LECTURE NOTES

Irrigation and Hydraulic Structures

(CIV- 604)

Semester:6th, *B.Tech.*

Department: *Civil Engineering*

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CHAPTER-5

Cross-Drainage Works

Cross - Drainage Works:

In an irrigation project, when the network of main canals, branch canals, distributaries, etc are provided, then these canals may have to cross the natural drainages like rivers, streams, nallahs, etc. at different points within the commanded area of the project. The crossing of canals with such obstacle cannot be avoided. Therefore, suitable structures must be constructed at the crossing point for the easy flow of water of the canal and drainage in the respective directions. These structures are called cross – drainage works. Thus, a cross – drainage work is a structure carrying the discharge from a natural stream across a canal intercepting the stream.

Figure 1. Canal crossing a natural drainage

Necessity of Cross - Drainage Works:

The cross – drainage work is required to dispose of the drainage water so that the canal supply remains uninterrupted. The canal at the cross – drainage work is generally taken either over or below the drainage. However, it can also be at the same level as the drainage. As we know that, canals are usually aligned on the watershed so that there are no drainage crossings. However, it is not possible to avoid the drainages in the initial reach of a main canal because it takes off from a diversion headworks (or storage works) located on a river which is a valley. The canal, therefore, requires a certain distance before it can mount the watershed (or ridge). In this initial reach, the canal is usually a contour canal and it intercepts a number of natural drainages flowing from the watershed to the river.

After the canal has mounted the watershed, no cross-drainage work will normally be required, because all the drainage originate from the watershed and flow away from it. However, in some cases, it may be necessary for the canal to leave the watershed and flow away from it. In that case, the canal intercepts the drainages which carry the water of the pocket between the canal and the watershed and hence the cross-drainage works are required.

A cross-drainage work is an expensive structure and should be avoided as far as possible. The number of cross-drainage works can be reduced to some extent by changing the alignment of the canal. However, it may increase the length and hence the cost of the canal. Sometimes it is possible to reduce the number of cross-drainage works by diverting the small drainages into large drainages or by constructing the cross-drainages work below the confluence of two drainages by shifting the alignment. However, the suitability of the site for the construction of the structure should also be considered while deciding the location of the cross-drainage works.

Types of Cross-Drainages Works:

Depending upon the relative positions of the canal and the drainage, the cross-drainage works may be classified into 3 categories as:

1. Canal over the drainage

(a) Aqueduct: An aqueduct is a structure in which the canal flows over the drainage and the flow of the drainage below is open channel flow. An aqueduct is similar to an ordinary road bridge (or railway bridge) across drainage, but in this case, the canal is taken over the drainage instead of a road (or a railway). A canal trough is to be constructed in which the canal water flows from upstream to downstream. This canal trough is to be rested on a number of piers. An aqueduct is provided when the canal bed level is higher than the H.F.L. of the drainage.

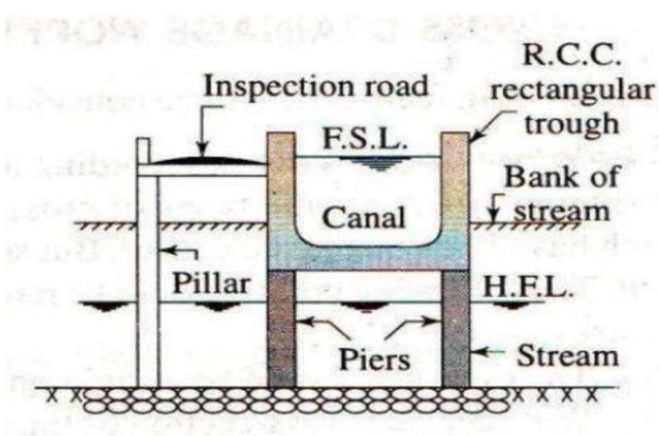


Figure 2. Aqueduct

(b) Syphon aqueduct: In a syphon aqueduct also the canal is taken over the drainage, but the flow in the drainage is pipe flow (i.e. the drainage water flows under syphonic action and there is no atmospheric pressure in the drainage). A syphon aqueduct is constructed when the H.F.L. of the drainage is higher than the canal bed level. When sufficient level difference is not available between the canal bed and the H.F.L. of the drainage to pass the drainage water, the bed of the drainage may be depressed below its normal bed level. The drainage is provided with an impervious floor at the crossing and thus a barrel is formed between the piers to pass the drainage water under pressure. Syphon aqueducts are preferred than Aqueducts, though costlier.

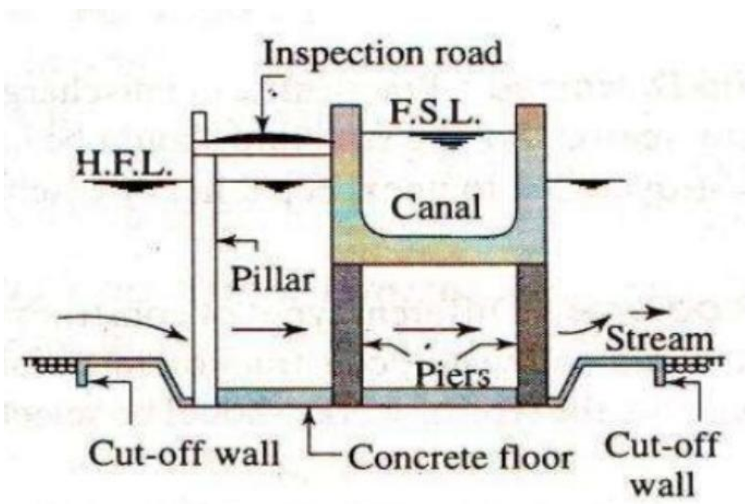


Figure 3. Syphon aqueduct

2. Canal below the drainage

(a) Superpassage: In a superpassage, the canal is taken below the drainage and the flow in the channel is open channel flow. A superpassage is thus reverse of an aqueduct. A superpassage is required when canal F.S.L is below the drainage bed level. In this case, the drainage water is taken in a trough supported over the piers constructed on the canal bed. The water in the canal flows under gravity and possess the atmospheric pressure.

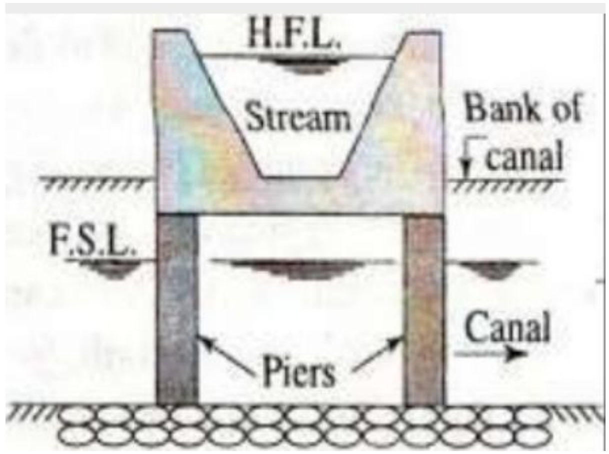


Figure 4. Superpassage

(b) Canal syphon: A canal syphon (or Simply a syphon) is a structure in which the canal is taken below the drainage and the canal water flows under symphonic action and there is no presence of atmospheric pressure in the canal. It is thus the reverse of a syphon aqueduct.

A canal syphon is constructed when the F.S.L. of the canal is above the drainage bed level. Because some loss of head invariably occurs when the canal flows through the barrel of the canal syphon, the command of the canal is reduced. Moreover, there may be silting problem in the barrel. As far as possible, a canal Syphon should be avoided.

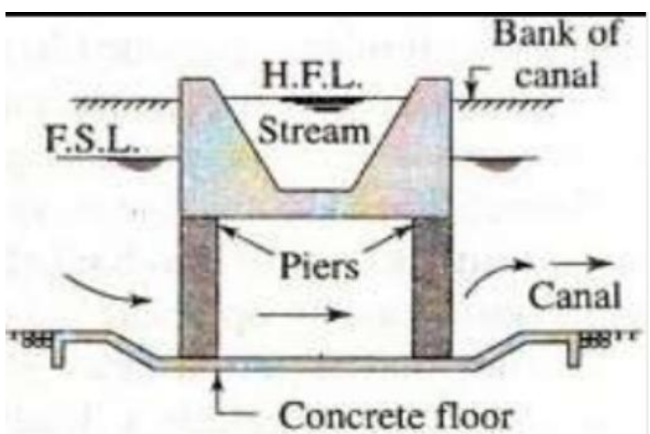


Figure 5. Canal Syphon

<u>3. Canal at the same level as drainage</u>

(a) Level crossing: A level crossing is provided when the canal and the drainage are practically at the same level. In a level crossing, the drainage water is admitted into the canal at one bank and is taken out at the opposite bank.

A level crossing usually consists of a crest wall provided across the drainage on the upstream of the junction with its crest level at the F.S.L. of the canal. The drainage water passes over the crest and enters the canal whenever the water level in the drainage rises above the F.S.L. of the canal. There is a drainage regulator on the drainage at the d/s or the junction and a cross-regulator on the canal at the d/s of the junction for regulating the outflows.

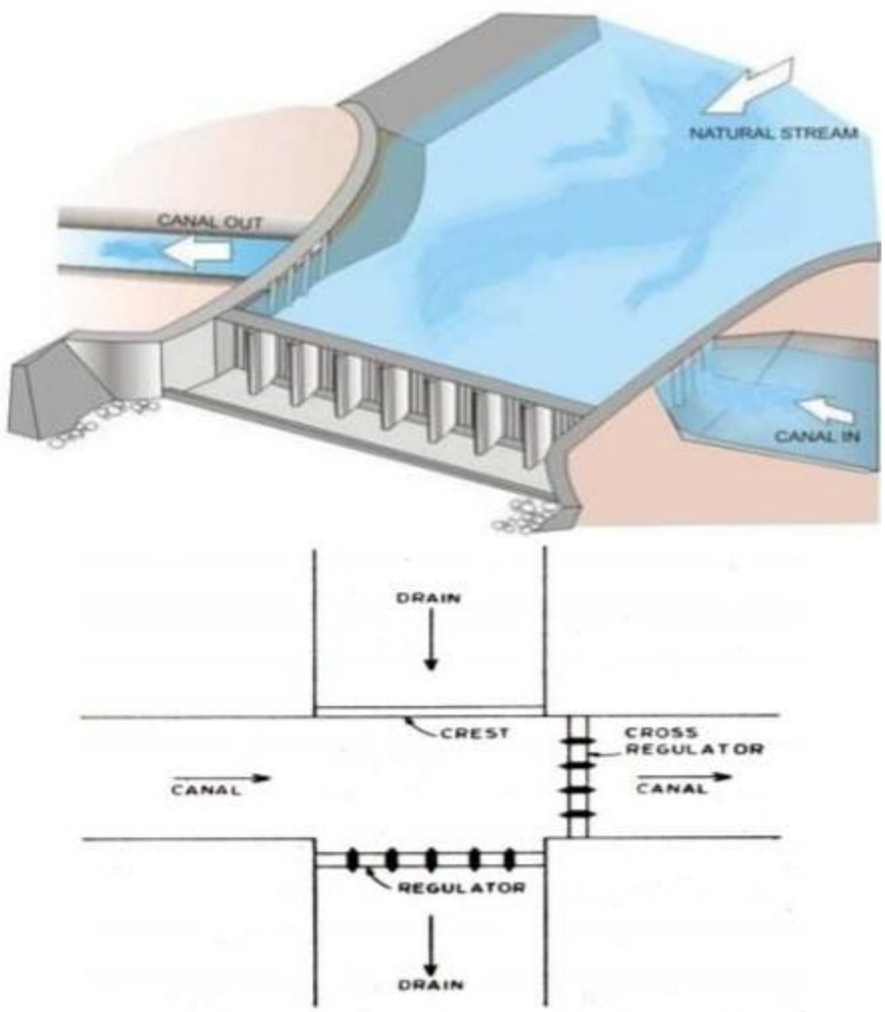


Figure 6. Level crossing

(b) Inlet and outlet: An inlet-outlet structure is provided when the drainage and the canal are almost at the same level, and the discharge in the drainage is small. The drainage water is admitted into the canal at a suitable site where the drainage bed is at the F.S.L. of the Canal. The excess water is discharged out the canal through an outlet provided on the canal at some distance downstream of junction. There are many disadvantages in use of inlet and outlet structure, because the drainage may pollute canal water and also the bank erosion may take place causing the deterioration of the canal structure so that maintenance costs are high. Hence, this type of structure is rarely constructed.

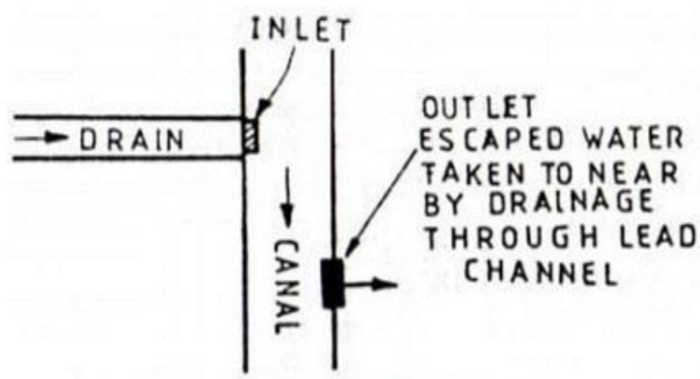


Figure 7. Inlet and Outlet

Selection of a suitable type of cross-drainage work:

The following points should be considered while selecting the most suitable type of cross – drainage work:

1. Relative levels and discharges: The relative levels and discharges of the canal and of the drainage mainly affect the type of cross – drainage work required. The following are the broad outlines:

- If the canal bed level is sufficiently above the H.F.L of the drainage, an aqueduct may be provided.
- If the F.S.L. of the canal is sufficiently below the bed level of the drainage, a super-passage is provided.
- If the canal bed level is only slightly below the H.F.L. of the drainage, and the drainage is small, a syphon aqueduct is provided.
- If the F.S.L. of the canal is slightly above the bed level of the drainage and the canal is of small size, a canal syphon is provided.
- If the canal bed and the drainage bed are almost at the same level, a level crossing is provided when the discharge in the drainage is large, and an inlet-outlet structure is provi1ded when the discharge in the drainage is small.

2. Performance: As far as possible, the structure having an open channel flow should be preferred to the structure having pipe flow. Therefore, an aqueduct should be preferred to a

syphon aqueduct. Similarly, a super-passage should be preferred to a canal syphon. The performance of inlet-outlet structures is not good and should be avoided.

3. Provision of road: A aqueduct is better than a super-passage because in the former, a road bridge can easily be provided along with the canal trough at a small extra cost, whereas in the latter, a separate bridge is required.

4. Size of drainage: When the drainage is of small size, a syphon aqueduct will be preferred to an aqueduct as the latter involves high banks and long approaches. However, if the drainage is of large size, an aqueduct is preferred.

5. Cost of earthwork: The type of cross-drainage work which does not involve a large quantity of earthwork should be preferred.

6. Foundation: The type of cross-drainage work should be selected depending upon the foundation available at the site.

7. Material of construction: Suitable types of material of construction in sufficient quantity should be available near the site for the particular type of cross – drainage work selected.

8. Cost of construction and overall cost: The cost of construction of cross-drainage work should not be

9. Subsoil water table: If the subsoil water table is high, the types of cross – drainage works which require deep excavation should be avoided.

10. Permissible loss of head: The cross-drainage works should be selected based on the permissible loss of head. Where the head loss cannot be permitted, a canal syphon should be avoided.

11. Canal alignment: The canal alignment is sometimes changed to achieve a better type of cross-drainage work. By changing the alignment, the type of cross- drainage work can be altered. The canal alignment is generally finalized after fixing the sites of the major cross – drainage works.

Selection of site of a cross-drainage work:

The following points should be considered while selecting the site of a cross-drainage work:

1. At the site, the drainage should cross the canal alignment at right angles. Such a site provides good flow conditions and also the cost of the structure is usually a minimum.

2. The stream at the site should be stable and should have stable banks.

3. For economical design and construction of foundations, a firm and strong sub-stratum should exist below the bed of the drainage at a reasonable depth.

4. The site should be such that long and high approaches of the canal are not required.

5. The length and height of the marginal banks and guide banks for the drainage should be small.

6. In the case of an aqueduct, sufficient headway should be available between the canal trough and the H.F.L of the drainage.

7. The water table at the site should not be high, because it can create dewatering problems for laying foundations.

8. As far as possible, the site should be selected downstream of the confluence of two streams, thereby avoiding the necessity of construction of two structures.

9. The possibility of diverting one stream into another stream upstream of the canal crossing should be considered, if found feasible.

10. A cross-drainage work should be combined with a bridge, if required. If necessary, the bridge site can be shifted to a cross-drainage structure or vice-versa.

Various Types of Aqueducts and Syphon-Aqueducts:

They may be classified into three types depending on the sides of the aqueduct:

<u>Type I:</u> In this type, the cross section of the canal is not changed. The original cross section of the canal with normal side slopes is retained. The length of the barrel through which the drainage passes under the canal is maximum in this type of structure because the width of the canal section is maximum. This type is suitable when the width of the drainage is small (less than 2.5 m).

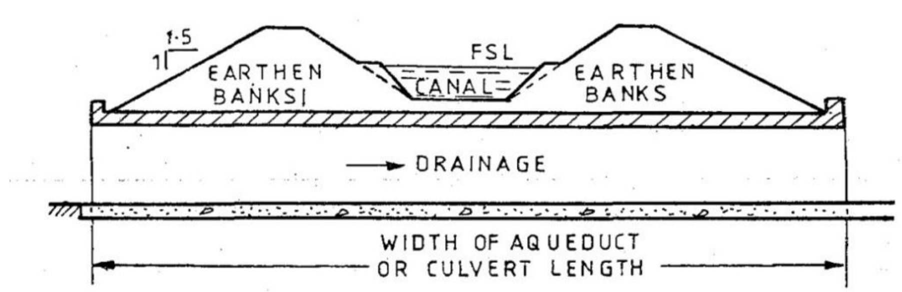


Figure 8. Type I Aqueduct

Type II. In this type, the outer slopes of the canal banks are discontinued and replaced by retaining walls. Thus the length of the barrel is reduced, but the cost of retaining wall is added to the overall cost. This type is suitable when the width of the drainage is moderate (2.5 to 15 m).

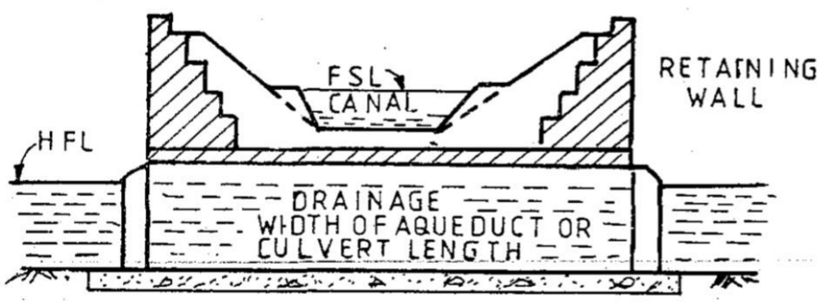


Figure 9. Type II Aqueduct

Type III. In this type, the entire earth section of the canal is discontinued and replaced by a concrete or masonry trough over the drainage. This type is suitable when the width of the drainage is very large (greater than 15 m), so that the cost of the trough and canal wing walls is less in comparison to the saving resulting from decreasing the length of barrel. In this type, the canal can be easily flumed which further reduces the length of the barrel.

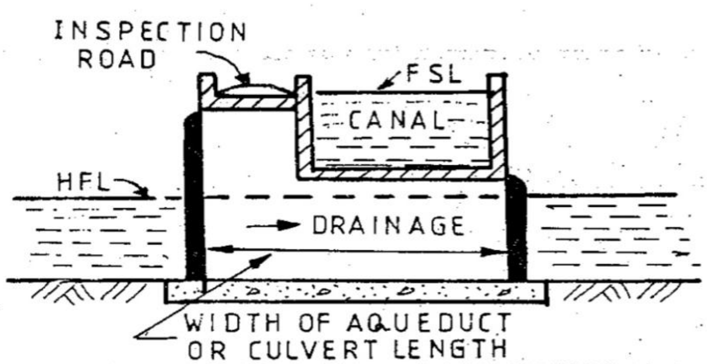


Figure 10. Type III Aqueduct

<u>Selection of the Suitable Type:</u> The selection of a particular type out of three types of aqueducts or syphon-aqueducts lies on the considerations of economy. The cheapest of the three types at a particular place shall be the obvious choice. In fact, in all cases, the cost of abutments and wing walls is independent of the length of the culvert along the canal. In type 1, no canal wings are required since the canal section is not at all changed. However, in this type, the width of the aqueduct is the largest. Type I will, therefore, prove economical only where the length of the aqueduct is small and where the cost of bank connections would be large in comparison to the savings obtained from the reduction in the width of the aqueduct. In type III, the width of the aqueduct is minimum but the cost of bank connections is maximum. This type is, therefore, suitable where the length of the aqueduct is very large and where the cost of bank connections would be small in comparison to the savings obtained from the reduction in the width of the aqueduct.

On the basis of above discussion, it can be concluded that the choice of a particular type depends mainly upon the length of the aqueduct (i.e. the width of the drainage) in relation to the size of the canal. The exact choice of a particular type in particular case can be made by working out the cost of all the types and then choosing the cheapest.

Design Considerations for Cross Drainage Works:

The following steps may be involved in the design of an aqueduct or a syphon-aqueduct. The design of a superpassage and a syphon is done on the same lines as for aqueducts and syphon aqueducts, respectively, since hydraulically there is no much difference between them, except that the canal and the drainage are interchanged by each other.

- **Determination of Maximum Flood Discharge**

The high flood discharge for smaller drains may be worked out by using empirical formulas; and for large drains, other reliable methods Such as Hydrograph analysis, Rational formula, etc. may be used.

- **Fixing the Waterway Requirements for Aqueducts and Syphon-Aqueducts**

An approximate value of required waterway for the drain may be obtained by using the Lacey's equation, given by

$$P = 4.75\sqrt{Q}$$

where P is the wetted perimeter in metres and Q is the Total discharge in cumecs.

For wide drains, the wetted perimeter may be approximately taken equal to the width of the drain and hence, equal to waterway required. For smaller drains, a smaller figure for the waterway than that given by Lacey s regime perimeter, may be chosen. The maximum permissible reduction in waterway from Lacey’s perimeter is 20%. Hence, for smaller drains, the width of the waterway provided should be so adjusted as to provide this required perimeter (minimum value =0.8 P). The decided clear water way width is provided in suitable number of bays (spans).

- **Afflux and Head Loss through Syphon Barrels**

The velocity through syphon barrels is limited to a scouring value of about 2 to 3 m/sec. A higher velocity may cause quick abrasion of the barrel surfaces by rolling grit, etc. and shall definitely result in higher amount of afflux on the upstream side of the syphon or syphon-aqueduct, and thus, requiring higher and longer marginal banks.

The head loss (h) through syphon barrels and the velocity (V) through them are generally related by Unwin's formula as

$$h = \left[1 + f_1 + f_2 \frac{L}{R}\right] \frac{V^2}{2g} - \frac{V_a^2}{2g} \qquad \text{(a)}$$

where L= Length of the barrel.

R= Hydraulic mean radius of the barrel.

V= Velocity of flow through the barrel.

$V_a =$ Velocity of approach and is often neglected.

$f_1 =$ Coefficient of head loss at entry = 0.505 for unshaped mouth

= 0.08 for bell mouth

$f_2 =$ is a coefficient such that the loss of head through the barrel due to surface friction is given by $f_2 \frac{L}{R} \frac{V^2}{2g}$.

After having fixed the velocity (V) through the barrels, the head (h) required to generate that much velocity can be found by using equation (a). The d/s HFL of the drain remains unchanged by the construction of works, and thus the u/s HFL can be obtained by adding h to the d/s HFL. The u/s HFL, therefore, gets headed up by an amount equal to h and is known as afflux. The amount of afflux is limited because the top of guide banks and marginal bunds, etc. are governed by this raised HFL. So a limit placed on afflux will limit the velocity through the barrels and vice versa. Hence, by permitting a higher afflux and, therefore, a higher velocity through the barrels, the cross-sectional area of syphon barrels can be reduced, but there is a corresponding increase in the cost of guide banks and marginal bunds and also the length of d/s protection is increased. Hence, an economic balance should be worked out and a compromise obtained between the barrel area and afflux. Moreover, in order to reduce the afflux for the same velocity, the entry is made smooth by providing bell mouthed piers and surface friction is reduced by keeping the inside surface of the barrels as smooth as possible.

- **Fluming of the Canal**

The contraction in the waterway of the canal (i.e. fluming of the canal) will reduce the length of barrels or the width of the aqueduct. This is likely to produce economy in many cases. The fluming of the canal is generally not done when the canal section is in earthen banks. Hence, the canal is generally not flumed in works of Type I and Type II. However, fluming is generally done in all the works of Type III. The maximum fluming is generally governed by the extent that the velocity in the trough should remain subcritical (of the order of 3 m/sec). Because, if supercritical velocities are generated, then the transition back to the normal section on the downstream side of the work may involve the possibility of the formation of a hydraulic Jump. This hydraulic Jump would lead to undue loss of head and large stresses on the work. The extent of fluming is further governed by the economy and permissible loss of head. The greater is the fluming, the greater is the length of transition wings upstream as well as downstream. This extra cost of transition wings is balanced by the saving obtained due to the reduction in the width of the aqueduct. Hence, an economic balance has to be worked out for any proposed design.

After deciding the normal canal section and the flumed canal section, the transition has to be designed to avoid sudden transition and the formation of eddies, etc. For this reason, the u/s or approach wings should not be steeper than 26.5^0 (i.e. 2:1 splay) and the d/s or departure wings should not be steeper than 18.5^0 (i.e. 3:1 splay). Generally, the normal earthen canal section is trapezoidal, while the flumed pucca canal section is rectangular. It is also not necessary to keep the same depth in the normal and flumed sections. Rather, it may sometimes be economical to increase the depth and still further reduce the channel width in cases where a channel encounters a reach of rocky terrain and has to be flumed to curtail rock excavation. But an increase in the water depth in the canal trough will certainly increase the uplift pressures on the roof as well as on the floor of the culvert, thus requiring larger roof and floor sections and lower foundations. Due to these reasons, no appreciable economy may be obtained by increasing the depth.

- **Design of Pucca Canal Trough:**

The canal trough is designed as follows:

For an Aqueduct: In case of an aqueduct, the bottom of the canal i.e. the roof of the culvert is subjected to the dead weight and the vertical load of water from the top, as shown in Figure 11.

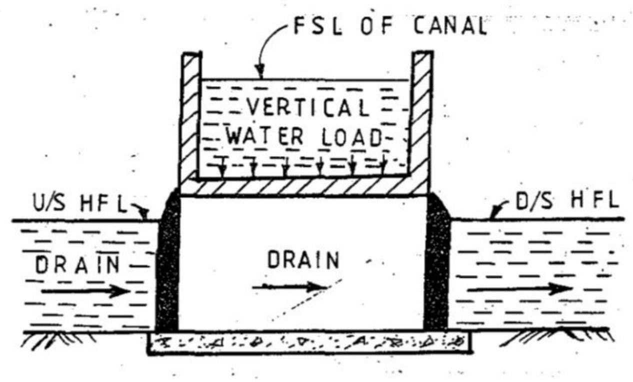


Figure 11. Loads acting in aqueduct

Since in an aqueduct, there is no uplift from the underside acting on the bottom of the canal bed, the canal bottom has to be designed for taking the dead weight and full water load by either providing a thickness sufficient to take this much of load merely by gravity or by providing reinforced concrete slab with reinforcement at its bottom.

The side walls of the canal are to be designed as retaining walls. They may be made of masonry or R.C.C. It is preferable to have an entire R.C.C. section. The retaining walls will be designed to carry the entire horizontal force exerted by the canal water and shall, therefore, carry reinforcement on the water face.

For an Aqueduct Syphon: In case of an aqueduct syphon, besides the vertical load of canal water, one more force comes into action i.e. the uplift pressure exerted by the drain water. The roof of the culvert i.e. the bottom-slab of canal should now be designed to withstand these two forces independently. Although these two forces act in a opposition to each other but still under the worst circumstances, there may be times when only one of them may be acting. For example, when drainage is flowing at its maximum high flood level, canal may be empty. Similarly, there may not be any drainage water touching the slab when the canal may be running full. Hence, the slab should be designed for

(i) full water load and dead weight, with no uplift

(ii) full-uplift with no water load.

When the slab is designed to counter-balance the maximum uplift, merely be gravity, it is sometimes found that the slab thickness required is less than what is required for the first condition i.e. when designed for full water load). But many a times, the thickness required for balancing uplift may exceed the thickness required for balancing the water load. In that case, it is generally not advisable to increase the thickness because any increase in thickness will result in lowering the levels of both the roof of the culvert as well as the bottom slab of culvert. This, in turn, will increase the uplift on the roof slab as well as on the bottom slab of culvert.

- **Design of bottom floor of Aqueduct and Syphon Aqueduct:**

The floor of the aqueduct or syphon-aqueduct is subjected to uplift due to two causes:

(a)Uplift due to water-table: This force acts where the bottom floor is depressed below the drainage bed, especially in syphon aqueducts.

The maximum uplift under the worst condition would occur when there is no water flowing in the drain and the water table has risen up to the drainage bed. The maximum net uplift in such a case would be equal to the difference in level between the drainage bed and the bottom of the floor, as shown in Figure 12.

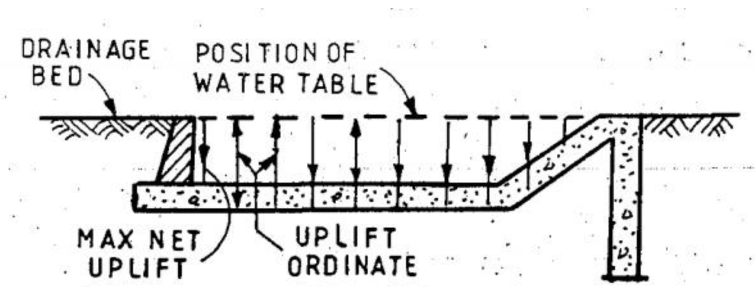


Figure 12

(b) Uplift due to seepage of water from the canal to the drainage: The maximum uplift due to this seepage occurs when the canal is running full and there is no water in the drain. The computations of this uplift, exerted by the water seeping from the canal on the bottom floor, are very complex and difficult, due to the fact that the flow takes place in three dimensional flow net. The flow cannot be approximated to a two dimensional flow, as there is no typical place across which the flow is practically two dimensional. Hence, tor the smaller works, Bligh's Creep theory may be used for assessing the seepage pressures. But, for the larger works, the uplift pressures must be checked by model studies. The seepage pressure can be evaluated by Bligh's theory as explained below:

The seepage flow occurs from the beginning of pucca canal trough (point a) and reappears in the drainage bed on either side of the impervious floor along the centre of the floor of the first culvert bay (say point c or point d) in Figure 13. Point b is the point under the centre of the floor of the first culvert bay.

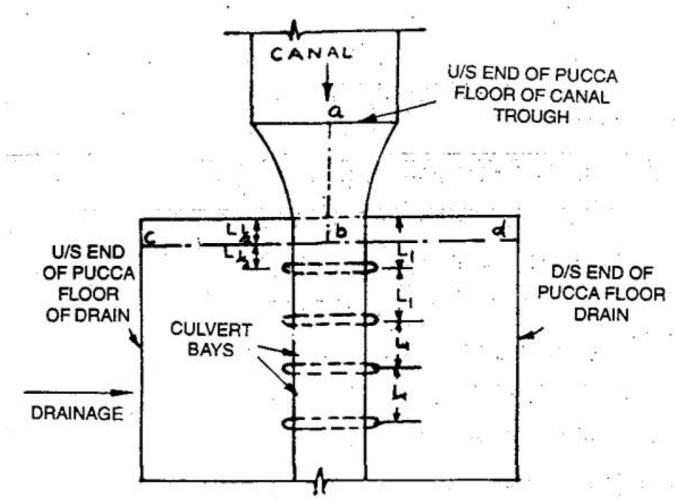


Figure 13

The seepage path from a to b and from b to c can be known. The total creep length will then be equal to a b + b c. If H is the total seepage head (i.e. H=FSL of canal - d/s bed level of drain), the residual head at the point b (i.e., H_b) is then given by Bligh's theory as equal to

$$H_b = H - \left[\frac{H}{ab+bc} \times ab\right]$$

$$H_b = \left[\frac{H}{ab+bc} \times ab\right]$$

The floor of the syphon-aqueduct must be designed for the total uplift which is equal to the sum of the uplift due to seepage plus the uplift due to static head. The total uplift may be partly resisted by the wt. of the floor and partly by bending in reinforcement.

Methods of reducing uplift on the floor: The uplift on the bottom floor may be reduced in two ways:

(i) By extending the impervious canal trough on either side of the drainage so as to increase the creep length ab. A puddle apron may be used in place of concrete floor, if clay is easily available.

(ii) By providing drainage holes in the culvert floor so as to release the uplift. If such relief holes are provided in the bottom floor; an inverted filter, should also be provided below the floor. The inverted filter would help in preventing the soil particles from getting out of the holes. The performance of such holes may not prove very successful in actual field as it appears to be on paper. Because, if these get choked or if there occurs some defect in filter system, there may be a danger of failure of work by excessive uplift or by undermining.

- **Design of Bank Connections:**

Two set of wings are required in aqueducts and syphon-aqueducts. These are

(i) Canal Wings or Land Wings.

(ii) Drainage Wings or Water Wings.

(i) Canal wings or Land wings: These wings provide a strong connection between the masonry or concrete sides of a canal trough and earthen canal banks. These wings are generally warped in plan so as to change the canal section from trapezoidal to rectangular. They should be extended up to the end of splay. These wings may be designed as retaining walls for maximum differential earth pressure likely to come on them with no water in the canal. The foundations of these wings should not be left on filled earth. They should be taken deep enough to give safe creep length.

(ii) Drainage wings or Water wings or River wings: These wing walls retain and protect the earthen slopes of the canal, guide the drainage water entering and leaving the work, and join it to guide banks and also provide a vertical cut-off for the water seeping from the canal into the drainage bed. The foundations of these wing walls should be taken below the deepest anticipated scour in the river. The sections of these wing walls should be capable of withstanding the maximum differential earth pressure likely to come on them.

The layouts of these sets of wings depend on the extent of contraction of canal and drainage waterways, and the general arrangement of the work.

Design of Canal Transitions:

The following methods may be used for designing the channel transitions:

(i) Mitra's method of design of transitions (when water depth remains constant).

(ii) Chaturvedi's method of design of transitions (when water depth remains constant).

(ii) Hind's method of design of transitions (when water depth may or may not vary).

(i) Mitra's Hyperbolic Transition when water depth remains constant:

Shri A.C. Mitra, Chief Engineer, U.P. Irrigation Deptt. (Retd.), has proposed a hyperbolic transition for the design of channel transitions. According to him, the channel width at any section X-A, at a distance x from the flumed section is given by

$$B_x = \frac{B_n . B_f . L_f}{L_f B_n - (B_n - B_f)x}$$

where B_n = Bed width of the normal channel section.

B_f = Bed width of the flumed channel section.

B_x = Bed width at any distance x from the flumed section

L_f = Length of transition.

(ii) Chaturvedi's Semi-Cubical Parabolic Transition when water depth remains constant:

Prof. R.S. Chaturvedi, Head of Civil Eagineering Deptt. in Roorkee University (Retd.), on the basis of his own experiments, had in 1963, proposed the following equation for the design of channel transitions when water depth remains constant.

$$x = \frac{L.B_n^{3/2}}{B_n^{3/2} - B_f^{3/2}} \left[1 - \left(\frac{B_f}{B_x}\right)^{3/2}\right]$$

Choosing various convenient values of B_x; the corresponding distance x can be computed easily from the above equation.

(ii) Hind's Method for the design of Transitions when water depth may also vary:

This is a general method and is applicable either when the depths in the flumed and unflumed portions are the same, or when these depths are different. In Figure 12, the contraction transition (i.e. the approach transition) starts at section1-1 and finishes at section 2-2. The flumed section continues from section 2-2 to section 3-3. The expansion transition starts at section 3-3 and finishes at section 4-4. From section 4-4 onwards, the channel flows in its normal cross-section and the conditions at this section are completely known. Let V and y with appropriate subscripts refer to velocities and depths at different sections.

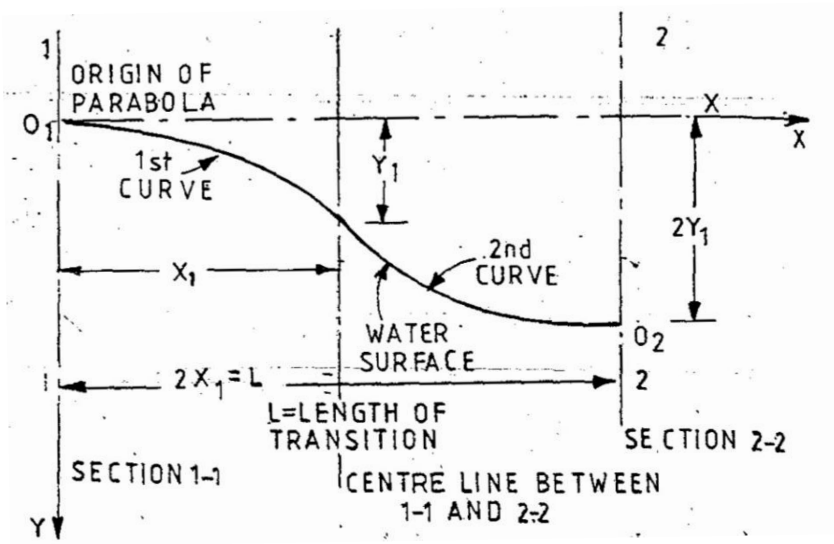


Figure 14. Water surface profile of transition contraction

The FSL at section 4-4 = Bed level at section 4-4 + y_4 = (known)

TEL at section 4-4=FSL at section 4-4 + $\frac{V_4^2}{2g} =$ (known)

Between section 3-3 and 4 4, there is an energy loss in the expansion, which is equal to 0.3 $\left(\frac{V_3^2 - V_4^2}{2g}\right)$

TEL at section 3-3 = TEL at section 4-4 (known) + 0.3$\left(\frac{V_3^2 - V_4^2}{2g}\right)$

As the trough dimensions at section 3-3 are known, V is also known and hence,

TEL at section 3-3 can be computed. Knowing TEL at 3-3; FSL at 3-3 can be calculated by subtracting $\frac{V_3^2}{2g}$ from TEL. Similarly, bed level at 3-3 can also be computed by

subtracting y_3 from FSL at 3-3.

Between sections 2-2 and 3-3, the channel flows in a trough of constant cross-section. The only loss in the trough (H_L) is the friction loss which can be computed with

Manning's formula, i.e.

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

$$Q = \frac{1}{n} A.R^{2/3}.\sqrt{\frac{H_L}{L}}$$

or $$H_L = \frac{Q^2.n^2.L}{A^2.\ R^{2/3}}$$

Adding this head loss H_L to TEL of section 3-3, the TEL at section 2-2 is obtained. The FSL at section 2-2 can then be obtained by subtracting $\frac{V_2^2}{2g}$ from TEL of 2-2. Similarly, the bed level at section 2-2 can be easily obtained by further subtracting y_2 from FSL at 2-2. Since the depth and velocity are constant in the trough, the TEL, FSL and bed lines are all parallel to each other from section 2-2 to 3-3. Between section 1-1 and 2-2, there is a loss of energy due to contraction. This loss is generally taken as equal to $0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$.

Thus the TEL at section 1-1 =TEL at section 2-2 + $0.2\left(\frac{V_2^2 - V_1^2}{2g}\right) = \text{known}$

Knowing TEL at section -1, FSL at 1-1 can be obtained by subtracting $\frac{V_1^2}{2g}$ from TEL at 1-1. Similarly, bed level at 1-1 can be obtained by subtracting y_1 from FSL at 1-1. The bed level, FSL and TEL having been determined at all the four sections, the total energy line may be drawn by assuming it to be a straight line between adjacent sections. The bed line may also be drawn straight between adjacent sections, provided the rise or fall in bed is small. However, if the change in bed level is considerable, the bed line in the transition section should be drawn as a smooth reverse curve, tangential to the bed lines at ends.

Example1. Design a suitable cross-drainage work, given the following data at the crossing of a canal and drainage.

Canal:	***Drainage:***
Full supply discharge = 32 cumecs	***High flood discharge = 300 cumecs***
Full supply level = R.L. 213.5	***High flood level = 210.0 m***
Canal bed level = R.L. 212.0	***High flood depth = 2.5 m***
Canal bed width = 20 m	***General ground level = 212.5 m.***

Trapezoidal canal section with 1.5 H: 1 V slopes.

Canal water depth = 1.5 m

Solution: Since the drainage is of large size, work of Type III will be adopted. Also, because the canal bed level (212.0) is much above the HFL of drainage (210.0), an aqueduct will be constructed. To affect economy, the canal shall be flumed.

Step 1: Design of Drainage Waterway:

Lacey's regime perimeter = P = 4.75 $\sqrt{Q}$ = 4.75 $\sqrt{300}$ = 82.3 m

Let the clear span between piers be 9 m and the pier thickness be 1.5 m.

Using 8 bays of 9 m each, clear waterway = $8 \times 9 = 72\ m$

Using 7 piers of 1.5 m each, length occupied by piers = $7 \times 1.5 = 10.5\ m$

Total length of waterway = 72 + 10.5 = 82.5 m

Step 2: Design of Canal Waterway:

Bed width of canal = 20.0 m

Let the width be flumed to 10.0 m

Providing a splay of 2: 1 in contraction, the length of contraction transition = $\frac{20-10}{2} \times 2 = 10.0\ m$

Providing a splay of 3: 1 in expansion, the length of expansion transition = $\frac{20-10}{2} \times 3 = 15.0\ m$

Length of the flumed rectangular portion of the canal between abutments = 82.5 m

In transitions, the side slopes of the canal section will be warped in plan from the original slope of 1.5: 1 to vertical.

Step 3: Head loss and bed levels at different sections:

At section 4-4

Area of trapezoidal canal section = (B + 1.5 y) y

$$= (20 + 1.5 \times 1.5)\ 1.5 = 22.5 \times 1.5 = 33.75\ m^2$$

Velocity = $V_4 = \left(\frac{Q}{A}\right) = \frac{32}{33.75} = 0.947$ m/sec

Velocity head = $\frac{V_4^2}{2g} = \frac{(0.947)^2}{2 \times 9{,}81} = 0.046$ m

R.L of bed at 4-4 = 212.0 m (given)

R.L of water surface at 4-4 = 212.0 + 1.5 = 213.5 m

R.L of TEL at 4-4 = 213.5 + 0.046 = 213.546 m

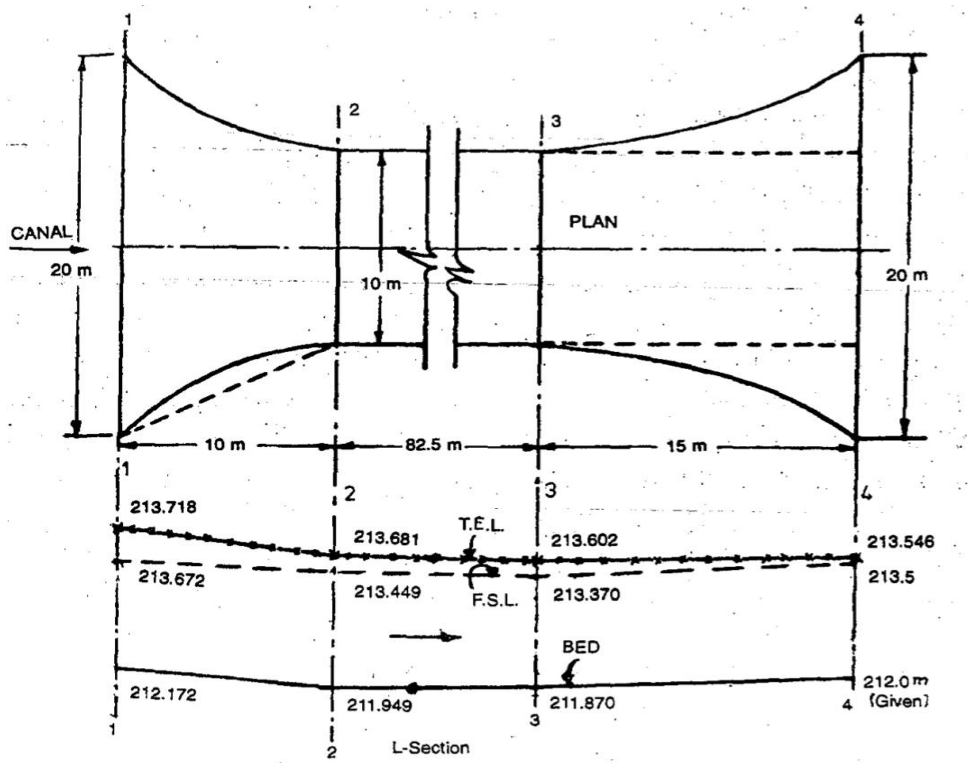


Figure 15. Plan and section of canal trough

At section 3-3

Keeping the same depth of 1.5 m throughout the channel, we have

Bed width = 10 m

Area of channel = $10 \times 1.5 = 15$ m^2

Velocity = V_3 = $\left(\frac{Q}{A}\right) = \frac{32}{15}$ = 2.13 m/sec

Velocity head = $\frac{V_3^2}{2g} = \frac{(2.13)^2}{2 \times 9{,}81} =$ 0.232 m

Assuming that the loss of head in expansion from section 3-3 to 4-4 is taken = 0.3 $\left(\frac{V_3^2 - V_4^2}{2g}\right)$

= 0.3 [0.232 – 0.046] = 0.056 m

R.L of TEL at section 3-3 = R.L of TEL at 4-4 + loss in expansion

= 213.546 + 0.056 = 213.602 m

R.L of water surface at 3-3 = R.L of TEL at 3-3 – velocity head

= 213.602 – 0.232 = 213.370 m

R.L of bed at 3-3 = 213.370 – 1.5 = 211.87 m

At section 2-2

From section 2-2 to 3-3, the trough section is constant. Therefore, area and velocity at 2-2 are same as at 3-3, there is a friction loss between 2-2 and 3-3 which is given by manning's formula

$$H_L = \frac{n^2.V^2.L}{R^{2/3}} = \frac{(0.016)^2 \times (2.13)^2 \times 82.5}{(1.16)^{2/3}} = 0.079 \text{ m}$$

R.L of TEL at section 2-2 = R.L of TEL at 3-3 + friction loss

= 213.602 + 0.079 = 213.681 m

R.L of water surface at 2-2 = R.L of TEL at 2-2 – velocity head

= 213.681 – 0.232 = 213.449 m

R.L of bed at 2-2 = 213.449 – 1.5 = 211.949 m

At section 1-1

Loss of head in contraction transition from 1-1 to 2-2 = $0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$

$$= 0.2\left(\frac{(2.13)^2 - (0.947)^2}{2 \times 9.81}\right) = 0.037 \text{ m}$$

R.L of TEL at section 1-1 = R.L of TEL at 2-2 + Loss in contraction

= 213.681 + 0.037 = 213.718 m

R.L of water surface at 1-1 = R.L of TEL at 2-2 – velocity head

= 213.718 – 0.046 = 213.672 m

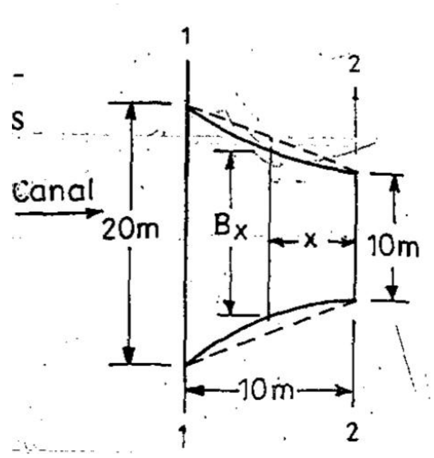
R.L of bed at 1-1 = 213.672 – 1.5 = 212. 172 m

Step 4: Design of Transitions:

(a) Contraction transition: Since the depth is kept constant, the transition can be designed on the basis of Mitra's method.

$$B_x = \frac{B_n . B_f . L_f}{L_f B_n - (B_n - B_f)x}$$

$$= \frac{20 \times 10 \times 15}{15 \times 20 - x\,(20-10)} = \frac{3000}{300-10\,x}$$



For various values of x lying between 0 to 10 m, various values of B_x are worked out by using the above equation as:

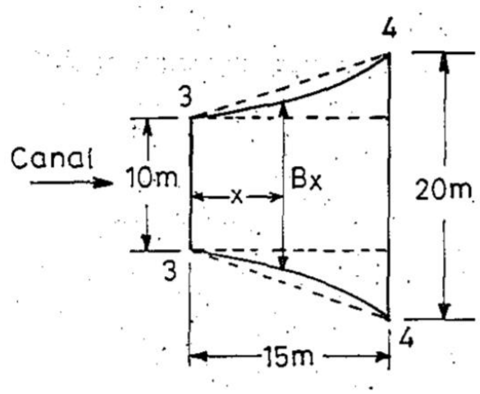
x in metres	0	2	4	6	8	10
$B_x = \frac{3000}{300 - 10\,x}$	10.0	11.11	12.5	14.29	16.67	20.0

The contraction transition can be plotted with these values.

(b) Expansion transition:

$$B_x = \frac{B_n . B_f . L_f}{L_f B_n - (B_n - B_f)x}$$

$$= \frac{20 \times 10 \times 15}{15 \times 20 - x\,(20-10)} = \frac{3000}{300-10\,x}$$



For various values of x lying between 0 to 15 m, various values of B_x are worked out by using the above equation as:

x in metres	0	2	4	6	8	10	12	14	15
$B_x = \frac{3000}{300 - 10\,x}$	10.0	10.71	11.54	12.5	13.64	15.0	16.67	18.75	20.0

The expansion transition can be plotted with these values.

Step 5: Design of Trough:

The trough shall be divided into two compartments of 5 m each and separated by an intermediate wall of 0.3 m thickness. The inspection road shall be carried on the top of left compartment as shown in figure below.

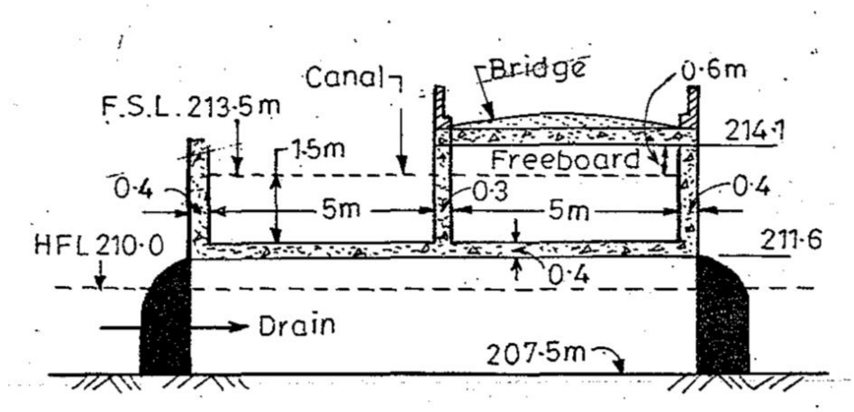


Figure 16

A freeboard of 0.6 m above the normal water depth of 1.5 m is sufficient, and hence the bottom level of bridge slab over the left compartment can be kept at 1.5 + 0.6 = 2.1 m above the bed level of trough. The entire trough section can be constructed in monolithic reinforced concrete and can be designed by usual structural methods.

Example2. Design a suitable cross-drainage work, given the following data at the crossing of two streams of water.

Irrigation channel:

Full supply discharge = 350 cumecs

Full supply level = R.L. 202.5

Canal bed level = R.L. 197.8

Canal bed width = 35 m

Side slopes = 1.5 H: 1 V

Full supply depth = 4.7 m

Natural Drainage:

Drainage bed level = 203.9 m

High flood level = 205.2 m

Catchment area of drainage up to cross-

-ing = 14.3 sq. km

The dickens formula may be used for computing H.F.Q. with its coefficient as 18.

Solution: The high flood discharge of the drainage at the point of crossing may be obtained by using Dickens formula:

$$Q = C.\ A^{3/2} = 18.\ (14.3)^{3/2} = 132 \text{ cumecs}$$

Since the bed level of drainage (203.9) is much above the canal FSL (202.5), the canal water can be taken below the drainage. Hence the CD work to be constructed at the crossing will be a super-passage. The design of the super-passage is to be done on the same lines as othat of an aqueduct.

Step 1: Design of canal waterway:

Flow velocity in the canal = $\frac{Discharge}{Area} = \frac{Q}{(B+0.5\ y)y} = \frac{350}{(35+0.5\ \times 4.7)\ 4.7} = 1.99$ m/sec.

This high velocity shows that the canal is already a lined canal, and much more lining cannot be affected. Hence the original bed width of 35 m can be continued as canal barrels below the drainage trough or slight fluming may be done. Let us adopt a clear waterway of 30 m in two spans, each of 15 m, with a central pier of width say 1.5 m, thus providing an overall linear waterway of 31.5 m between abutments, and this will be the length of drainage troughs.

Providing a splay of 2: 1 in contraction, the length of contraction transition = $\frac{35 - 31.5}{2} \times 2 = 5\ m$

Providing a splay of 3: 1 in expansion, the length of expansion transition = $\frac{35 - 31.5}{2} \times 3 = 7.5\ m$

Length of the flumed rectangular portion of the canal will be equal to the width of the drainage troughs = 50.5 m.

The piers, abutments, wing walls and return walls of the canal will be designed as those of a bridge taking the load of drainage trough (including the load of water and inspection road, etc) instead of a bridge deck slab.

Step 2: Design of drainage waterway:

Lacey's regime perimeter P = 4.75 $\sqrt{Q} = 4.75\sqrt{132} = 54.5\ m$

The total length of the waterway is generally chosen equal to P, although it can be slightly reduced to affect economy, but too much contraction of the drainage poses problems, and hence too much fluming is never done.

Let us provide 6 RCC compartments, each of clear width equal to 8 m, thus giving

Clear waterway = $8 \times 6 = 48\ m$

Using 5 partition walls of 0.3 m thick each, length occupied by walls = $5 \times 0.3 = 1.5\ m$

Therefore, total length of waterway provided = 48 + 1.5 = 49.5 m.

The two side walls of the RCC drainage trough may be kept 0.4 m thick each, with 49.5 m as aggregate waterway between them. Thus,

End to end length of drainage trough = 49.5 + 0.8 ≈ 50.5 m.

Thus, the length of the rectangular portion of the canal will also be equal to 50.5 m

Since the drainage has also been slightly flumed and kept lesser than P, so lets design its contraction and expansion lengths.

Assuming 2: 1 convergence, the length of contraction transition = $\frac{54.5 - 49.5}{2} \times 2 = 5\ m$

Providing a splay of 3: 1 in expansion, the length of expansion transition = $\frac{54.5 - 49.5}{2} \times 3 = 7.5\ m$

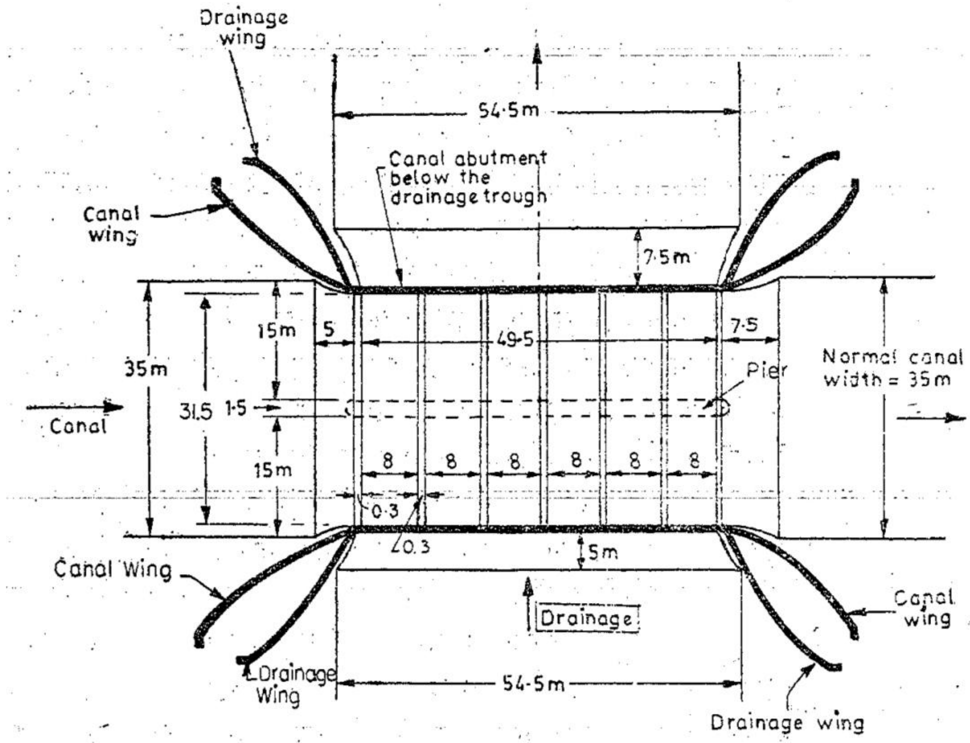


Figure 17. Inductive plan of super passage crossing

The wing walls will be constructed to reduce the drainage waterway width from 54.5 to 49.5 m on u/s and return walls will be constructed to expand the drainage waterway from 49.5 to 54.5 m on d/s. These wings will be extended so as to enter the berms of the drain.

The length of the drainage pucca rectangular trough will be equal to 31.5 m (i.e. equal to the rectangular waterway of canal).

Step 3: Head loss and Bed levels of different sections along the length of drainage trough:

At section 4-4

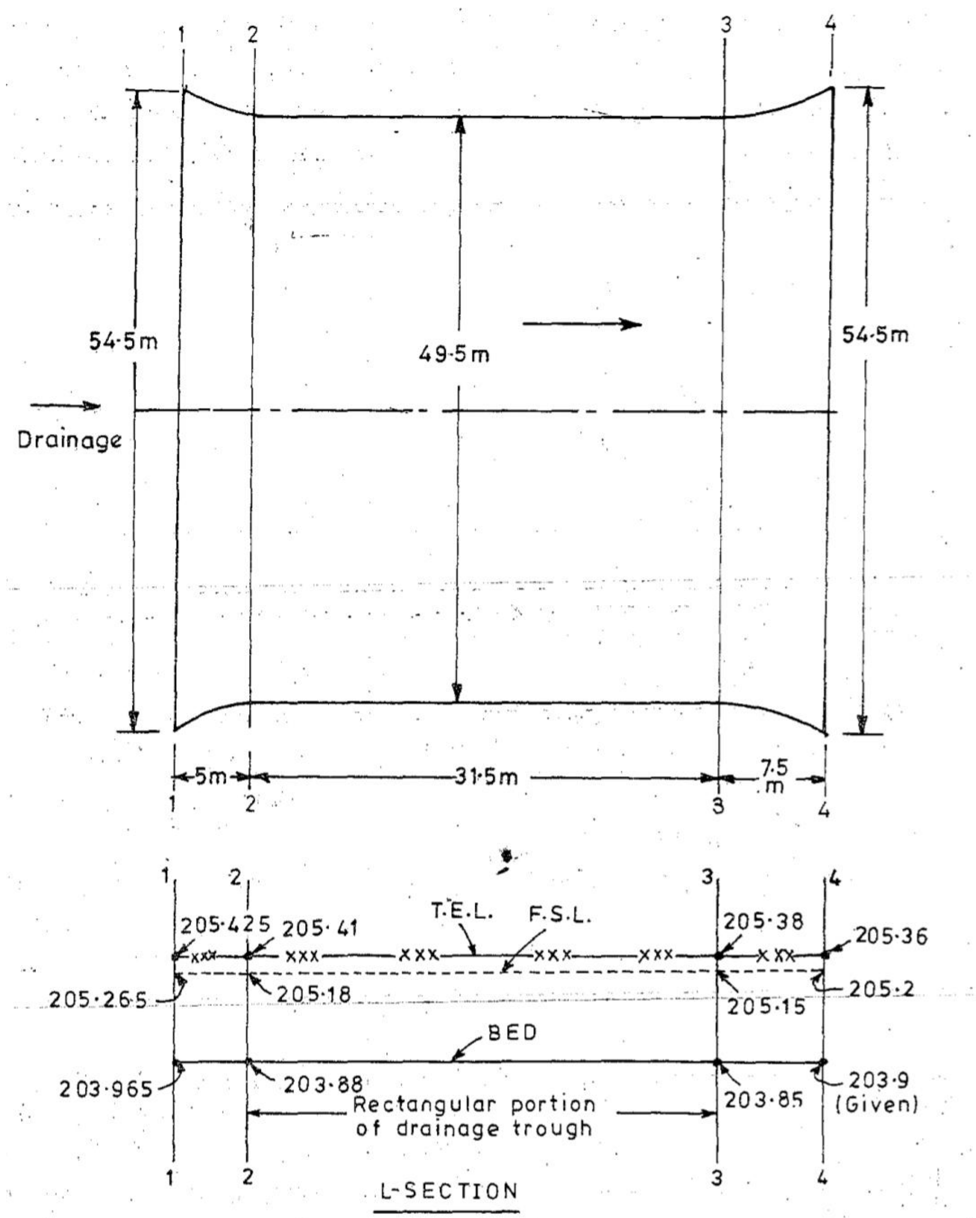


Figure 18. Plan and L-section of drainage trough carried over the canal

Area of natural drainage section = width (= perimeter) × depth

= 54.5 × (205.2 – 203.9) = 70.85 m^2

Velocity = $V_4 = \frac{Q}{A} = \frac{132}{70.85} = 1.86$ m/sec, say 1.9 m/sec

Velocity head = $\frac{V_4^2}{2g} = \frac{(1.9)^2}{2 \times 9.81} = 0.16$ m

R.L of bed at 4-4 = 203.9 m (given)

R.L of water surface at 4-4 = 203.9 + 1.3 = 205.2 m

R.L of TEL at 4-4 = 205.2 + 0.16 = 205.36 m

At section 3-3

Keeping the same depth of 1.3 m throughout the channel, we have

Clear waterway = 8 $\times$ 6 = 48 m

Area of flow = 48 $\times 1.3 = 62.4$ m^2

Velocity = V_3 = $\left(\frac{Q}{A}\right) = \frac{132}{62.4}$ = 2.12 m/sec

Velocity head = $\frac{V_3^2}{2g} = \frac{(2.12)^2}{2\times 9.81} =$ 0.23 m

Assuming that the loss of head in expansion from section 3-3 to 4-4 is taken = 0.3 $\left(\frac{V_3^2 - V_4^2}{2g}\right)$

= 0.3 [0.23 – 0.16] = 0.02 m

R.L of TEL at section 3-3 = R.L of TEL at 4-4 + loss in expansion

= 205.36 + 0.02 = 205.38 m

R.L of water surface at 3-3 = R.L of TEL at 3-3 – velocity head

= 205.38 – 0.23 = 205.15 m

R.L of bed at 3-3 = 205.15 – 1.3 = 203.85 m

At section 2-2

From section 2-2 to 3-3, the trough section is constant. Therefore, area and velocity at 2-2 are same as at 3-3, there is a friction loss between 2-2 and 3-3 which is given by manning's formula

$$H_L = \frac{n^2.V^2.L}{R^{2/3}} = \frac{(0.016)^2\times(2.12)^2\times 31.5}{(1.23)^{2/3}} = 0.0275 \text{ m, say } 0.03 \text{ m}$$

R.L of TEL at section 2-2 = R.L of TEL at 3-3 + friction loss

= 205.38 + 0.03 = 205.41 m

R.L of water surface at 2-2 = R.L of TEL at 2-2 – velocity head

= 205.41 – 0.23 = 205.18 m

R.L of bed at 2-2 = 205.18 – 1.3 = 203.88 m

At section 1-1

Loss of head in contraction transition from 1-1 to 2-2 = $0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$

= 0.2 [0.23 – 0.16] = 0.015 m

R.L of TEL at section 1-1 = R.L of TEL at 2-2 + Loss in contraction

= 205.41 + 0.015 = 205.425 m

R.L of water surface at 1-1 = R.L of TEL at 2-2 – velocity head

= 205.425 – 0.16 = 205.265 m

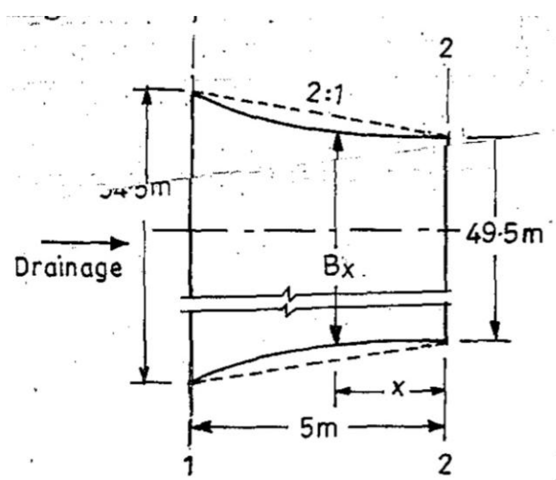
R.L of bed at 1-1 = 205.265 – 1.3 = 203.965 m

Step 4: Design of Transitions:

(a) Contraction transition: Since the depth is kept constant, the transition can be designed on the basis of Mitra's method.

$$B_x = \frac{B_n . B_f . L_f}{L_f B_n - \left(B_n - B_f\right)x}$$

$$= \frac{54.5 \times 49.5 \times 5}{5 \times 54.5 - x\ (54.5 - 49.5)} = \frac{2700}{54.5 - x}$$



For various values of x lying between 0 to 5 m, various values of B_x are worked out by using the above equation as:

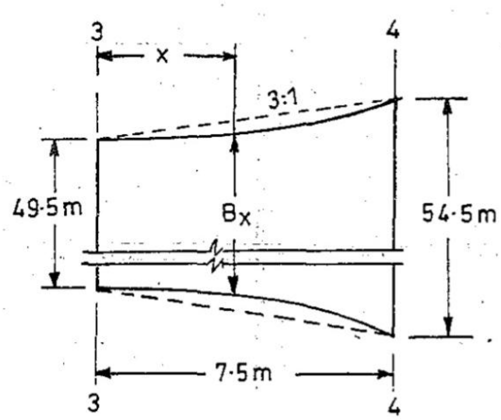
x in metres	0	1	2	3	4	5
$B_x = \frac{2700}{54.4 - x}$	49.5	50.5	51.5	52.5	53.5	54.5

The contraction transition can be plotted with these values.

(b) Expansion transition:

$$B_x = \frac{B_n . B_f . L_f}{L_f B_n - (B_n - B_f)x}$$

$$= \frac{54.5 \times 49.5 \times 7.5}{7.5 \times 54.5 - x\ (54.5 - 49.5)} = \frac{4050}{81.7 - x}$$



For various values of x lying between 0 to 7.5 m, various values of B_x are worked out by using the above equation as:

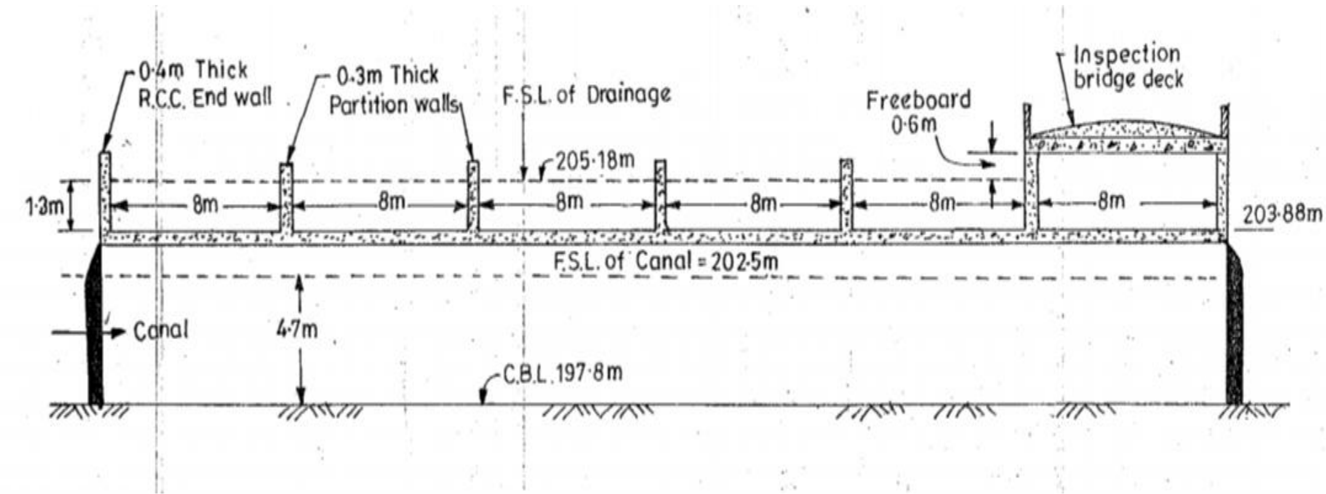
x in metres	0	2	4	6	7.5
$B_x = \frac{4050}{81.7 - x}$	49.5	50.7	52.0	53.3	54.5

The expansion transition can be plotted with these values.

Step 5: Design of Drainage Trough:

The RCC drainage trough has been divided into six compartments of 8 m each and separated by intermediate walls (5 in no.) of 0.3 m thickness. The end walls of the trough have tentatively been kept as 0.4 m wide each. The inspection road shall be carried on the top of end compartment as shown in fig.

A freeboard of 0.6 m above the normal water depth of 1.3 m is sufficient, and hence the bottom level of bridge slab over the end compartment can be kept at 1.3 + 0.6 = 1.9 m above the bed level of trough. The entire trough section can be constructed in monolithic reinforced concrete and can be designed by usual structural methods.



Example3. Design a syphon aqueduct for the following data:

Canal:	***Drainage:***
Full supply discharge = 50 cumecs	***Maximum flood discharge = 450 cumecs***
Full supply level = 201.80	***High flood level = 200.50***
Canal bed level = 200.00	***Bed level = 198.00***
Canal bed width = 36 m	***Natural ground level = 200.00***
Side slopes = 1.5 H: 1 V	
Water depth = 1.8 m	
Assume other data, if required.	

Solution: Step 1: Drainage waterway:

Lacey's equation: P = 4.75 $\sqrt{Q} = 4.75\sqrt{450}$ = 100.76 m

Let us provide 10 spans of 9 m each. Let the width of pier be 1.5 m.

Total waterway = 10 × 9 + 9 × 1.5 = 103 m > 100.76 m (O.K.)

Let the velocity in the barrel be 2 m/s.

Area of flow = 450/2 = 225 m^2

Height of barrel = $\frac{225}{10 \times 9} = 2.50\ m$

Provide 10 rectangular barrels, each 9m wide and 2.50 m high.

Step 2: Canal waterway:

Let the flumed width be 18 m. the trough carrying the canal is divided into three compartments, each 6 m wide. Let the intermediate walls be 0.4 m thick and the outer walls be 0.5 m thick. Let the bottom slab be 0.5 m thick. An inspection road 6 m wide shall be provided on the extreme left compartment.

Providing a free board of 0.5 m, the height of trough = 1.80 + 0.50 = 2.30 m

Bottom level of road slab = 200.00 + 1.80 + 0.75 = 202.55

Bottom level of trough slab = 200.00 – 0.50 = 199.50

Top level of impervious floor = 199.50 – 2.50 = 197.00

Step 3: Head loss through the barrel:

$$h = \left[1 + f_1 + f_2 \frac{L}{R}\right] \frac{V^2}{2g}$$

For a square entry, $f_1 = 0.505$

Also, for cement plaster, $f_2 = a\left(1 + \frac{b}{R}\right) = 0.00316\ \left(1 + \frac{0.030}{R}\right)$

R = hydraulic radius = $\frac{9 \times 2.5}{2\ (9+2.5)}$ = 0.98

Therefore, $f_2 = 0.00316\ \left(1 + \frac{0.030}{0.98}\right)$ = 0.0032

L = length of barrel = $6 \times 3 + 0.4 \ \times 2 + 0.5 \ \times 2 = 19.80\ m$

V = velocity through barrel = $\frac{450}{90 \times 2.50}$ = 2.0 m/s

Therefore, h = $\left[1 + \ 0.505 + \ 0.0032 \ \times \frac{19.80}{0.98}\right] \times \frac{2.0^2}{2 \times 9.81} = 0.31\ m$

u/s HFL = 200.50 + 0.31 = 200. 81.

Step 4: Uplift Pressure:

(a) Uplift pressure on the underside of trough:

The level of HGL after entry = 200.81 – 0.505 $\times \left(\frac{2^2}{19.62}\right) = 200.71$

Uplift head (h) = 200.71 – 199.50 = 1.21 m

Uplift pressure = w h = 1.21 $\times$ 9.81 = 11.87 kN/m^2

Self weight of concrete slab = 0.5 $\times$ 24 = 12 kN/m^2

As the downward weight is greater than the uplift pressure, it is not necessary to provide any top reinforcement and anchorage.

(b) Uplift pressure on the impervious floor:

Let us assume the thickness of floor as 0.80 m

Bottom level of floor = 197.00 – 0.80 = 196.20

Static head due to water table: When the water table rises to the bed level of drainage, static head (h_1) = 198.00 – 196.20 = 1.80 m

Seepage head: Maximum seepage head = FSL – Bed level of drainage = 201.80 – 198.00 = 3.80 m.

Let the length of impervious floor in the canal bed beyond the trough be 18 m. let the length of impervious floor in the drainage bed beyond the barrel be 5.50 m.

Total creep length = 18 + $\frac{9}{2} + \frac{19.80}{2} +$ 5.50 = 37.9 m

Residual seepage head upto the centre of span of the first barrel = $\frac{3.80}{37.9} \times$ 15.4 = 1.54 m

Total uplift head = 1.80 + 1.54 = 3.34 m

Self weight of floor = 0.8 × 24 = 19.2 kN/m^2

Uplift pressure = 3.34 × 9.81 = 32.77 kN/m^2

Net uplift pressure = 32.77 – 19.20 = 13.57 kN/m^2

The necessary reinforcement should be provided at the top of floor to resist this uplift pressure.

Step 5: Total length of impervious floor:

Length of floor under trough	= 19.80 m
Length of u/s cut water	= 1.00 m
Horizontal length of d/s ramp	= 5.00 m
Length of d/s cut water	= 0.50 m
Length of horizontal floor on u/s	= 1.00 m
Width of u/s cutoff	= 0.50 m
Total length	= $\boxed{27.80\text{ m}}$

Cutoff walls: Normal scour depth, R = 0.47 $\left(Q/f\right)^{1/3}$ = 0.47 $\left(450/1.0\right)^{1/3}$ = 3.60 m

Scour level on u/s = 200.81 – 1.5 ×3.60 = 195.40

Scour level on d/s = 200.50 – 2.0 ×3.60 = 193.30

Provide d/s cutoff wall up to a level of 193.30

Protection works: Depth of u/s cut-off wall from bed, d_1 = 198.00 - 195.40 = 2.60 m

Length of u/s protection = 2 d_1 = 5.20 m

Provide 0.40 m thick brick pitching. The pitching should be preceded by a toe wall, 0.4 m thick and 1.0 m deep at the u/s end.

Depth of d/s cut-off wall from bed, d_2 = 198.00 – 193.30 = 4.7 m

Length of d/s protection = 2 d_2 = 9.40 m

Provide 0.4 m thick brick pitching. The pitching should be followed by a toe wall, 0.4 m thick and 1.50 m deep at the d/s end.

Step 6: Design of canal transition:

Let us adopt a splay of 2:1 in contraction transition and a splay of 3:1 in expansion transition.

Length of contraction transition = $2 \times \left(\frac{36-18}{2}\right)$ = 18 m

Length of expansion transition = $3 \times \left(\frac{36-18}{2}\right)$ = 27 m

At section 4-4

Area of flow = (36 + 1.5 × 1.8) × 1.8 = 69.66 m^2

Velocity = $V_4 = \frac{Q}{A} = \frac{50}{69.66} = 0.72$ m/sec

Velocity head = $\frac{V_4^2}{2g} = \frac{(0.72)^2}{2 \times 9.81} = 0.026$ m

R.L of bed at 4-4 = 200.00 m (given)

R.L of water surface at 4-4 = 201.80

R.L of TEL at 4-4 = 201.80 + 0.026 = 201.826

At section 3-3

Area of flow = $18 \times 1.80 = 32.4$ m^2

Velocity = $V_3 = \left(\frac{Q}{A}\right) = \frac{50}{32.4} = 1.54$ m/sec

Velocity head = $\frac{V_3^2}{2g} = \frac{(1.54)^2}{2 \times 9.81} = 0.121$ m

Assuming that the loss of head in expansion from section 3-3 to 4-4 is taken = $0.3\left(\frac{V_3^2 - V_4^2}{2g}\right)$

$= 0.3\left(\frac{1.54^2 - 0.72^2}{19.62}\right) = 0.028$ m

R.L of TEL at section 3-3 = R.L of TEL at 4-4 + loss in expansion

=201.826 + 0.028 = 201.854 m

R.L of water surface at 3-3 = R.L of TEL at 3-3 – velocity head

= 201.854 – 0.121 = 201.733 m

R.L of bed at 3-3 = 201.733 – 1.8 = 199.933 m

At section 2-2

From section 2-2 to 3-3, the trough section is constant. Therefore, area and velocity at 2-2 are same as at 3-3, there is a friction loss between 2-2 and 3-3 which is given by manning's formula

$$H_L = \frac{n^2.V^2.L}{R^{2/3}} = \frac{(0.016)^2 \times (1.54)^2 \times 103.00}{(1.50)^{2/3}} = 0.037 \text{ m}$$

$$\left[R = \frac{18 \times 1.80}{18+2 \times 1.80} = 1.50 \text{ m}, L = 103.0 \text{ m}, n = 0.016\right]$$

R.L of TEL at section 2-2 = R.L of TEL at 3-3 + friction loss

= 201.854 + 0.037 = 201.891 m

R.L of water surface at 2-2 = R.L of TEL at 2-2 – velocity head

= 201.891 – 0.121 = 201.770 m

R.L of bed at 2-2 = 201.770 – 1.8 = 199.970 m

At section 1-1

Loss of head in contraction transition from 1-1 to 2-2 = $0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$

$$= 0.2\left(\frac{1.54^2 - 0.72^2}{19.62}\right) = 0.019 \text{ m}$$

R.L of TEL at section 1-1 = R.L of TEL at 2-2 + Loss in contraction

= 201.891 + 0.019 = 201.910 m

R.L of water surface at 1-1 = R.L of TEL at 2-2 – velocity head

= 201.910 – 0.026 = 201.884 m

R.L of bed at 1-1 = 201.884 – 1.8 = 200.084 m

Step 4: Design of Transitions:

Let us design the transitions by Chaturvedi's method.

(a) Contraction transition:

$$x = \frac{L.B_n^{3/2}}{B_n^{3/2} - B_f^{3/2}}\left[1 - \left(\frac{B_f}{B_x}\right)^{3/2}\right]$$

$$= \frac{18 \times (36)^{3/2}}{(36)^{3/2} - (18)^{3/2}}\left[1 - \left(\frac{18}{B_x}\right)^{3/2}\right]$$

or $$1 - 0.0359\,x = 76.37/(B_x)^{3/2}$$

or $$B_x = \left[\frac{76.37}{1 - 0.0359\,x}\right]^{2/3}$$

The widths B_x are calculated as:

$x\ (m)$	0	3	6	9	12	15	18
B_x	18	19.42	21.16	23.35	26.20	30.14	36

The contraction transition can be plotted with these values.

(a) Expansion transition:

$$x = \frac{L.B_n^{3/2}}{B_n^{3/2} - B_f^{3/2}}\left[1 - \left(\frac{B_f}{B_x}\right)^{3/2}\right]$$

$$= \frac{27 \times (36)^{3/2}}{(36)^{3/2} - (18)^{3/2}}\left[1 - \left(\frac{18}{B_x}\right)^{3/2}\right]$$

or $$1 - 0.0239\,x = 76.37/(B_x)^{3/2}$$

or $$B_x = \left[\frac{76.37}{1 - 0.0239\,x}\right]^{2/3}$$

The widths B_x are calculated as:

$x\ (m)$	0	3	6	9	12	15	18	21	24	27
B_x	18	18.91	19.96	21.15	22.55	24.20	26.19	28.65	31.77	36.00

The expansion transition can be plotted with these values.

Example4. Design a canal syphon for the following data:

Canal:

Full supply discharge = 50 cumecs

Full supply level = 202.30

Canal bed level = 200.00

Canal bed width = 20 m

Side slopes = 0.5 H: 1 V

Drainage:

Catchment area = 65 km^2

Dicken's coefficient = 5.0

Bed level = 201.00

High flood level = 203.50

Assume other data, if required.

Solution: The high flood discharge of the drainage at the point of crossing may be obtained by using Dickens formula:

$$Q = C. A^{3/2} = 5. (65)^{3/2} = 115 \text{ cumecs}$$

Step 1: Design of drainage waterway:

Lacey's regime perimeter P = $4.75 \sqrt{Q} = 4.75 \sqrt{115} = 50.93\ m$

Provide a bed width of 48 m. The drainage section will not be flumed.

Step 2: Design of canal waterway:

Flow velocity in the canal = $\frac{Discharge}{Area} = \frac{Q}{(B+0.5\ y)y} = \frac{50}{(20+0.5 \times 2.3)\ 2.3} = 1.03$ m/sec.

The minimum fluming ratio of the canal is 40%.

$$\text{Flumed width} = 0.4 \times 20 = 8.0 \text{ m}$$

Provide 2 barrels, 4 m wide each, with an intermediate wall 0.3 m thick.

Let us assume the height of the barrel as 2.5 m.

Velocity through the barrel = $\frac{50}{2\ (4 \times 2.5)}$ = 2.5 m/s

This is less than the maximum permissible velocity.

Froude no. of incoming flow, $F_1 = \frac{V}{\sqrt{g\ D_1}} = \frac{2.5}{\sqrt{9.81 \times 2.3}} = 0.53 < 1.00$

The flow in the barrel is subcritical. Therefore, the hydraulic jump will not form.

Step 3: Head loss and Bed levels of different sections along the length of drainage trough:

Width of the canal in the flumed section = 8.30 m

Let us provide 2:1 splay in contraction transition and 3:1 splay in expansion transition.

Length of contraction transition = 2 $\left(\frac{20-8.30}{2}\right)$ = 11.70 m

Length of expansion transition = 3 $\left(\frac{20-8.30}{2}\right)$ = 17.55 m

The drainage bed width is 48 m. To account for side slopes of the drainage, let us take the length of barrel as 60 m. In the transition, the side slopes of the canal shall be warped from a slope of 0.5: 1 to vertical.

Let us provide a water depth of 3 m at the exit of the barrel (i.e. at section 3-3 of the transition).

At section 4-4

Area of flow = (20 + 0.5 × 2.3) × 2.3 = 48.65 m^2

Velocity = $V_4 = \frac{Q}{A} = \frac{50}{48.65} = 1.03$ m/sec

Velocity head = $\frac{V_4^2}{2g} = \frac{(1.03)^2}{2 \times 9.81} = 0.054$ m

R.L of TEL at 4-4 = 202.30 + 0.054 = 202.354 m

R.L of water surface at 4-4 = R.L of TEL at 4-4 – velocity head

= 202.354 – 0.054 = 202.300 m

R.L of bed at 3-3 = 202.300 – 2.3 = 200.000 m

At section 3-3

Area of flow = $8.3 \times 3 = 24.9$ m^2

Velocity = $V_3 = \left(\frac{Q}{A}\right) = \frac{50}{24.9} = 2.01$ m/sec

Velocity head = $\frac{V_3^2}{2g} = \frac{(2.01)^2}{2 \times 9.81} = 0.206$ m

Assuming that the loss of head in expansion from section 3-3 to 4-4 is taken = 0.3 $\left(\frac{V_3^2 - V_4^2}{2g}\right)$

$$= 0.3\left(\frac{2.01^2 - 1.03^2}{19.62}\right) = 0.046 \text{ m}$$

R.L of TEL at section 3-3 = R.L of TEL at 4-4 + loss in expansion

= 202.354 + 0.046 = 202.400 m

R.L of water surface at 3-3 = R.L of TEL at 3-3 – velocity head

= 202.400 – 0.206 = 202.194 m

R.L of bed at 3-3 = 202.194 – 3 = 199.194 m

At section 2-2

R.L of TEL at section 2-2 = R.L of TEL at 3-3 + friction loss

= 202.400 + 0.280 = 202.680 m

R.L of water surface at 2-2 = R.L of TEL at 2-2 – velocity head

= 202.680 – 0.206 = 202.474 m

R.L of bed at 2-2 = 202.474 – 3 = 199.474 m

At section 1-1

Loss of head in contraction transition from 1-1 to 2-2 = 0.2$\left(\frac{V_2^2 - V_1^2}{2g}\right)$

$$= 0.2\left(\frac{2.01^2 - 1.03^2}{19.62}\right) = 0.031 \text{ m}$$

R.L of TEL at section 1-1 = R.L of TEL at 2-2 + Loss in contraction

= 202.680 + 0.031 = 202.711 m

R.L of water surface at 1-1 = R.L of TEL at 2-2 – velocity head

= 202.711 – 0.054 = 202.657 m

R.L of bed at 1-1 = 202.657 – 2.3 = 200.357 m

Step 4: Design of Transitions:

(a) Contracting transition

Length = 2 (20 – 8.3)/2 = 11.70 m

Drop in water levels = 202.657 – 202.474 = 0.183 m

2 x_1 = 11.70 m, or x_1 = 5.85;   2 y_1 = 0.183 m, or y_1 = 0.0915

Therefore,   C = (y_1/ x_1^2) = 0.0915/ $(5.85)^2$ = 2.67×10^{-3}

y = $(2.67 \times 10^{-3})\ x^2$

The values of y are calculated for x = 0, 3, 5.85 m measured from section 1-1 as well as section 2-2.

Origin at 1-1

x (m)	0	3	5.85
y (m)	0	0.024	0.092

Origin at 2-2

x (m)	0	3	5.85
y (m)	0	0.024	0.092

(b) Expanding transition

Length = 3 (20 – 8.3)/2 = 17.55 m

Drop in water levels = 202.300 – 202.194 = 0.106 m

2 x_1 = 17.55 m, or x_1 = 8.775; 2 y_1 = 0.106 m, or y_1 = 0.053

Therefore,   C = (y_1/ x_1^2) = 0.053/ $(8.775)^2$ = 6.883×10^{-4}

y = $(6.883 \times 10^{-4})x^2$

The values of y are calculated for x = 0, 3, 6 and 8.775 m

Origin at 3-3

x (m)	0	3	6	8.775
y (m)	0	0.0062	0.0248	0.0533

Origin at 4-4

x (m)	0	3	6	8.775
y (m)	0	0.0062	0.0248	0.0533

Bed levels:

(a) Contracting transition:

Bed level at 1-1 = 200.357, Bed level at 2-2 = 199.474

Drop in bed = 200.357 – 199.474 = 0.883 m

Drop per meter length = 0.883/11.7 = 0.0755 m

Assuming a linear drop, the bed levels are found as:

x (m)	0	3	5.85	8.70	11.70
Drop in bed (m)	0	0.226	0.441	0.657	0.883

(b) Expanding transition:

Rise in bed = 200.00 – 199.194 = 0.806 m

Rise per unit length = 0.806/17.550 = 0.0459 m

x (m)	0	3	6	8.775	11.550	14.55	17.55
Rise in bed (m)	0	0.138	0.276	0.403	0.530	0.668	0.806

Determination of bed widths:

As A = (B + r D) D, we have B = A/D – r D

The bed widths have been determined in the tables below:

(a) Contraction transition:

Distance from section 1-1 (m)	Drop/rise in water level (m)	Water surface elevation	TEL	Velocity head h_a (m)	Velocity V = $\sqrt{2\,g\,h_a}$	Side slope	Area of flow A = Q/V	Bed level	Depth D (m)	Bed width B = A/D – r D
0	0	202.657	202.711	0.054	1.030	0.5:1	48.550	200.357	2.300	20.00
3	0.024 from 1-1	202.633	202.703	0.070	1.172	0.372:1	42.662	200.131	2.502	16.120
5.85	0.092 from 1-1	202.565	202.696	0.130	1.597	0.25:1	31.309	199.916	2.649	11.157
8.70	0.024 from 2-2	202.498	202.688	0.190	1.931	0.128:1	25.893	199.700	2.798	8.896
11.70	0	202.474	202.680	0.206	2.010	0:1	24.900	199.474	3.000	8.300

(b) Expansion transition:

Distance from section 3-3 (m)	Drop/rise in water level (m)	Water surface elevation	TEL	Velocity head h_a (m)	Velocity V = $\sqrt{2\,g\,h_a}$	Side slope	Area of flow A = Q/V	Bed level	Depth D (m)	Bed width B = A/D – r D
0	0	202.194	202.400	0.206	2.010	0:1	24.900	199.194	3.000	8.300
3	0.0062 from 3-3	202.200	202.392	0.192	1.941	0.085:1	25.760	199.332	2.868	8.739
6	0.0248 from 3-3	202.219	202.384	0.165	1.799	0.171:1	27.793	199.470	2.749	9.640
8.775	0.0531 from 3-3	202.247	202.377	0.130	1.597	0.250:1	31.309	199.597	2.650	11.152
11.550	0.00248 from 4-4	202.275	202.370	0.095	1.365	0.329:1	36.630	199.724	2.551	13.520
14.550	0.0062 from 4-4	202.294	202.362	0.068	1.155	0.415:1	43.288	199.862	2.432	16.790
17.550	0	202.300	202.354	0.054	1.030	0.5:1	48.650	200.000	2.300	20.000

Step 5: Invert levels:

The invert levels of the barrel at the entrance and exit are kept the same as the bed levels at section 2-2 and 3-3 resp. Thus

Invert level at entrance = 199.47

Invert level at exit = 199.19

Invert level of the barrel under the base of drainage = Bed level of drain – thickness of concrete slab – earth fill over the slab – height of barrel

= 201.00 – 0.40 – 0.60 – 2.5 = 197.50

Therefore, the invert level of the barrel in the middle 48 m length would be kept as 197.50. The invert floors at the entry and exit shall be joined by providing suitable slopes in the floor of the barrel.

Step 6: Face walls:

Face walls are provided at the top of barrels at the entrance and exit of the barrels to support the outer slopes of drainage banks. The top level of the face walls is kept equal to that of the transition walls. The face walls are monolithic with RCC slab at the top of the barrels. These are designed as retaining walls.

Step 7: Impervious floor:

The length of the impervious floor of the barrel should be sufficient to provide a safe exit gradient. Let us provide the impervious floor in three-fourth length of the transition.

Length of u/s impervious floor = $0.75 \times 11.70 = 8.78$ m, say 9.0 m

Length of d/s impervious floor = $0.75 \times 17.55 = 13.16$ m, say 13.0 m

The impervious floor will be subjected to a max. uplift when high flood is passing in the drainage and the barrel is empty and the subsoil water table has risen up to the bed level of canal.

(a) Static head: <u>In the middle portion of the barrel</u>

Deepest invert level = 197.50

Let us assume the thickness of the barrel floor as 0.30 m

Bottom level of the barrel floor = 197.50 – 0.30 = 197.20

Static head = 200.00 – 197.20 = 2.80 m

<u>At the d/s end of the barrel</u>

Let us assume the thickness of floor as 1.50 m

Invert level = 199.19

Bottom level of the barrel floor = 199.19 – 1.50 = 197.69

Static head = 200.00 – 197.69 = 2.31 m

(b) Seepage head: Using Bligh's creep theory.

Total creep length L = half the barrel width + length of barrel below drainage banks + length of d/s impervious floor

= 2.0 + 6.00 + 13.0 = 21.0 m

Maximum seepage head = HFL in drain – bed level of canal

H_s = 203.50 – 200.00 = 3.50 m

Uplift pressure head at the centre of the barrel = $\left(\frac{3.50}{21.0}\right) \times 19.0 = 3.17$ m

Total uplift pressure head = 2.80 + 3.17 = 5.97 m

Thickness of floor required = 5.97/ 2.24 = 2.70 m

Uplift pressure due to seepage at the end of the barrel floor = $\frac{3.50}{21.0} \times 13.00 = 2.17$ m

Total uplift pressure = 2.31 + 2.17 = 4.48 m

Thickness of floor = 4.48/2.24 = 2.0 m

Provide 2.0 m thick cement concrete floor and reduce it to 1.0 m at the end of d/s floor.

The floor thickness at other points can be found in the same manner. A thickness of 1.5 m has been provided at the u/s end of barrel and reduced to 1.0 m at the end of u/s floor.

Cut-off walls and pitching:

Cut-off walls, 1 m deep and 0.5 m wide, have been provided at the u/s and d/s ends of the impervious floor. In the remaining parts of the u/s and d/s transitions, 0.4 m thick dry brick pitching is provided. Toe walls, 1.0 m deep and 0.4 m wide, are provided at the ends of the pitching.