64}$

D = The depth of flow in the earthen channel.

Vo = feet/sec.

Critical velocities as per this formula are given in the Annex-B.

It indicates a velocity of 2'/sec for channel depth of 3' and 3.8'/sec for depth of 10'. **Kennedy** however, recommends a safe velocity of 3.5'/sec for channels 10' deep for Punjab soils to check against side erosion problems, which affect the stability of the channel sections.

For channels carrying suspended load of different particle sizes & grades and running through different soil terrains, as well as channels deeper than 10', one must select a maximum velocity near about 3.5'/sec and not much higher.

(c) In view of (a) and (b) above we recommend a minimum velocity of 2'/sec for earthen channels to avoid siltation problems and a velocity around 3.5'/s to guard against bank erosion.

(d) As regards the maximum velocity permissible in concrete lined channels, it could be about 2.0 to 2.5 m/s depending on quality of concrete and a little more upto about 3.0m/sec. in reinforced concrete ducts.

ii) With this velocity and the design discharge, find the area of the canal section needed for conducting the designed quantity of flow – Q.

$$\text{Area} = \frac{Q}{V}\left[\frac{\text{Discharge}}{\text{Velocity}}\right]$$

iii) Select the value of 'n' i.e. rugosity of section for the specific condition, soil strata and the transported sediments. The tables in Annex-B give values recommended by VEN-TI-CHOW in the book "OPEN CHANNEL HYDRAULICS".

iv) For the calculated area of channel section, select an appropriate section width and depth of flow. As a general rule, width is kept many times the depth, basically so that the canal supply level does not go much lower and reduce command of irrigation area.

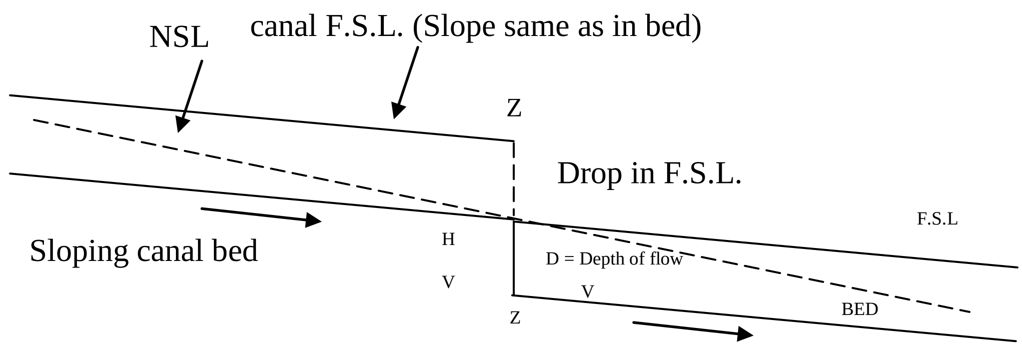
v) With the selected width and depth calculate value of "R" the hydraulic mean dept $\dfrac{\text{Area}}{\text{Wetted perimeter}}$

vi) With all other factors in the formula now known calculate the slope "S". It may however be observed that the whole exercise is tied to the controlling slope of the terrain and the exercise may have to be repeated before finalizing.

1.3 In alluvial plains of Punjab and Sindh and in similar terrain elsewhere, the natural ground slopes are very gentle and therefore, the design of irrigation channels with ruling bed gradients, which could generate a "Regime Velocity" i.e. that neither prompts silting process nor scours the bed and side embankments of earthen channels, assumes much more importance and care. In this connection, Sir **Gerald Lacey** collected a lot of data on L.B.D.C. system in Punjab, on the canals which had achieved regime flow over decades. Study of this data led him to standardize the design of channels in alluvial soils. His recommended formulae are predominantly used in the design of Irrigation Channels, with regime velocity and channel parameters for various flows.

1.4 It may also be mentioned here that selection of the most economical section of a channel requires that the section below ground level i.e. where earth has to be cut or excavated, equals the two embankments i.e. left bank and right bank which have to be built over ground level. This way the cut earth is enough to be dumped on sides and forms the compacted earth fill banks, thus no extra land for spoil dumping is needed, neither earth has to be brought from the adjoining borrow pit areas for formation of the banks.

1.5 The selection of full supply levels in the canal are controlled by the command levels of the area, where the water has to be conveyed for irrigation of the cropped fields. The sketch below gives a long-section of a canal and illustrates the need for a drop structure at a suitable location. It may be noted that the canal is generally an earthen channel while the needed structure would be a solid mass of brick or stone masonry or cement concrete mix, the later being more prevalent specifically in larger structures.



Above figure shows the naturally sloping terrain, the designed F.S.L. and bed levels with a constant slope all along the channel. At the Section "Z-Z" the canal has to negotiate a drop in water levels which would release high energy and therefore, need a solid structure with the provision of destroying the extra energy generated before the water passes down to the lower section of the earthen canal. In the absence of such an energy dissipating structure, the operation of the canal will not be possible.

1.6 The amount of energy generated at a drop depends on two factors;

(a) Quantity of flow.

(b) The drop in water level.

For the mathematical analysis of above factors following parameters are worked out:

i) Total flow being Q, the intensity of maximum flow per unit length of the weir is calculated. This is denoted as 'q', and the parameter is cubic feet per second (cusec) or cubic meter per second (cumec)

$q = \frac{Q}{B}$ Where, Q is in cusecs & B in feet (Alternately in meters in metric units)

ii) Drop or amount of fall is calculated as upstream F.S.L. minus downstream F.S.L. in the channel.

iii) The intensity of flow ‘q’ as assessed above will be correct if the flow across the total width of the canal is uniform, however in practice it is found that the channel width being large (in case of major canals/rivers or streams) it is never uniform and concentration may take place along the water way approaching the weir. This would be due to oblique approach conditions in large rivers and due to silt deposits causing formation of small islands which mask some of the waterway in front of the weir and upstream of it, reducing flow intensity in its front and concentration of flow across the remaining portion of the weir.

In case of river barrages, this aspect needs special study, however in major and medium canal waterways a concentration of flow may be to the extent of 15% to 25%, depending on the local physical conditions and the remedy would be to adopt the design intensity of flow with an increase of 15% to 25% over the average “q” value.

1.7 The destruction of exit energy generated on the downstream of a fall structure is achieved in following ways:

i) By aeration.

ii) Through formation of a hydraulic jump at the point of termination of glacis.

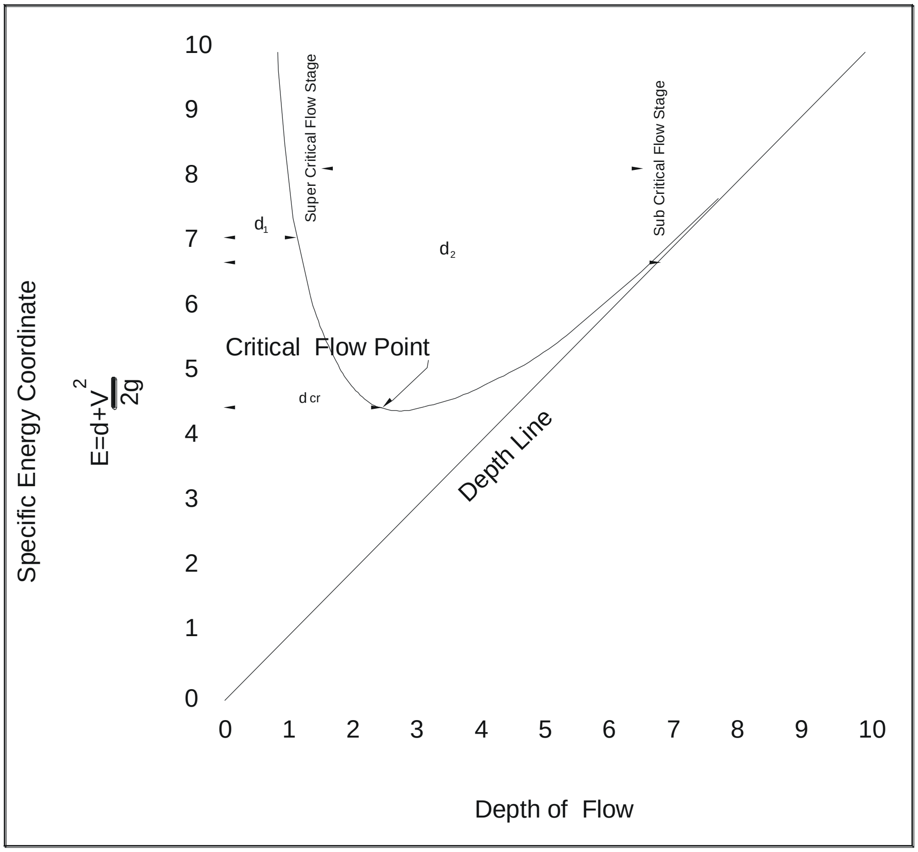
iii) By impact against designed concrete obstacles as well as depth of water.

iv) Travel of disturbed water after the jump through a calculated length within a deeper cistern where all the excessive energy is dissipated before the water enters the earthen channel.

2.0 Mechanism of formation of a hydraulic jump

In the early designs of canal falls, enough importance was not given to the fact that a properly formed hydraulic jump could dissipate most of the undesirable kinetic energy. These days, however, sufficient knowledge is available about the characteristics of the hydraulic jump to be able to predict, and therefore control its formation. This helps utilise the phenomenon to the maximum benefit and dissipate upto 90% of the released energy.

2.1 It will be worthwhile to describe here briefly the theory of formation of a hydraulic jump, with the help of specific energy diagram given below. This represents a constant discharge.



Specific energy at a point is given by the formula:

$$E = d + \frac{V^2}{2g}$$

It is evident from the above diagram that for a particular discharge "q" per foot run, streamline flow can take place at two different depths, having the same specific energy. However, between these two conjugate depths, is a point where the specific energy is the minimum. This is the point of critical flow. Below this, the flow will take place at a lesser depth and higher velocity and as such the conditions are termed as "hyper critical". On the other side the stage is sub-critical when the depth is greater than the critical flow but the velocity is less. Now, if a flow in hyper-critical stage is abruptly changed to sub-critical, the change in depth and velocity will take place at a point causing a non-uniform flow phenomenon, called "hydraulic jump" (Ref: Fig.1). The greater the difference in depth before and after the jump, more distinct will be the formation of standing wave.

It also follows from the specific energy diagram that smaller the depth before the jump, greater will be the conjugate depth required for its formation. In this transformation a large amount of energy is dissipated which may be upto ninety percent.

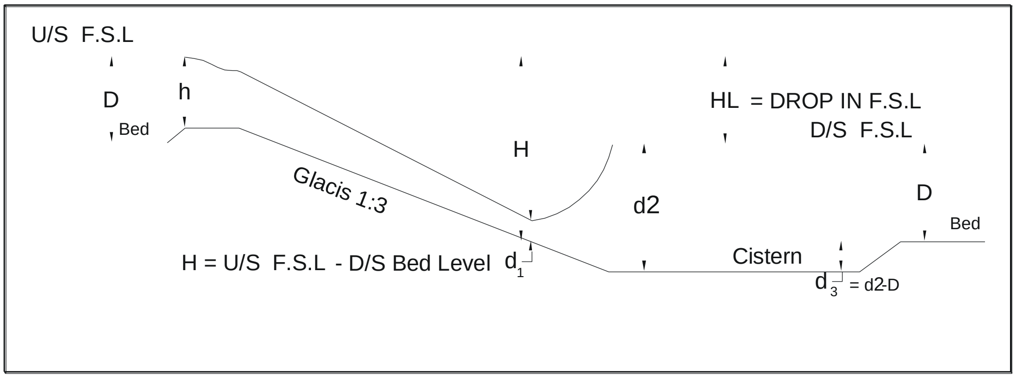


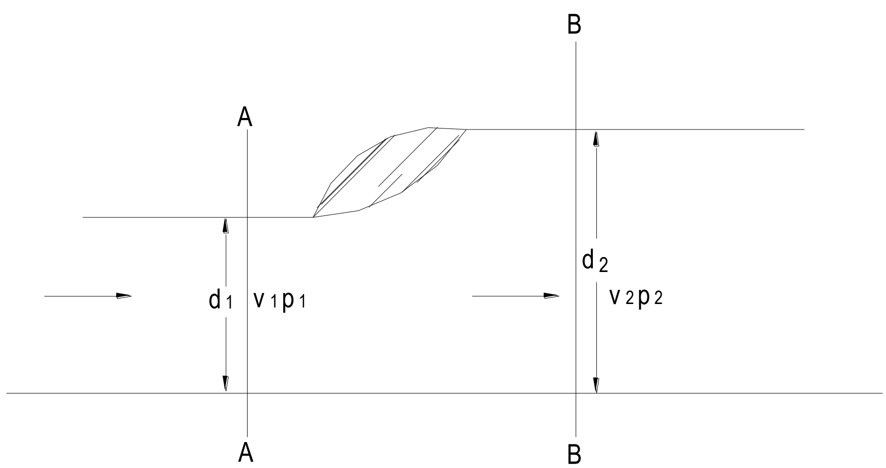
Figure-1

Hydraulic engineers utilize this phenomenon for dissipation of greater part of the energy below the fixed fall structures. The remaining energy is dissipated by aeration and impact of water on water or fixed impact blocks and as well as friction.

2.2 A simple mathematical relationship can be derived considering equilibrium of mass of water, for the formation of a hydraulic jump on a level floor, neglecting loss of energy in friction.

Referring to the sketch below

Section AA is before formation of the jump and BB after it takes place.



At section AA average pressure $p_1 = \frac{wd_1}{2}$

At section BB average pressure $p_2 = \frac{wd_2}{2}$

Horizontal force on AA $= p_1 d_1 = \frac{wd_1^{\ 2}}{2}$

Horizontal force on BB $= \frac{wd_2^{\ 2}}{2}$

Force = change of momentum/second = Mass x change of velocity per second

$$\frac{wd_2^{\,2}}{2} - \frac{wd_1^{\,2}}{2} = \frac{wq}{g}(v_1 - v_2)$$

as $\frac{q}{d_1} = v_1$ and $\frac{q}{d_2} = v_2$

substituting and simplifying we get

$$d_2^{\,2} + d_2 d_1 = \frac{dq^2}{gd_1}$$

solving the equation

$$d_2 = -\frac{d_1}{2} \pm \sqrt{\frac{2q^2}{gd_1} + \frac{d_1^{\,2}}{4}}$$

$$\text{Or} = \frac{d_1}{2} \pm \sqrt{\frac{2v_1^2 d_1}{g} + \frac{d_1^2}{4}} \quad \text{as } q = v_1 d_1$$

This also proves that smaller the value of d_1, greater will be d_2 required for the formation of a jump and vice versa. Incidentally, it is also observed that if the standing wave is formed at the toe of the glacis or a little upstream the slope, a well defined standing wave is formed and the location is almost stable. If it is formed further downstream on a level portion, the location may fluctuate and dissipation of energy may be reduced.

3.0 As regards the profile of glacis, it should normally be parabolic to be in conformity with the trajectory of the falling jet so that the under-surface of water sticks to the masonry all along its path. This will avoid the formation of vacuum. Mr. **Montague** gives the following equation for an ideal profile of glacis;

$$x = v\sqrt{\frac{4}{g}} + \sqrt{y} + y$$

Where

x = horizontal co-ordinate in the direction of flow.

y = vertical co-ordinate (below horizontal line)

v = velocity at downstream edge of crest, i.e., critical velocity. (Origin of co-ordinates will be the downstream edge of crest).

However, in practice it is not convenient to construct a parabolic profile, so a straight sloping glacis is adopted and the most suitable slope has been established through model tests to be with 1:2.5 to 1:3 (more accurately 1:3)

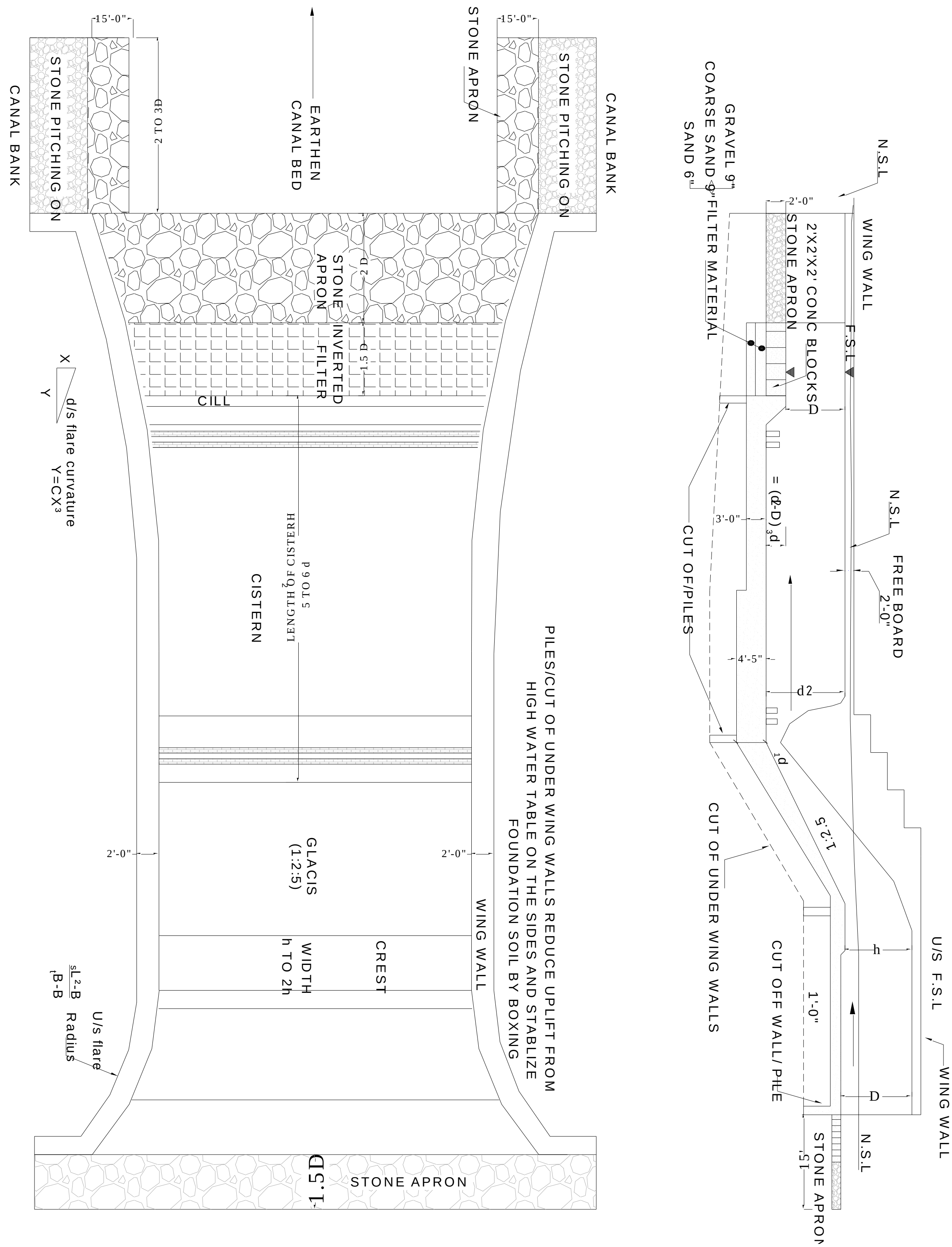
4.0 Length and Depth of Cistern required for Dissipation of Energy

The normal depth of flow available in a channel on the downstream of a fall is not sufficient for the formation of a hydraulic jump and consequently to provide the required conjugate depth (d_2), the floor is depressed to form a cistern. The depth of cistern, below the normal bed level of the canal, will therefore, be equal to the difference of d_2 and the normal depth of canal flow "D". It has been observed that the turbulence and eddies, created in the region of the standing wave, die out almost completely within a length 4 to 5 times the value of d_2. It is clarified here that the value of d_2 here includes concentration so it shall be 1.2 to 1.25 times d_2 as per concentration assumed. The cistern depth denoted as d_3 will thus be 1.25 d_2-D.

The standing wave in a properly designed fall shall be formed at the toe of the glacis or a little upstream as also mentioned earlier, at the point where the required depth d_2 is available. Consequently, the minimum length of the depressed floor (Cistern) should be about 5 times the d2. Some designers prefer greater length of the cistern to make it more effective and extend the length from 5.5 to 6.0 times d2 (including concentration). This will be preferable in case of large structures with higher flows and very wide weirs.

5.0 PRACTICAL DESIGN APPROACH

Figure – 2 gives general outline features of a fall Structure, showing various components which would help to understand the concept more clearly.



CANAL BANK
STONE PITCHING ON
15'-0"
2 TO 3D
EARTHEN CANAL BED
STONE APRON
STONE PITCHING ON
CANAL BANK
STONE APRON
INVERTED FILTER
2 D
1.5 D
CILL
d/s flare curvature
Y=CX³
CISTERN
LENGTH OF CISTERN
5 TO 6 d2
PILES/CUT OF UNDER WING WALLS REDUCE UPLIFT FROM HIGH WATER TABLE ON THE SIDES AND STABLIZE FOUNDATION SOIL BY BOXING
2'-0"
GLACIS
(1:2:5)
WING WALL
CREST
WIDTH
h TO 2h
U/s flare
Radius
1.5D
STONE APRON
GRAVEL 9"
COARSE SAND 9" FILTER MATERIAL
SAND 6"
N.S.L
2'-0"
WING WALL
2'X2'X2' CONC BLOCKS
STONE APRON
F.S.L
D
CUT OF/PILES
3'-0"
4'-5"
N.S.L
FREE BOARD
2'-0"
d2
d1
1:2.5
CUT OF UNDER WING WALLS
CUT OFF WALL/ PILE
1'-0"
h
U/S F.S.L
D
WING WALL
N.S.L
STONE APRON
15'

5.1 To take up detailed design following data may be collected;

i) Full supply discharge of the canal.

ii) An L-Section of the canal showing following parameters;

a) F.S.L. upstream of the fall.

b) F.S.L. downstream of the fall.

c) Full supply depth of canal u/s and d/s.

d) Bed slope of the canal u/s & d/s.

e) Natural ground levels in an adequate length of the reach from u/s to d/s of the structure. In fact an L-Section of the canal in this reach showing all the above features may be obtained, also showing bed width, depth and bank side slopes.

f) Value of **Lacey's** factor adopted in the design of the channel or **Manning** co-efficient 'n' in either of the design approaches.

5.2 Analytical Approach

The calculation of the location of hydraulic jump has been dealt with in detail in Para 2.0. The basic issue is to calculate the value of d_1, the lower conjugate depth and its location at the toe of the glacis. Figure-1 given in Para 2.1 may be referred. Value of "H" can be easily found by subtracting the d/s bed level from u/s F.S.L, presuming the situation when the falling water has still not arrived up to the end of Cistern, but the cistern is full because of early seepage. Consequently depth "d_1" is likely to occur where the cistern full level touches the glacis just upstream of its toe. At this point V_1 can be calculated by using the formulae,

$$V = \sqrt{2gH}$$

This would be neglecting upstream approach velocity and glacis friction. With value of 'q' known & V_1, thus calculated, we can find d_1 as under:

$$\frac{q}{V_1} = d_1$$

The value of calculated d_1 is of course little less than the actual, but the approximation is conservative as lower d_1 gives higher d_2, and hence more depth of cistern

Substitute values of v_1 and d_1 in the formula in Para 2.2, to calculate the upper conjugate depth, required for formation of the hydraulic jump.

To find the cistern depth and length it has already been explained that an amount of 25% concentration of q is adopted. Now this can be achieved by multiplying the calculated d_2 by a factor of 1.25. It may be observed that the calculated value of d_2 is invariably more than the water supply depth in the canal on the downstream therefore a depressed cistern, below the canal bed, has of necessity to be built. Thus the additional created depth helps in two ways, one to provide the conjugate depth and secondly provide additional cushion of water for absorbing the impact of falling water.

It may be observed that we have already adopted a little higher value of drop at the glacis (which would actually be $H\text{-}d_1$) and also neglected the glacis friction. The specific design is in fact empirical, based on model studies and the objective is to enhance the safety of the structure, Concentration of "q" value to 125% is also for ensuring the safety as uniformity of flow all along the weir length is not practically possible.

However, if one decides to be more immaculate depending on the choice of the Designer and his previous experiences, the following iterative procedure can be adopted;

i) Assume a value of d_1 as per previous experience, or the one calculated above.

ii) Calculate $V_1 = \dfrac{q}{d_1}$

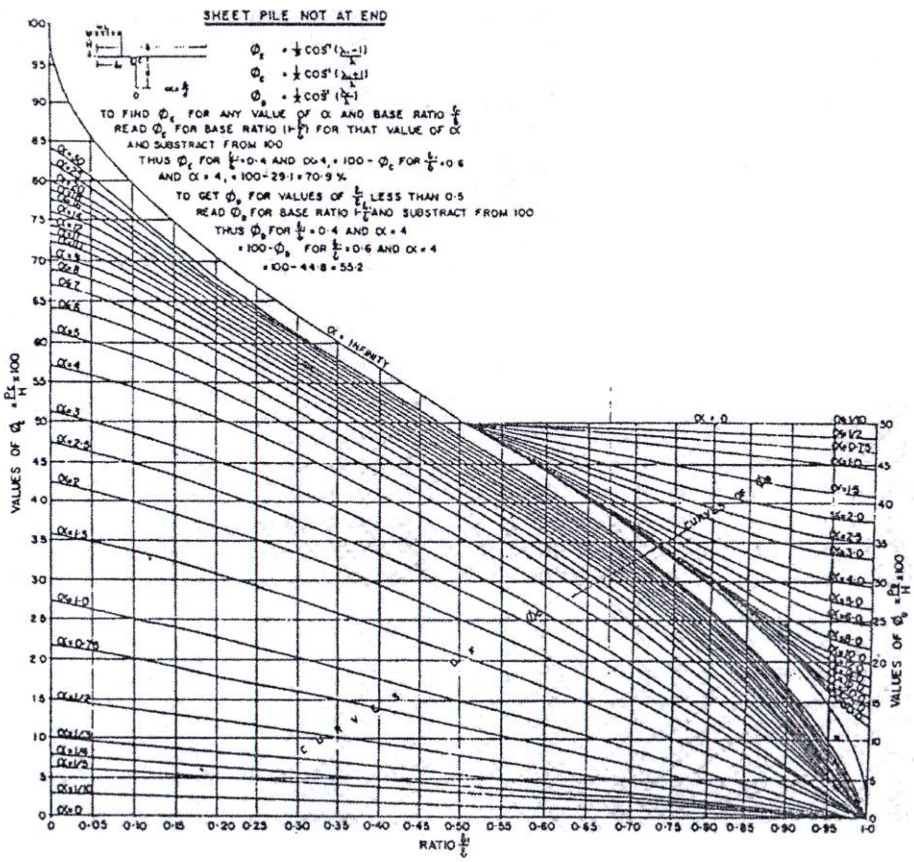
iii) Calculate v_1 from a drop equal to $H - d_1$ i.e. $V_1 = \sqrt{2g\,(H\text{-}d_1)}$

iv) Both the values of V_1 should be as close as possible. In case of difference another iterative calculation may be carried out to reduce the difference.

5.3 By Using Monograms

For practical design work, graphs have been developed by various authors which can directly provide Ef_2 (energy of flow on the downstream) for different intensities of discharge and total drop (W.L upstream – W.L d/s). The attached figure-3, provides the graphs prepared by Mr. **Khosla**, which can be conveniently used.

The value of Ef_2 is used to obtain depth and length of cistern, as it is a little more than d_2. Allowance for concentration of flow is, however, added to the value of Ef_2 thus obtained. This allowance may vary from 15% to 25%, depending upon the degree of concentration in individual cases. As a general rule lesser concentration is provided for higher discharge intensities, where the length of floor may, however, be upto 6 times Ef_2 (including concentration).



(Figure –3)

5.4 Location of Hydraulic Jump

The formation of standing wave depends on the discharge per unit width, afflux & shape of glacis, etc. Unlike level and smooth floor, wherein the position of standing wave is unstable and cannot be closely predicted, its position is more definite on glacis toe or a little upstream and can be closely predicted.

5.5 Impact and Deflector Blocks within Cistern

It has been concluded through model tests and prototypes since constructed, that rows of blocks staggered with respect to each other, if provided just downstream of the glacis improve the dissipation of energy to an appreciable extent. Generally two lines of these blocks are provided. The most suitable height of the blocks is equal to D/7, where D is normal depth of flow in the canal. The optimum distance of the upstream row, from the toe of glacis, is twice the depth of cistern 'd_3'. The first row of blocks is staggered in plan with respect to downstream row as shown in figure 4.0. The distance between 1st and 2nd row is equal to d_3. The impact Blocks are built as R.C.C components and are designed to withstand full impact of high velocity water at the toe of the glacis.

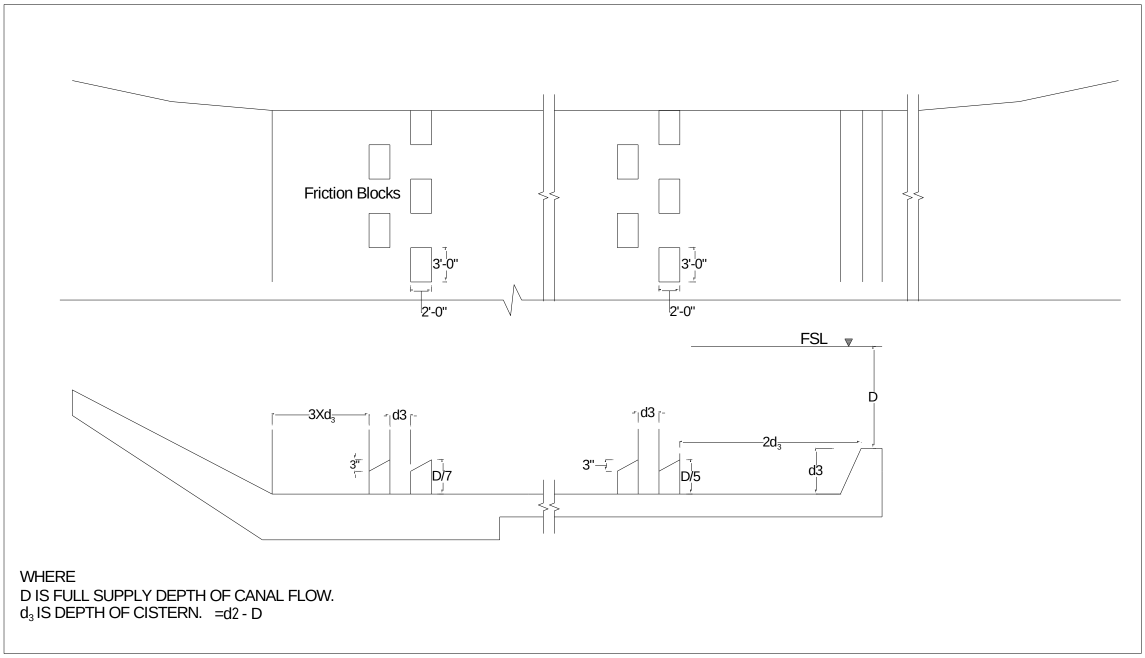


Fig - 4

Two similar rows of blocks are also erected close to the end of cistern, just short of the end cill. These serve to deflect the high velocity streams still sticking to the bed upwards to the surface to conform to normal pattern of flow in the earthen channel, where the velocity is higher in the upper layers. The optimum height of these blocks at the d/s end is equal to D/5. It is however, noted that if the height of these blocks is increased beyond the optimum, they cause harmful turbulence and eddies at the very start of earthen section. All the blocks in the four rows are 3 ft. long along the cistern width and 2 ft. wide, while there is a gap of 3 ft. between two consecutive blocks in each row. The rows are staggered in plan. The height of blocks has been given, however upstream height of each block is lower by 3 inches than its downstream height. This is for the water to be directed upwards.

5.6 Limit of optimum drop in case of design of fall structure on Alluvial Soils

In case of drop structures built on alluvial soils, there is a limit up to which a steep glacis can be considered advisable in view of the following:

i) The structural stability against horizontal thrust of surface and seepage water as well as buoyancy due to higher water table, specifically during earthquakes when slippage risk is higher.

ii) Uplift pressure on the cistern floor and glacis may be excessive in case of higher head across, more so, when the crest is gated and water may be further headed up on the upstream while the downstream water level is lower than F.SL. This situation may be arising due to operational needs.

iii) Exit gradient at the end of the structure may be high, which would undermine the structure by piping of soil material.

In view of the above factors, it is recommended that for alluvial soils a single drop structure may be restricted to drops of 20’ or near about. It would be more practical and economical to design a double drop fall within the same structure if the drop exceeds this limit by not more than about 40 to 50%. The two figures 5A & 5B explain the proposed outline. The philosophy is to extend the length of the structure to elongate the path of sub-surface seepage thus consuming the additional head and reducing the exit gradient. The two drops and cisterns also help in more efficient destruction of surface released energy.

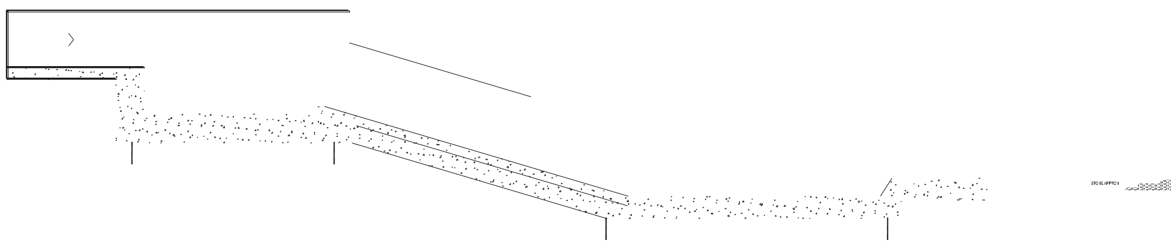


Fig: 5A

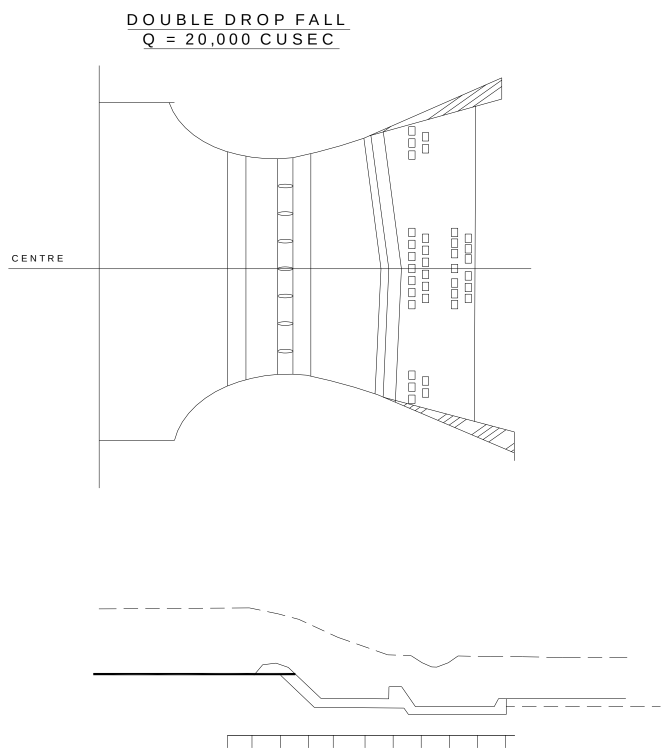


Fig: 5B

5.6.1 A Double Drop OR Split Drop Structure

The parameters of design for a double drop fall consist of the following:

(i) Decision about the extent of drop to be catered for at each stage.

(ii) Provision of a baffle at suitable distance away from the toe of glacis of the first fall, to create two distinct drops.

(iii) Provision of a suitable cistern and length of floor downstream of the baffle to dissipate the energy created at the second fall (which also includes a part of energy left over from the first fall). A row of deflector blocks is also added at the end of the floor which improves the condition of flow to a large extent.

Parameters contained in (i) and (ii) are fixed arbitrarily and depend upon designer's experience. However, as an approximate guide, the first drop should be about 60% of the total fall. As regards the distance of the baffle at its apex from the toe of the upper glacis, it should be about 1.25 Ef2 in case of a steep glacis (1 in 1.5 to 1 in 2). For glacis of a flatter slope this distance will be 2 to 3 times the Ef2.

Note: In case the drop exceeds 30' or so it will be imperative to build two separate and independent structures, the second one being some distance further downstream.

6.0 Constriction of Water way

The economy in cost of construction without a proportionate increase in the cost of maintenance is of prime concern to an engineer. When falls are coupled with bridges or gated canal regulators, an appreciable economy can be affected if the water way is laterally constricted. However, a constricted structure requires additional length and depth of cistern, greater lengths of pucca floor, flared-out walls and flexible stone apron. Therefore, to arrive at a suitable amount of constriction is a ticklish job. Sometimes different alternatives have to be worked out and costs estimated. In case of a major fall requiring heavy expenditure, model experiments have also to be conducted to evolve an efficient and an economical design.

As a general rule, fluming upto 66% (constriction by 33%) may be provided in most of the cases. The bigger the discharge or higher the drop, lesser is the constriction recommended. However, in case of major falls, where additional devices like guide vanes have been used to fan out the flow properly, it has been observed that fluming upto 50% can be carried out, provided upstream fluming approaches and downstream divergence is also properly designed.

In view of above it is obvious that the initial design will depend upon the engineers experience and then, if considered necessary, model experiments can be conducted to check the results.

6.1 Upstream Approach Curve

If the canal water way is constricted or flumed at the weir site, it naturally requires provision of flared out wing walls to effect the desired reduction of water way at the crest. Main consideration in designing the upstream approaches, and for that matter also the downstream divergence, is to see that the flow remains even across the full water way, so that the energy dissipation devices provided are most effective. This is because in case of excessive concentration of flow at some point across the waterway, the economical length and depth of cistern provided will prove inadequate for the localized area. Moreover, cross flow may take place due to differential pressure, causing turbulence and eddies, which if continued beyond pucca floor will cause scour/erosion.

In case an abrupt contraction is provided on the approach to the crest, separation takes place from the very start, the flow converges towards the middle causing concentration & boundary effects will be visible on sharp corners. The best practice, therefore, is to design the upstream approach walls as parabolic curves, giving hydraulically smooth entry conditions, where the flow sticks to the walls and does not separate out. Keeping in view the economy of cost however, the convergence can be limited to about 1 in 2. Normally, however, the parabolic approach is not provided, and instead following formula is used which gives the arc of a circle.

$$R = \frac{L^2 - B_s^{\,2}}{B - B_t}$$

Where

L	=	Length of splay.
B_s	=	Side contraction.
B	=	Normal channel width.
B_t	=	Constricted width at throat.

Approach conditions based on above formula also give reasonably smooth hydraulic conditions.

6.2 Divergence and Fanning out of Flow

As explained above a proper divergence and fanning out of the flow downstream is very essential for the dissipation of energy and delivery of effluent at the normal earthen section in harmony with the pattern of flow in the regime channel. This can be affected with the help of following:

a) Giving easy and smooth divergence such that the side streams fan out along the downstream flared walls.

b) Providing some aerating devices on the glacis.

c) Advancing crest in a bow.

d) Varying crest levels of side bays (slightly lowering).

e) Providing a baffle wall with crest at highest level in the middle and slightly sloping towards each side.

f) In case of a gated regulator structure the release of flow should be done uniformly by equal opening and closing of gates.

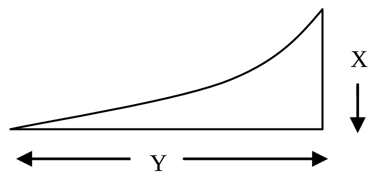
General layout of the drop structure is given in Fig. 2.

6.3 In case the side divergence is abrupt or not in line with the expansion of the fast flow streams, the flow will break away from the flank walls, causing regions of negative or zero velocities along the sides, and a parallel jet in the middle. These conditions prompt a return flow in the earthen section at the end of the fall structure, causing side erosion. The jet in the middle also scours the bed, forming dangerous pits, which if not checked can cause a total collapse of the structure.

The most appropriate divergence is about 1 in 5, with flared out walls constructed along a parabolic curve, the equation of the same being

$$Y = CX^3$$

Where $Y = 5X$



More gradual divergence than 1 in 5 will be hydraulically better but very costly. However, in case of straight divergence even with 1 in 5 opening out, the silent zones do appear on the sides.

Another point worth noting is that the divergence in a normal design i.e., where extra devices like baffle wall and vanes are not provided, should start from the toe of the glacis. This is so, as a long glacis with uniform section, creates parallel stream lines, evenly distributed across the whole width. If the divergence is started at the beginning or the middle of the glacis, the high velocity jet in the middle shoots out straight, leaving the flared out walls at the very start. This causes concentration towards the centre.

6.4 R.C.C. aero-baffles or hooks may be provided on the glacis to cause extensive aeration. These also help to even out the flow. However in such cases a model test is recommended.

6.5 As explained previously, the concentration of flow in the middle bays starts from the very upstream due to the constriction of water way. This can be offset to a great extent by constructing the crest in the plan in form of a bow, its apex being on the centre line. The actual radius for the bowing crest or the amount to which the side bays should be set-back, will depend on the designer's judgment. However, as a general rule, the bowing has to be a little more than 'h' i.e., the head of water over the crest. The alternative would be to depress crest level in the side bays which should normally be of the order of 0.5' to 0.75'. It may also be considered necessary in case of a large structure to carry out models tests once a tentative design is prepared.

6.6 In case of canals with high discharge, where the waterway is very wide, and the fluming has to be done, the problem of maintaining a uniform intensity at all points on the glacis and the cistern, is very acute. Here, if no special arrangements are made, the back flow along the flares is very pronounced. To obviate this hazard, thin curving diaphragms (vanes/divide walls) are provided longitudinally, starting from downstream edge of the crest and ending somewhere in the middle of the cistern. These vanes/diaphragms are spaced closer on the sides, to force the discharge towards the dead pockets of the channel, and allow the discharge of the intermediate bays to expand in between. The dead pockets are thus eliminated.

The height of the vanes is a matter of judgment. These are normally submerged. If a baffle wall across the water way is also provided in the design at the toe of the glacis, the vanes will abut into the same, at the lower end. The principle for fixing the alignment of the vanes is, that these should at every point along their length, divide the area of waterway proportionately. Model tests will, however, give the optimum height and the angle of curvature.

A typical diagram showing vanes is given in Fig.6. In case properly designed vanes are constructed, the side divergence can be reduced to 1 in 3, which would reduce costs on construction of downstream structure.

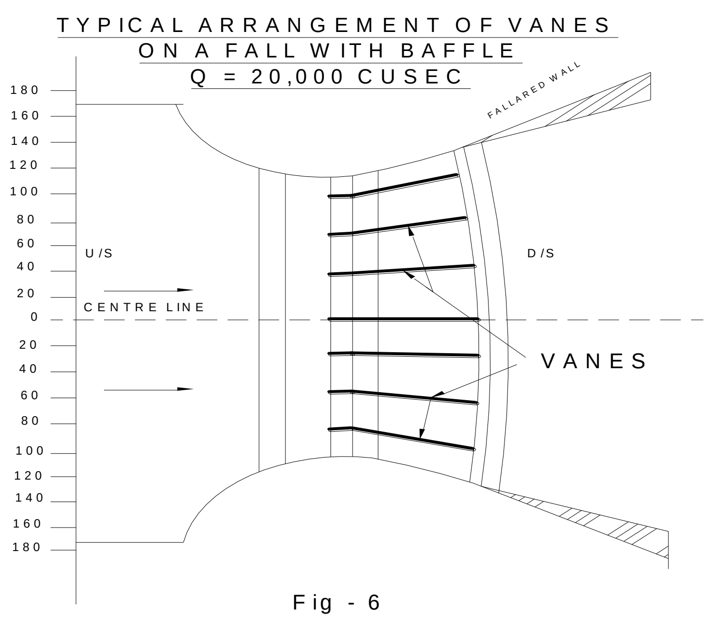


Fig - 6

The baffle at the toe is generally provided in high drop falls to create a subsidiary hydraulic jump, but it also serves to make the flow uniform. Model studies show that a baffle wall which is concave to the direction of flow spreads out the flow and vice versa. The curvature should however, be very easy. The distance of baffle wall from the toe of glacis in the middle is normally not more than $1.25d_2$ /1.25 Ef_2. At the end of it, it is between 0.75 to 1.0 Ef_2/d_2. This gives a very easy curvature.

To make the baffle wall more effective in checking the low velocity zones on the sides, it is sometime lowered a little on the sides. The lowering is gradual and slight being zero in the middle and maximum of 0.5' at the end.

The design of such baffle wall and divide vanes was model tested by the Irrigation Research Institute Lahore for the outfall structure of Balloki Sulemanki link canal. The baffle and the vanes were built by the Author in 1958. Later while working in the Design Directorate the Author designed a similar baffle wall for the Marala Ravi outfall, which was also constructed and worked successfully.

7.0 Scour at the End of Fall Structure

There is a possibility that at the downstream end of the pucca structure, some scour may develop due to residual turbulence and excess velocity of flow continuing until the start of earthen canal. The expected scour is given by "Laccy" in his formulae below for non-cohesive soils;

$$R = 0.9\left(\frac{q^2}{f}\right)^{1/3}$$

Where 'f' is Laccy's silt factor for the particular soil strata and transported suspended silt. Actually the scour depth 'R' is the depth of the channel which will develop in the strata for transporting a certain flow quantity 'q' with a non silting and non scouring velocity (regime velocity) and is measured below the full supply level. In case of a higher velocity developing close to a structure or obstruction, higher scour will develop, which will naturally deepen the local bed level and create a scour pit endangering the structure. It is therefore, necessary to provide a stone protection near the structures to cater against the additional depth for which the recommended dimensions are as under;

Class-A	Straight reach	1.25 R
Class-B	Moderate Bend	1.50 R
Class-C	Sever Bend	1.75 R
Class-D	Right angle bend	2.00 R

Thus to protect against scouring of the bed beyond "R", a cut off wall or sheet piling would be necessary. The bottom of cut off wall or any other protection must reach a level below the designed bed level equal to 0.5R or 1.0R as per the situation and grading given above.

The cut off depth in the canals is generally provided upto 0.5 R to 1.0 R below canal bed at the downstream end of floor and 0.3 to 0.5 R on the upstream end (below canal bed level) and may be a little deeper as per the experience of the designer.

It is also observed here that the depth of downstream cutoff and the total length are the two important features which control the exit gradient of subsurface flow generated due to heading up of water at the fall structure.

This aspect of controlling of exit gradient within safe values is dealt later on, in the paper.

7.1 Inverted filter and flexible stone apron

At the end of the cistern an inverted filter is laid with about 12” to 24” bottom layer of stone crush or gravel over which heavy concrete blocks 3’ x 3’ x 3’, are laid so that the filter material is not lifted or dislodged by subsoil water uplift or surface flow turbulence, which may not have died in the cistern area. This filter area may be 20’ or so long, or 1.5 to 2.0 times the F.S depth of canal. The longer length may be adopted, depending on the size of the structure and provided along the total canal cross-section width. The filter helps to break the seepage water residual energy head, so that the undermining of soil is not initiated, which might cause the collapse of the floor. See Fig: 7 & 8.

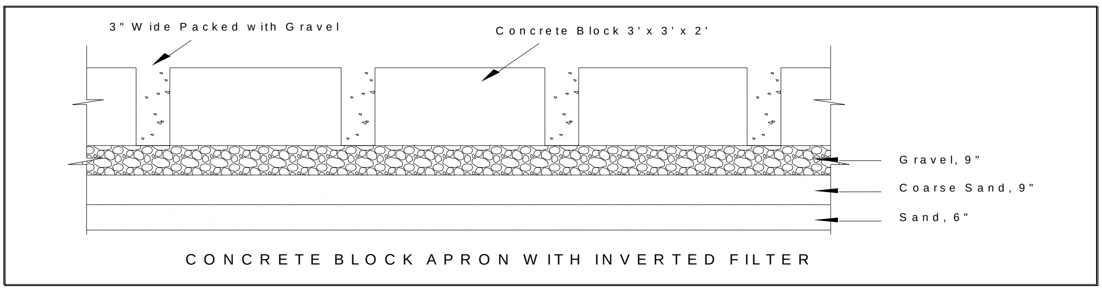


Fig: 7

After the filter area, a flexible stone apron is laid up to some length both in the bed and side slopes of the canal. This is to protect any erosive action, if higher velocity still persists. The length of d/s stone apron is equal to 1.5 to 2.0 time the F/S depth of water in the canal ie. 1.5 to 2.0 D. the thickness may be 2.0’ to 2.5’. Similarly, a stone apron is laid on the u/s of the structure in a length equal to 1.0D.

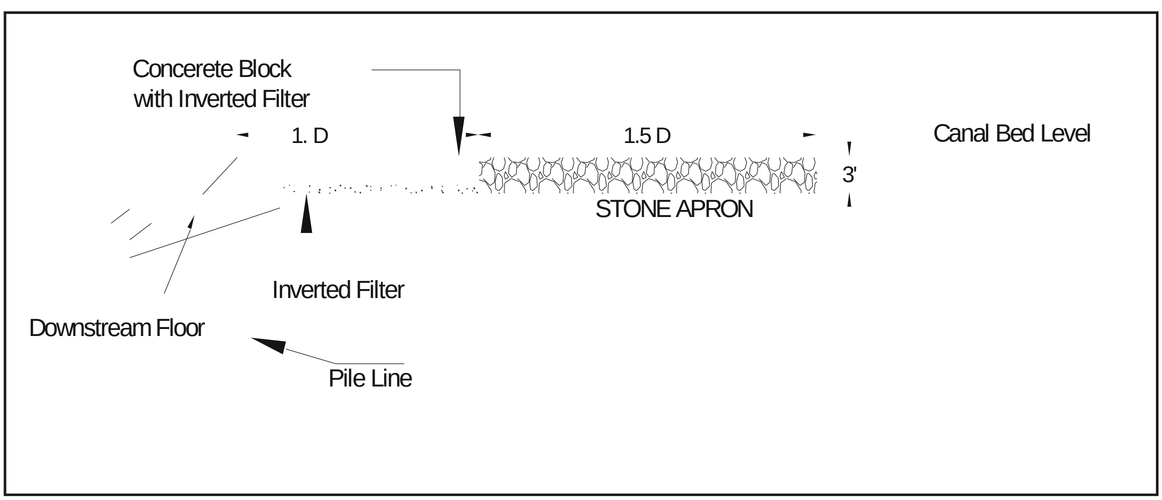


Fig-8

8.0 Sub surface flow

In the foregoing, only the surface flow conditions were discussed along with the means to control the generated energy, so as to safeguard against the possible damages to the structure and the earthen canal. Now, the sub surface flow through the foundation soil due to heading up of water at the weir has to be discussed as this can also cause damage to the structure, if not adequately protected against.

8.1 Bligh's Creep Theory

Water creeps along the base profile of the weir at its joint with the subsoil due to head of water across the structure. The percolating water loses its head en-route and finally comes out at the downstream end. **Bligh**, believed that creep is more important and destructive in undermining a weir than percolation through the soil below it. Salient features of the theory are;

(i) Hydraulic gradient line joining u/s and d/s water levels is constant through the impervious floor length of weir.

(ii) The creep starts from the upstream joint between the base profile of the floor and the subsoil and emerges out at the downstream end of the total length traversed being the creep length.

(iii) Creep length is the sum of the horizontal as well as vertical travel along both faces of a cutoff.

(iv) Percolating water emerges at the downstream end of the floor with a velocity known as exit velocity. In order that the exit velocity is safe, the creep length should be sufficient.

To ensure safety of the weir against piping and uplift pressure, **Bligh** suggested that creep length needed to dissipate a unit-head of water is different for different types of soil materials and is denoted by creep co-efficient 'c'. For some types of material it was suggested as under:

Type of soil	Gravel	Boulder or shingle & gravel & sand mixed	Coarse grained sand (central and south Indian rivers)	Fine micaceous sand (north Indian rivers)	Light sand and silt
Value of c	5	5 – 9	12	15	18

a) Against Piping:

The weir is safe against piping if exit/upward pressure at the end of the floor is negligible which would be the case if creep length is sufficient to provide a safe hydraulic.

b) Up-lift Pressure:

The up-lift pressure under the floor is the residual head at the point, according to the creep length traverse from upstream end. The floor should be designed for the balance head all along its length.

c) Total head 'H' is the higher value for the two cases;

i) Highest W.L upstream minus highest level d/s when the channel is flowing.

ii) Highest headed up water level minus, level with nil flow on the d/s.

8.2 Lane's Weighted Creep Theory

Lane modified **Bligh**'s creep theory and enunciated Lane's weighted creep theory, according to which the vertical length is three times more effective than the horizontal creep length. The floor at steeper than 45° slope are also given this weightage. Thus, Lane's weighted creep theory is same as Bligh's corrected for vertical cutoff and sloping faces.

8.3 Limitations of Creep Theory

Creep theory is effective on impervious soils like clay. Now with the introduction of flow net, analysis/exit gradient theory on pervious soils, the creep theory is no longer being used in design of structures in the Indus Valley plains, the soil where in are predominantly alluvial/pervious.

The anomalies in the creep theory are;

i) In case of loss of contact between the soil and the structure due to heaving or existence of thin layers of highly pervious soil, the results are doubtful.

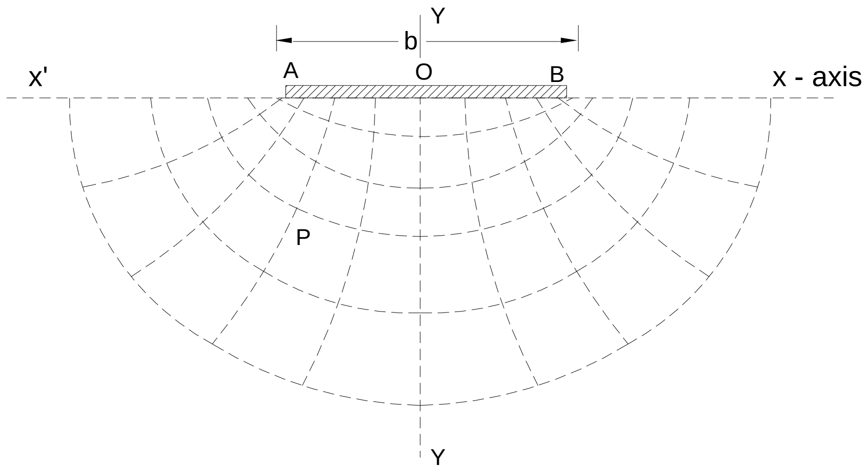
ii) Lane's weightage factor is empirical and lacks much supporting data.

iii) In case of vertical cut offs very closely placed, there may be short circuiting of creep line flow.

iv) Even in case of a structure built directly on rock, the creepage has to be blocked through the joint by proper bondage/keys.

8.4 FLOWNET ANALYSIS OF SUBSOIL SEEPAGE (Theory of Exit Gradient)

In the hydraulic design of structures involving subsoil flow and certain cases of surface flow a diagram known as **flow net** is constructed. It furnishes the pattern of stream lines and the equi-potential lines, which form two systems of lines mutually cutting at right angles and dividing the area enclosed by the boundaries of flow into curvilinear squares. The flow net is very useful in design for computing uplift pressure, exit gradients, pore pressures and seepage quantities in subsoil flow and in surface flow for predicting cavitation effects, etc.



FLOW LINE UNDER SIMPLE FLOOR

Figure-9

In constructing flow net, the hydraulic boundary conditions of the problem and their effect on the shape of the flow lines are considered. The bottom profile of the weir represents the top-most and the shortest stream line and the top surface of the foundation rock represents the lowest flow line and also the longest. The other stream lines lie between these two and their shapes must represent a gradual transition from one to the other. The upstream and downstream horizontal ground surfaces represent equi-potential lines and hence the stream lines at inlet and exit should have vertically downward and upward directions respectively.

In using flow net for the design of a structure, the equi-potential lines mean that the hydraulic potential for all points on that line is the same.

R	=	Force acting on a soil particle. This is resultant of F and W_s
F	=	Force acting in the direction of flow
W_s	=	Submerged weight of particle acting downwards

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F = Force acting in the direction of flow

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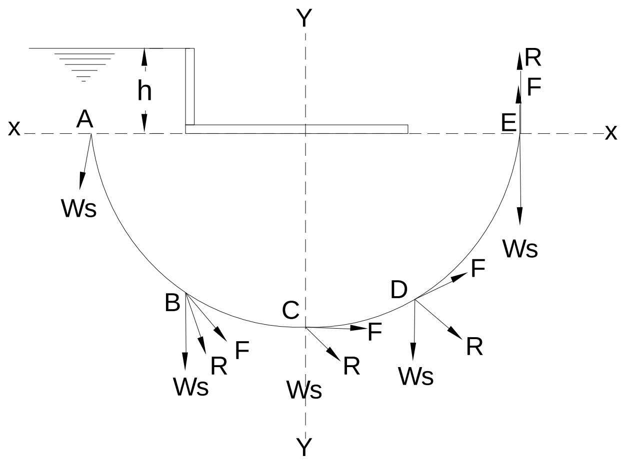


Fig: 10 Flow Line ABCDE

For stability of the soil particle under the floor, the resultant R must have no upward component. The vertical component of R from point A to point C is downward. Therefore, the floor is not subjected to any upward force. From point C to point E, force F has vertical component in the upward direction. This shows that the floor is subjected to upward force due to sub-soil water. This force goes on increasing and becomes maximum at E, the point of exit. If a particle at the exist is to be stable, the resultant force R must be zero.

	R	=	O
Or	$W_s - F$	=	O
Or	W_s	=	F

At this point the force "F" will be resisted by the submerged weight of particles acting downwards. Even a slightest increase in the value of "F" will lead to instability which will result in the lifting of the soil particles. This state is called the state of floatation. The gradient or pressure at which this occurs is called exit or floatation gradient G_E or G_F. The corresponding force is called Critical Force, F_C

Exit Gradient,

G_E or Floatation gradient		=	Critical Force, F_C
		=	Submerged Weight, W_S of sand particles
If	w	=	Weight of unit vol. of water
	ρ	=	Specific gravity of sand particles
	ε	=	Pore space in unit vol.

(i) Specific gravity of sand particles is 2.65

(ii) Then weight of soil particles per unit vol. = $2.65 \times w$

(iii) Submerged weight of above = $(2.65 - 1.0)\ w = 1.65\ w$

(iv) If the pore space of natural soil strata is 40%,

Buoyant weight of the above will be = $1.65 \times (1.0 - .04) \times w$

= $0.99\ w$

(v) As in these calculations the parameter is specific gravity, "w" will be replaced by figure 1.0, being specific gravity of water.

Thus for the particular soil and void ratio the value of G_E / Floatation gradient = 0.99

As is evident, values of G_E / G_F vary with pore space and density of soil particles. The value of G_E for various pore spaces and densities are given below:

Values of exit gradient G_E or Floatation Gradient, G_F

Pore Space	Densities					
	2.8	2.6	2.4	2.2	2.0	1.8
0.20	1.44	1.28	1.12	0.96	0.80	0.46
0.25	1.35	1.20	1.05	0.90	0.75	0.60
0.30	1.26	1.12	0.98	0.84	0.70	0.56
0.35	1.17	1.04	0.91	0.78	0.65	0.52
0.40	1.08	0.96	0.84	0.72	0.60	0.48
0.45	0.99	0.88	0.77	0.66	0.55	0.44

If the residual force of seepage water "R", at the downstream end of the floor is more than the restraining forces of the sub-soil "F", which tends to hold the latter in position, the sand particles are lifted up, and process of undermining or piping sets in and will lead to the collapse of the structure.

It may be observed that while calculating actual values of G_E for any structure being designed the formation of scour holes or retrogression of downstream levels may have to be considered which increases the head across. It is therefore, a requirement to keep adequate safety factor in the calculated exit gradient. The factors are noted in the Table below;

The values of factor of safety

Soil	Factor of Safety	Exit Gradient, GE
Shingle	4 to 5	0.25 to 0.20
Coarse Sand	5 to 6	0.2 to 0.17
Fine Sand	6 to 7	0.17 to 0.14

To calculate exit gradients for any structure following formula may be applied;

Exit Gradient, $G_E = \dfrac{H}{d} \times \dfrac{1}{\pi\sqrt{\lambda}}$

Where $\lambda = \dfrac{\sqrt{1+\alpha^2}}{2}$

And $\alpha = \dfrac{d}{b}$

Where G_E = Exit Gradient

H = Head

d = Depth of d/s sheet pile

b = Total floor length

To facilitate calculations following curves have been prepared by various authors from which G_E can be read directly.

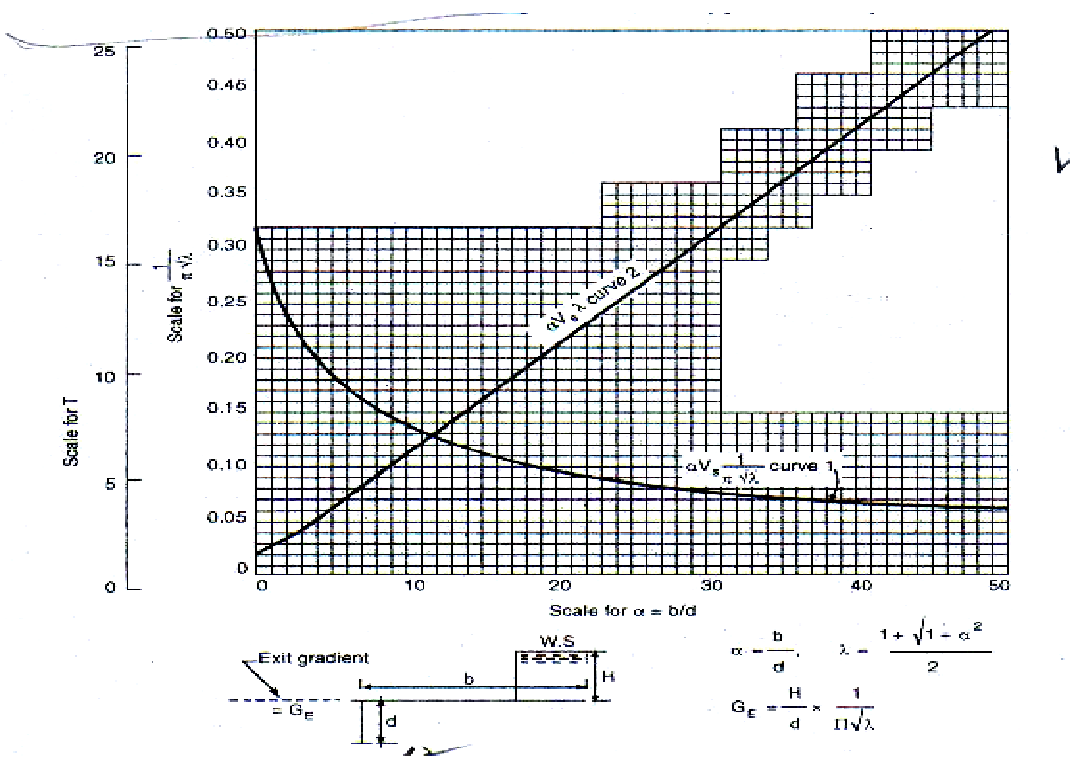


Fig-11: Exit Gradient

9.0 SUB-SURFACE FLOW BASED DESIGN PARAMETERS

9.1 By now, we have completed the 1st stage design of the drop structure covering the surface flow conditions and the appurtenant components as below;

i) u/s approach conditions and fluming arrangement including the floor, approach curvature & flexible stone protection, wing walls, cut off depths/piles etc.

ii) Weir crest above u/s floor level, calculations for head over crest and crest width.

iii) Intensity of flow and glacis design, formation of hydraulic jump and other energy dissipation devices.

iv) Cistern depth and length.

v) D/s requirement for end cut off pile to meet the minimum scouring needs.

vi) Inverted filter at end of the cistern and further flexible floor protection.

vii) We have also discussed the creep analysis, flow net solution and the calculation of Exit-gradient G_E, or Floatation gradient to check against piping/undermining of floor.

9.2 Having designed a safe structure to meet surface flow requirements, we have now to design the floor and other appurtenants to guard against up-lift pressures due to the residual force of seepage water flowing from u/s end to the d/s, because of the head potential.

Before we proceed further it must be amply clarified that control of exit-gradients and uplift pressures on the sub-structure, depends on two parameters.

a) The length of u/s floor, which could reduce pressure, because of long length of traverse. Naturally longer the seepage lines of flow through the soil more is the head loss and thus lesser the uplift pressures and value of exit-gradient.

b) The depth of end cut off/sheet pile is specifically helpful in reducing the exit-gradient and thus the risk of piping.

It is however, noted that the end pile increases the uplift on the floor upstream of it.

c) It is thus very important to first check the value of G_E with the depth of end cut off/pile designed earlier on the basis of surface scour only. If the GE is critical we will have to increase the depth of cut-off/pile or increase the length of upstream floor.

The optimization of floor length or end sheet pile depth will need one or two attempts. It must also be realized that the optimization process must also take into consideration the cost effectiveness of the alternatives, i.e. cost of lengthening the upstream floor vis-à-vis increasing the depth of pile. Additional uplift pressures caused by increasing depth of pile will need additional thickness of floor upstream of the pile.

d) It is necessary to mention here that in these structures the uplift is countered generally by gravity ie. Weight of solid mass of masonry/concrete. An RCC raft can also be built to take the uplift loading in case there are heavy piers/masonry walls erected to support heavy bridges or regulation gates. This possibility may however, exist generally on river barrages that too when the total gravity/weight as needed is provided by heavy piers placed close to each other so that an economical length of R.C.C slab can be designed.

The design against uplift is discussed in next Para 11.0.

10.0 Functions of Piles

i) Primary function of upstream end piles is to guard against the possible damage to the rigid floor because of increase in flow velocity from the wider earthen section of the canal to the reduced rigid section. At the junction scour is likely to develop as discussed earlier. A deeper cut off would check the possibility of ripping up of floor as well as enlargement of the pit.

ii) Another pile line of equal depth is provided under the weir crest as a second defense line in case the 1st line is damaged.

iii) Similarly the downstream end sheet pile line at the end of cistern, under the sill, protects against possible scouring if the high velocity flow downstream the glacis is not fully dampened within the cistern length and some balance turbulence reaches the earthen stream.

iv) A middle pile line is provided at the toe of the glacis (start of cistern) to guard against slippage of the inclined section against the horizontal thrust of surfaces and subsurface water. It also acts as a defense line in case of extensive damage.

v) Piles under the wing walls are sunk up to the same depth as cross sheet pile line. This helps to reduce uplift pressure on the floor due to high sub-soil water table in the vicinity.

vi) All this grid of pile lines under the wing walls and across the width provides a boxing effect to stabilize and strengthen the foundation soil, and also mitigates the effect of seismic thrust from any direction.

vii) End sheet piles reduce the exit gradient, as deeper the pile lower will be the exit gradient. It is however observed that the uplift pressures on the floor upstream of a sheet pile increases with the depth of pile, which effect has to be countered by increasing floor thickness (weight). This aspect is further discussed in next Para.

11.0 Uplift Pressure on the Weir Floor

In Para 8.4, we have discussed flow lines network under a simple horizontal floor. However, the weir structures are generally not straight and have rising weir crest, sloping glacis with 1:2 to 1:3 slopes, deeper cistern lengths. Cut offs/piles going deeper into soil and thick floors to counter uplift pressure (and impact). The flow network therefore, gets distorted and it is necessary to account for the distortion in calculating the correct uplift pressure so as to enable a safe design to cater for the same. The mathematical treatment of the complex structures is possible with the help of Sehwarz Christoffel transformation. As the treatment is tardish, some authors have developed graphs for direct and easy solution for facilitating the design engineers. These graphs are given in Figures 12 & 13.

The barrage floor is split in sections to help solution by consulting the graphs as per detailed procedure given on the same figures.

Exit Gradient at C is $G_E = \frac{H}{\pi d} \times \frac{1}{\sqrt{\lambda}}$

The value of GE are shown in figure and table given earlier

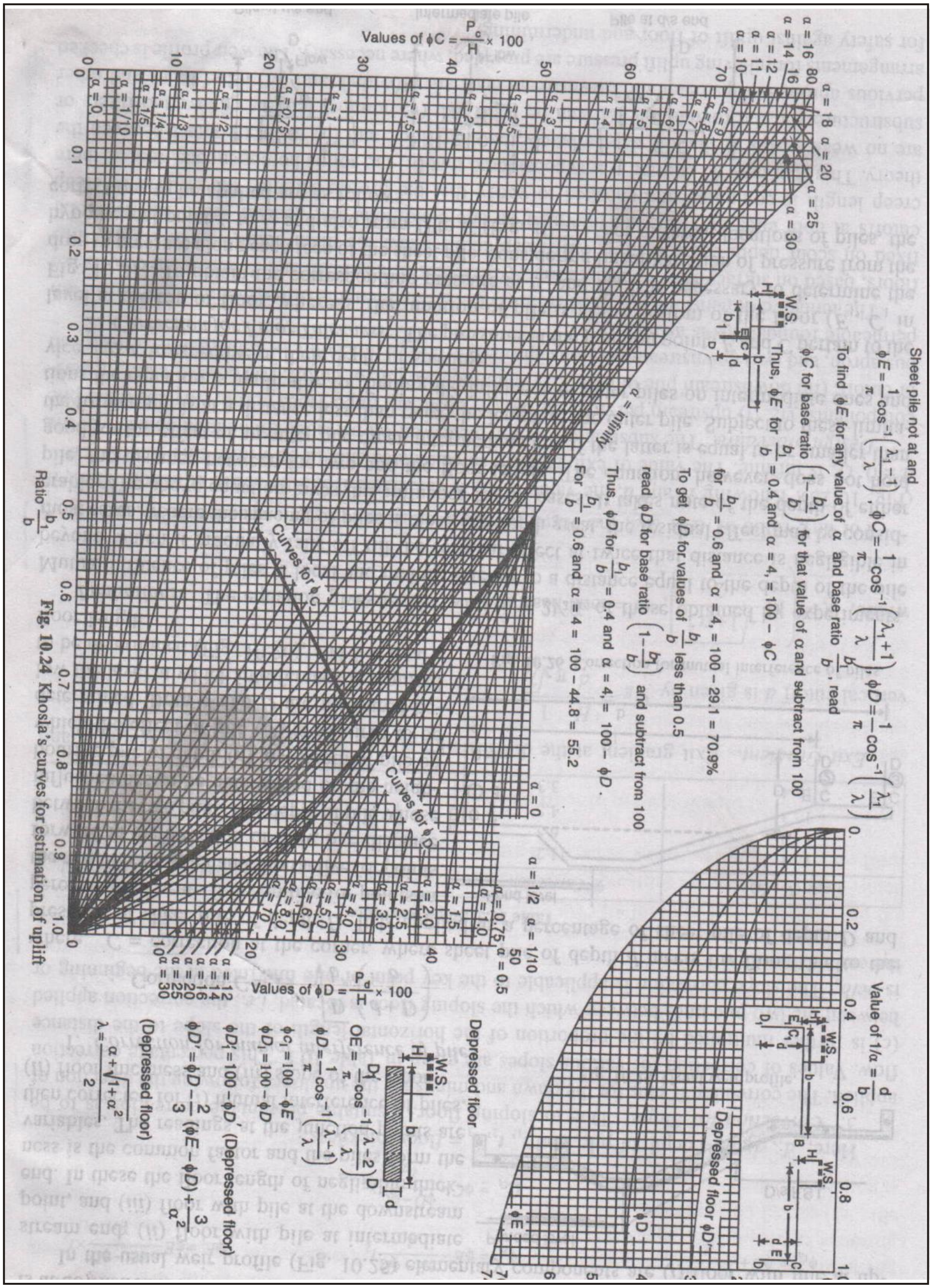


Fig-12: Pressure – Sheet Pile Not at End

10.1 Pressure Corrections

Pressure obtained above are for the elementary shapes of the floor. To obtain pressure on the actual floor (complex floor), these are to be corrected for;

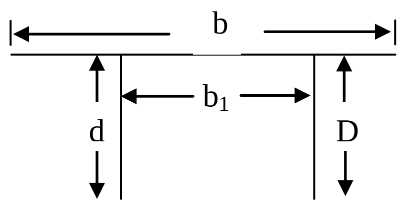
a) Mutual interference of piles;

b) Slope of the floor, and

c) Floor thickness

a. Mutual Interference of Piles

Floor with multiple lines of piles are shown below. Correction for mutual interference of piles is given by the following formula;



$$C = \pm 19 \frac{\sqrt{D}}{b_1} \quad \frac{d + D}{b}$$

Where C = the correction to be applied as percentage of head

b_1 = distance between the two piles

d = depth of pile on which the effect of pile (D) is sought to be determined

b = total floor length

D = depth of the pile, the influence of which on the neighboring pile of depth 'd' has to be determined.

The correction 'C' is added for points in the rear or back water and subtracted from points forward in the direction of flow. This equation is not applicable in the case of an outer pile or an intermediate pile if the latter is equal to or smaller than the former and is at a distance less than twice the length of the outer pile.

b. Slope Correction

The corrections, due to mutual interference of piles are applicable to horizontal floors and vertical sheet piles as these are cases of common occurrence. However, in case of sloping floors the pressure percentages under a floor sloping down or sloping up in the direction of flow of water are respectively greater or less than those under a horizontal floor for the same base ratios. To determine pressure percentage under a sloping floor a suitable correction is applied for sloping glacis. The correction being plus (+) for the down and minus (-) for the up slopes, following the direction of flow. The graph and table gives values of correction to be applied.

Correction to be applied is

$$= \pm \frac{b_3}{b_1} \times c$$

Where b_3 = is the horizontal length of the sloping floor

c = is a factor

b_1 = is the horizontal distance between two piles.

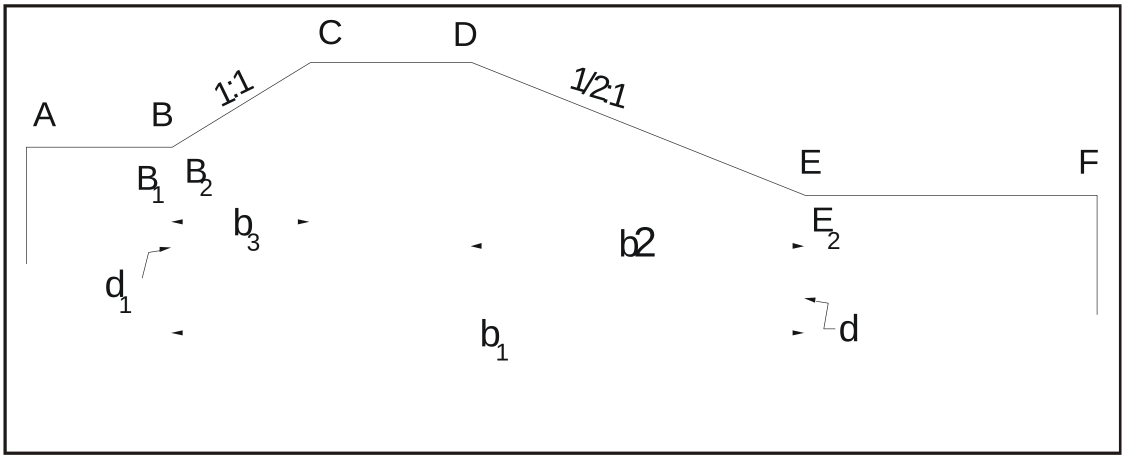


Figure-13

For up slope of floor (B-C) at B (1:1)

Slope correction	=	11.2%
Length of Slope	=	b_3 (along horizontal)
Spacing of piles	=	b_1
Correction at B & C	=	$(-)\ \frac{b_3}{b_1} \times 11.2$

For down slope of floor (D-E) at E (1/2:1)

Slope correction	=	6.5%
Length of Slope	=	b_2 (along horizontal)
Spacing of piles	=	b_1
Correction at B & C	=	$(-)\ \frac{b_2}{b_1} \times 6.5$

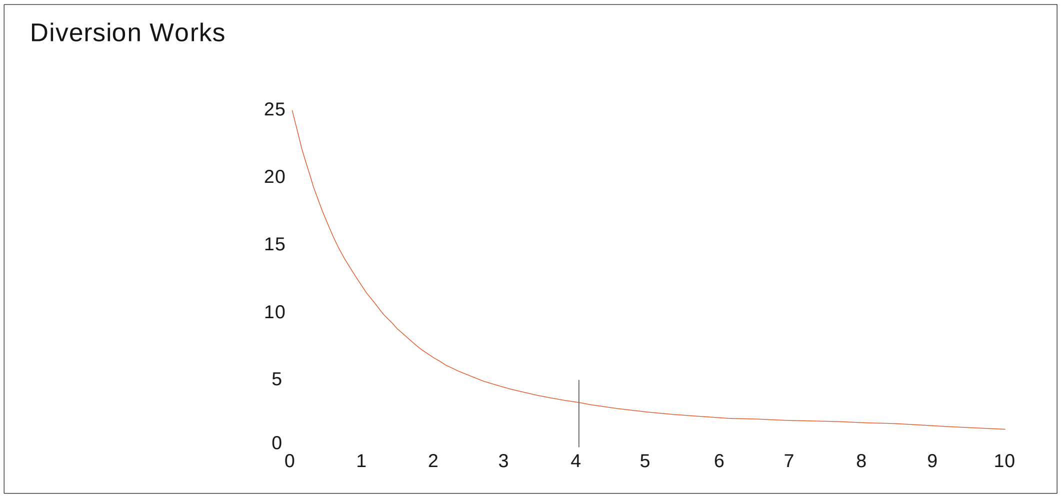


Fig: 14

The values of correction are given below:

Slope (V/H)	Correction % of pressure
1 in 1	11.2
1 in 2	6.5
1 in 3	4.5
1 in 4	3.3
1 in 5	2.8
1 in 6	2.5
1 in 7	2.3
1 in 8	2.0

c. Correction for Floor Thickness

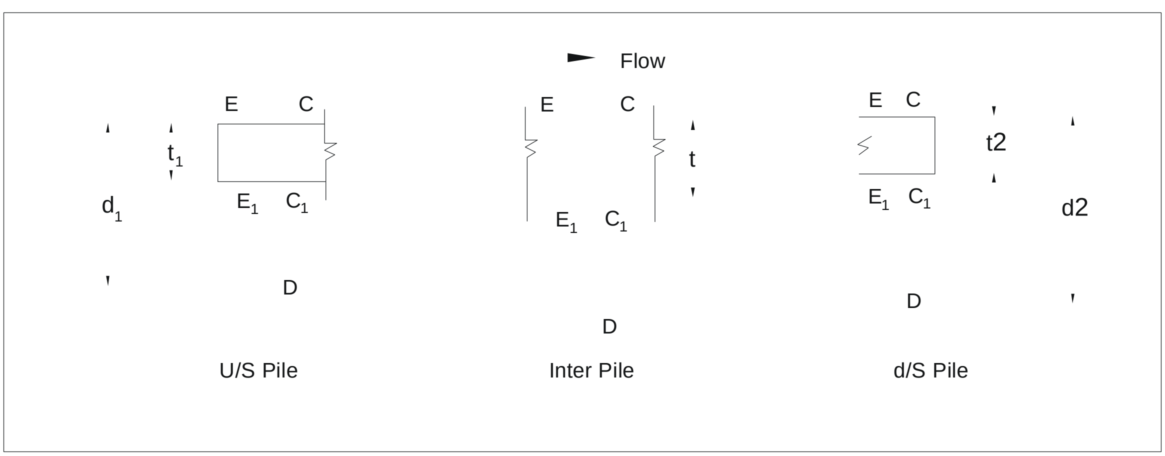


Fig: 15

The pressure at the junction points E and C pertain to the level at the top of floor, whereas, the junction of pile is at the bottom of the floor (E_1, C_1 in Fig-15). To obtain the corresponding pressure at E_1 and C_1, it is necessary to determine the drop in pressure along each face of the sheet pile, considering linear variation of pressure from the hypothetical point E to D and also from D to C (Fig-15). For different locations of piles, the corrections to be applied are as follows.

Correction for u/s pile	*Correction for intermediate pile*	*Correction for end pile*
Corrected pressure at C_1	*Corrected pressure at E_1*	*Corrected pressure at E_1*
$\o_{C1} = \o_C + \left(\frac{\o_D - \o_C}{d_1}\right) t_1$	$\o_{E1} = \o_E - \left(\frac{\O_E - \o_D}{d_2}\right) t_2$	$\o_{E1} = \o_E - \left(\frac{\O_E - \o_D}{d_3}\right) t_3$

Where,

t_1, t_2, t_3 = thickness of floors as u/s, Inter, and d/s piles,

d_1, d_2, d_3 = depth of piles on u/s, Inter and d/s piles.

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3. Irrigation Engineering by R.K. Sharma and T.K. Sharma-2007.
4. Notes of the Author from 1959 onwards, from design assignments of a score of important structures on the major canals of Pakistan and numerous other structures, he carried out. These Structure have been satisfactorily functioning for over the last half a century or so.
5. Hydraulic Model studies carried out by Irrigation Research Institute, Lahore on design of falls. Specifically are the studies carried out regarding design of baffles, divide vanes regarding even spreading out of flow on the down-stream.

Annexure – A

OPEN CHANNEL HYDRAULICS
(VEN TE CHOW)

DEVELOPMENT OF UNIFORM FLOW AND ITS FORMULAS

VALUES FOR THE COMPUTATION OF THE ROUGHNESS COEFFICIENT BY EQ. (5-12)

Channel Conditions		Values	
Material Involved	Earth	n_0	0.020
	Rock cut		0.025
	Fine gravel		0.024
	Coarse gravel		0.028
Degree of irregularity	Smooth	n_1	0.000
	Minor		0.005
	Moderate		0.010
	Severe		0.020
Variations of channel cross section	Gradual	n_2	0.000
	Alternating occasionally		0.005
	Alternating frequently		0.010-0. 015
Relative effect of obstructions	Negligible	n_3	0.000
	Minor		0.010-0. 015
	Appreciable		0.020-0. 030
	Severe		0.040-0. 060
Vegetation	Low	n_4	0.005-0. 010
	Medium		0.010-0. 025
	High		0.025-0. 050
	Very high		0.050-0. 100
Degree of meandering	Minor	n_5	1.000
	Appreciable		1.150
	Severe		1.300

Uniform Flow
Values of the Roughness Coefficient n
(Boldface figures are values generally recommended in design)

Type of channel and description	*Minimum*	*Normal*	*Maximum*
A. Closed Conduits Flowing Partly Full			
A-1. Metal			
a. Brass, smooth	0.009	0.010	0.013
b. Steel			
1. Lock-bar and welded	0.010	0.012	0.014
2. Riveted and spiral	0.013	0.016	0.017
c. Cast iron			
1. Coated	0.010	0.013	0.014
2. Un coated	0.011	0.014	0.016
d. Wrought iron			
1. Black	0.012	0.014	0.015
2. Galvanized	0.013	0.016	0.017
e. Corrugated metal			
1. Sub-drain	0.017	0.019	0.021
2. Storm drain	0.021	0.024	0.030
A-2. Nonmetal			
a. Lucite	0.008	0.009	0.010
b. Glass	0.009	0.010	0.013
c. Cement			
1. Neat, Surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
d. Concrete			
1. Culvert, straight and free of debris	0.010	0.011	0.013
2. Culvert with bends, connections and some debris	0.011	0.013	0.014
3. Finished	0.011	0.012	0.014
4. Sewer with manholes, inlet, etc., straight	0.013	0.015	0.017
5. Unfinished, steel form	0.012	0.013	0.014
6. Unfinished, smooth wood form	0.012	0.014	0.016
7. Unfinished, rough wood form	0.015	0.017	0.020
e. Wood			
1. Stave	0.010	0.012	0.014
2. Laminated, treated	0.015	0.017	0.020
f. Clay			
1. Common drainage tile	0.011	0.013	0.017
2. Vitrified sewer	0.011	0.014	0.017
3. Vitrified sewer with manholes, inlet, etc.	0.013	0.015	0.017
4. Vitrified sub-drain with open joint	0.014	0.016	0.018
g. Brick work			
1. Glazed	0.011	0.013	0.015
2. Lined with cement mortar	0.012	0.015	0.017
h. Sanitary sewers coated with sewage slimes, with bends and connections	0.012	0.013	0.016
i. Paved invert, sewer, smooth bottom	0.016	0.019	0.020
j. Rubble masonry, cemented	0.018	0.025	0.030

Development of uniform flow and its formulas
Values of the Roughness Coefficient n (continued)

Type of channel and description	*Minimum*	*Normal*	*Maximum*
B. Lined or Built-up Channels			
B-1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
B-2. Nonmetal			
a. Cement			
1. Neat, Surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Wood			
1. Planned, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplanned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	
d. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. Glazed	0.011	0.013	0.015
2. In cement mortar	0.012	0.015	0.018
g. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035
h. Dressed ashlar	0.013	0.015	0.017
i. Asphalt			
1. Smooth	0.013	0.013	
2. Rough	0.016	0.016	
j. Vegetal lining	0.030		0.500

UNIFORM FLOW
Values of the Roughness Coefficient n (continued)

Type of channel and description	*Minimum*	*Normal*	*Maximum*
C. Excavated or Dredged			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline – excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean, bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
D. Natural Streams			
D-1 Minor streams (top width at flood stage <100ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, tress and brush along banks sub-merged at high stages			
1. Bottom: gravels, cobbles and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070

Development of Uniform Flow and its Formulas
Values of the Roughness Coefficient n (continued)

	Type of channel and description	*Minimum*	*Normal*	*Maximum*
D-2.	Flood plains			
	a. Pasture, no brush			
	1. Short grass	0.025	0.030	0.035
	2. High grass	0.030	0.035	0.050
	b. Cultivated areas			
	1. No crop	0.020	0.030	0.040
	2. Mature row crops	0.025	0.035	0.045
	3. Mature field crops	0.030	0.040	0.050
	c. Brush			
	1. Scattered brush, heavy weeds	0.035	0.050	0.070
	2. Light brush and trees in winter	0.035	0.050	0.060
	3. Light brush and trees in summer	0.040	0.060	0.080
	4. Medium to dense brush in winter	0.045	0.070	0.110
	5. Medium to dense brush in summer	0.070	0.100	0.160
	d. Trees			
	1. Dense willows, summer, straight	0.110	0.150	0.200
	2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
	3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
	4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
	5. Same as above but with flood stage reaching branches	0.100	0.120	0.160
D-3	Major streams (top width at flood stage <100ft). The n value is less than that for minor streams of similar description, because banks offer less effective resistance.			
	a. Regular section with no boulders or brush	0.025		0.060
	b. Irregular and rough section	0.035		0.100

Annexure – B

Table giving values of $v_0 = 0.84\ D^{.64}$ (Kennedy's v_0) for all values of D (the depth) from 1 to 11.9

D	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9
0	0.000	0.192	0.300	0.889	0.467	0.539	0.605	0.668	0.728	0.735
1	0.840	0.693	0.954	0.994	1.042	1.088	1.132	1.179	1.223	1.266
2	1.310	1.352	1.392	1.433	1.472	1.511	1.549	1.587	1.624	1.661
3	1.696	1.732	1.767	1.803	1.837	1.873	1.907	1.941	1.975	2.007
4	2.040	2.073	2.105	2.137	2.168	2.200	2.231	2.261	2.292	2.323
5	2.352	2.382	2.412	2.441	2.470	2.500	2.529	2.558	2.586	2.615
6	2.645	2.673	2.701	2.729	2.756	2.785	2.813	2.840	2.866	2.893
7	2.920	2.947	2.973	3.000	3.026	3.050	3.076	3.101	3.127	3.153
8	3.178	3.203	3.228	3.254	3.279	3.305	3.330	3.355	3.380	3.404
9	3.428	3.453	3.478	3.502	3.526	3.550	3.574	3.598	3.622	3.645
10	3.667	3.690	3.713	3.736	3.759	3.783	3.806	3.829	3.852	3.875
11	3.897	3.920	3.943	3.966	3.988	4.010	4.033	4.055	4.077	4.099