14

Cross Drainage Works

14.1. Introduction

A cross drainage work is a structure which is constructed at the crossing of a canal and a natural drain, so as dispose of drainage water without interrupting the continuous canal supplies. In whatever way the canal is aligned, such cross drainage works generally become unavoidable. In order to reduce the cross drainage works, the artificial canals are generally aligned along the ridge line called water-shed. When once the canal reaches the watershed line, cross drainage works are generally not required, unless the canal alignment is deviated from the watershed line. However, before the watershed is reached, the canal which takes off from the river has to cross a number of drains, which move from the watershed towards the river, as shown in Fig. 14.1. At all such crossings c_1, c_2, c_3, c_4, etc. cross drainage works are required.

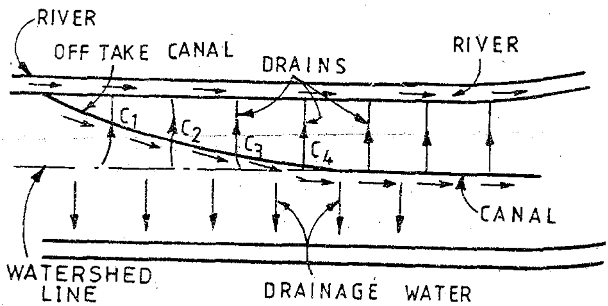


Fig. 14.1

A cross drainage work is generally a costly construction and must be avoided as far as possible. Since a watershed canal* crosses minimum number of drains, such an alignment is preferred to a contour canal which crosses maximum number of drains. The number of cross drainage works may also be reduced by diverting one drain into another and by changing the alignment of the canal, so that it crosses below the junction of two drains.

14.2. Types of Cross-drainage works

The drainage water intercepting the canal can be disposed of in either of the following ways :

(1) By passing the canal over the drainage. This may be accomplished either through (*i*) an *aqueduct;* or through a (*ii*) *syphon-aqueduct.*

(2) By passing the canal below the drainage. This may be accomplished either through (*i*) a *super-passage;* or through a (*ii*) *canal syphon* generally called a *syphon.*

(3) By passing the drain through the canal, so that the canal water and drainage water are allowed to intermingle with each other. This may be accomplished through (*i*) a *level crossing*; or through (*ii*) *inlets and outlets.*

* Also called a ridge canal.

All these different types of cross drainage works are described below in details.

14.2.1. Aqueduct and Syphon Aqueduct. In these works, the canal is taken over the natural drain, such that the drainage water runs below the canal (see Fig. 14.2) either freely or under syphoning pressure. *When the HFL of the drain is sufficiently below the*

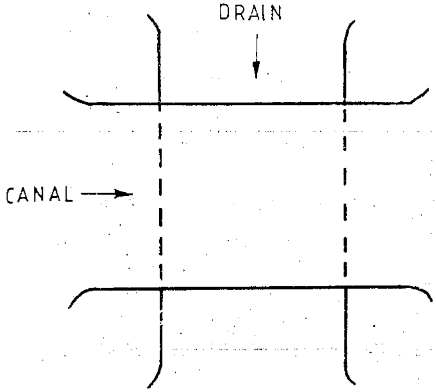


Fig. 14.2. Canal taken over the drain in an aqueduct or a syphon aqueduct (Line Plan of Crossing).

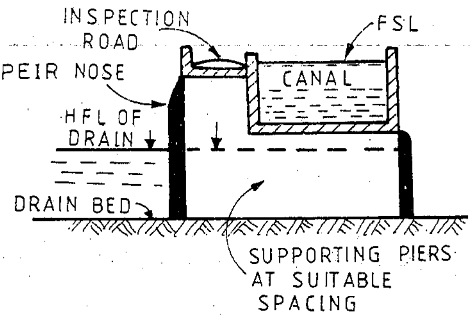


Fig. 14.3. Typical cross-section of an aqueduct.

bottom of the canal, so that the drainage water flows freely under gravity, the structure is known as an Aqueduct (Fig. 14.3). However, if the HFL of the drain is higher than the canal bed and the water passes through the aqueduct barrels under syphonic action, the structure is known as Syphon Aqueduct (See Fig. 14.4).

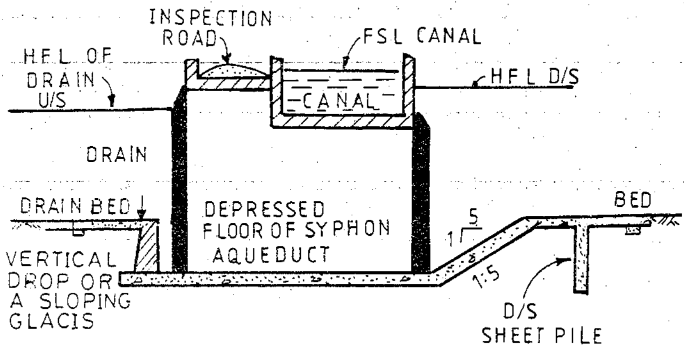


Fig. 14.4. Typical cross-section of a Syphon Aqueduct.

In this type of works, the canal water is taken across the drainage in a trough supported on piers. An inspection road is generally provided alongwith the trough, as shown. An aqueduct is just like a bridge except that instead of carrying a road or a railway, it carries a canal on its top. An aqueduct is provided when sufficient level difference is available between the canal and the natural drainage, and canal bed level is sufficiently higher than the torrent level. In Sirsa, a city near Roper in Punjab, an excellent aqueduct having 20 spans of about 13 m each has been constructed to carry a canal having bed width of 28 metres and a discharge of about 360 cumecs, with a torrent discharge of about 4300 cumecs. A difference of 3.3 metres was available between the bed level of canal and that of torrent in this case.

In the case of a syphon aqueduct, the drain bed is generally depressed and provided with pucca floor, as shown in Fig. 14.4. On the upstream side, the drainage bed may be joined to the pucca floor either by a vertical drop (when drop is of the order of 1 m) or by a glacis of 3 : 1 (when drop is more). The downstream rising slope should not be steeper than 5 : 1.

In this type of cross-drainage works (*i.e.* when the canal is taken over the drainage), the canal remains open to inspection throughout, and the damage caused by floods are rare. However, during heavy floods, the foundations of the work may be susceptible to scour; or waterway of the drain may get choked with debris, trees, etc.

14.2.2. Super-passage and syphon. In these works, the drain is taken over the canal such that the canal water runs below the drain (Fig. 14.5) either freely or under

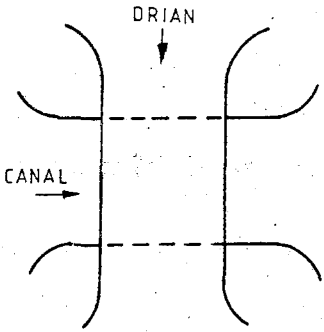


Fig. 14.5. Drain taken over the canal in a Superpassage or in a Syphon. (Line Plan of Crossing)

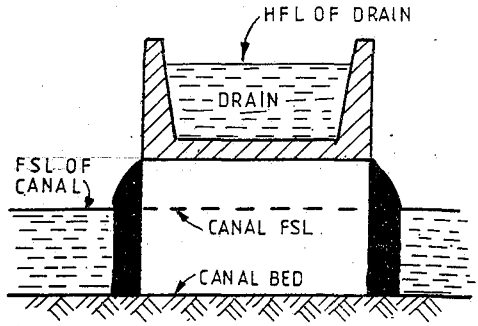


Fig. 14.6. Typical cross-section of a Superpassage.

syphoning pressure. *When the FSL of the canal is sufficiently below the bottom of the drain trough, so that the canal water flows freely under gravity, the structure is known as a Superpassage (Fig. 14.6). However, if the FSL of the canal is sufficiently above the bed level of the drainage trough, so that the canal flows under syphonic action under the trough, the structure is known as a canal syphon or a Syphon (Fig. 14.7).*

A superpassage is thus the reverse of an aqueduct, and similarly, a syphon is a reverse of an aqueduct syphon. However, in this type of cross-drainage works, the inspection road cannot be provided along the canal and a separate bridge is required for the road-way. For affecting economy, the canal may be flumed, but the drainage trough is never flumed.

In the case of a syphon, the canal bed is depressed and a ramp is provided at the exit (see Fig. 14.7) so that the trouble of silting is minimised.

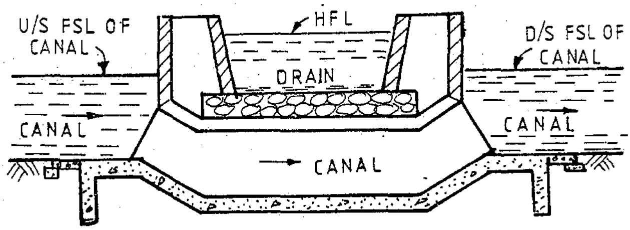


Fig. 14.7. Typical cross-section of a Canal Syphon (generally called a Syphon).

14.2.3. Level Crossing. In this type of cross-drainage work, the canal water and drain water are allowed to intermingle with each other. A level crossing is generally provided when a large canal and a huge drainage (such as a stream or a river) approach each other practically at the same level. A typical layout of a level crossing is shown Fig. 14.8.

A regulator is provided across the torrent (drainage) just on the downstream side of the crossing so as to control the discharge passing into the torrent. At the outgoing canal, a regulator is also provided so as control the discharge into the canal. A regulator at the end of the incoming canal is also sometimes required. The arrangement is practically the same as is provided on a canal headworks. This arrangement is generally provided when a huge sized canal crosses a large torrent carrying a very high but short lived* flood discharge. In this arrangement, the perennial drainage discharge is sometimes advantageously used, so as to augment the canal supplies.

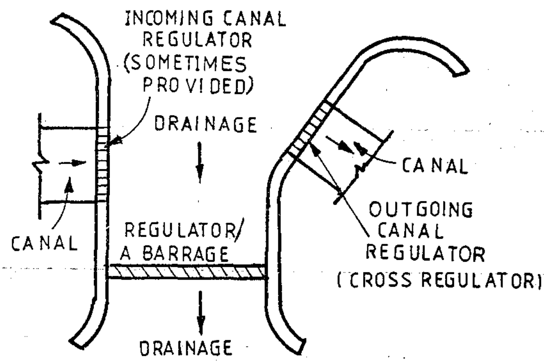


Fig. 14.8. Typical layout of a Level Crossing.

During dry season, when there are no floods, the torrent regulator is generally kept closed and the outgoing canal regulator is kept fully open, so that the canal flows without any interruption. During floods, however, the torrent regulator is opened so as to pass the flood discharge. A beautiful level crossing has been provided in Eastern U.P. under Sarda Sahayak Pariyojna ; where a canal carrying 370 cumecs crosses the Sarda river carrying a high flood discharge of the order of 10,000 cumecs.

14.2.4. Inlets and Outlets. An inlet is a structure constructed in order to allow the drainage water to enter the canal and get mixed with the canal water and thus to help in augmenting canal supplies. Such a structure is generally adopted when the drainage discharge is small and the drain crosses the canal with its bed level equal to or slightly higher than the canal F.S.L. Moreover, for the canal to remain in regime, the drain water must not admit heavy load of silt into the canal. Thus, in an inlet, the drainage water is simply added to the canal.

But, when the drainage discharge is high or if the canal is small, so that the canal section cannot take the entire drainage water, an outlet may sometimes be constructed to escape out the additional discharge at a suitable site, a little downstream along the canal. It is not necessary that the escaped discharge should be equal to the admitted discharge.

Similarly, it is also not necessary, that the number of inlets and outlets should be the same. There may be one outlet for two or three inlets. The outlet is generally combined with some other work where arrangement for escaping is in any case to be provided or may be added at a small extra cost.

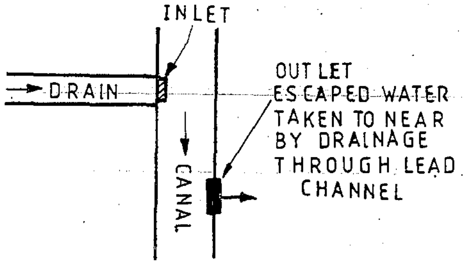


Fig. 14.9. Inlet and outlet (Plan).

An *inlet* essentially consists of an open cut in a canal bank, suitably protected by pitching, to admit the upland drainage

* When floods are intermittent and not continuous.

water into the canal. The bed and sides of the canal are also pitched for a certain distance upstream and downstream of the inlet. Similarly the *outlet* is another open cut in the canal bank with bed and sides of the cut properly pitched. The escaping water from the outlet is taken away by a lead channel to some nearby drain, on the downstream side of the surface outlet.

This type of cross-drainage work (*i.e.* those requiring intermingly of canal water with drainage water) are inferior to aqueduct or superpassage type of works, but they are cheaper. Hence, the aqueduct or superpassage type of works are generally used when high flood drainage discharge is large and continues for a sufficient time. A level crossing is used when the high flood drainage discharge is large but short lived. Inlets and outlets are used when the high flood drainage discharge is small.

14.3. Selection of a Suitable Type of Cross-Drainage Work

The relative bed levels, water levels, and discharge of the canal and the drainage are the primary factors which govern and dictate the type of cross drainage work that may prove to be most suitable at a particular place. For example, if the bed level of the canal is sufficiently above the HFL of the drain, an aqueduct is the first and obvious choice. But, if the bed level of the drain is sufficiently above the canal FSL, a superpassage may be constructed. Similarly, when a canal carries a small discharge compared to the drain, the canal may be taken below the drain by constructing a syphon, as against a syphon aqueduct which is adopted when the drain with smaller discharge can be taken below a large canal.

However, in actual field, such ideal conditions may not be available and the choice would then depend upon many other factors, such as :

(*i*) Suitable canal alignment.

(*ii*) Nature of available foundation.

(*iii*) Position of watertable and availability of dewatering equipment.

(*iv*) Suitability of soil for embankment.

(*v*) Permissible head loss in canal.

(*vi*) Availability of funds.

The relative bed levels of the canal and the drainage may be changed and manipulated by suitably changing the canal alignment, so that the point of crossing is shifted upstream or downstream of the drainage. For example, if the canal alignment is such that sufficient headway is not available between HFL of drain and bed of the canal, (although canal bed is higher) a syphon aqueduct has to be normally adopted. But, however, if other conditions (enumerated above) are not favourable for the construction of a syphon-aqueduct, the canal alignment may be changed so that the crossing is shifted to the downstream where drainage bed is low and thus sufficient headway becomes available for constructing an aqueduct in place of syphon aqueduct. The canal alignment is, therefore, finalised only after finalising the cross drainage works.

Compared to an aqueduct, a superpassage is inferior and should be avoided whenever possible. Similarly, a syphon-aqueduct (unless large drop in drainage bed is required) is superior to a syphon. A level crossing may become inevitable in certain cases. For example, when a large canal crosses a large torrent at almost equal bed levels, a level crossing may remain to be the only answer. An inlet may be adopted when a small drain crosses the canal with its bed level equal to canal FSL or slightly higher than it (*i.e.* bed conditions similar to those favouring the choice of a syphon or a superpassage). Inlets, though cheaper, are not preferred these days because their performance has not been very satisfactory.

14.4. Various Types of Aqueducts and Syphon-Aqueducts

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

Type I. In this type, the sides of the aqueduct are earthen bank with complete earthen slopes. The length of the culvert through which the drainage water has to pass under the canal should not only be sufficient to accommodate the water section of the canal but also the earthen banks of the canal with adequate slopes (Fig. 14.10).

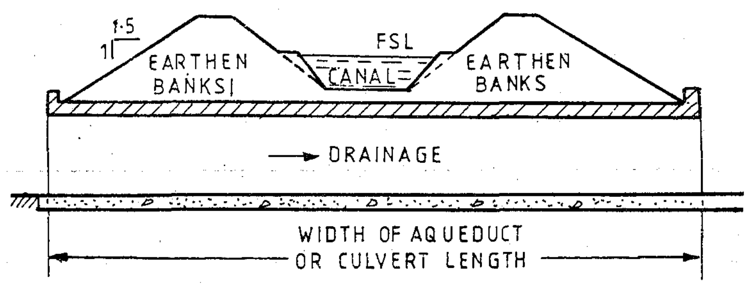


Fig. 14.10. Aqueduct (Type I)

Type II. In this type, the canal continues in its earthen section over the drainage, but the outer slopes of canal banks are replaced by retaining walls, thereby, reducing the length of the drainage culvert by that much extent (Fig. 14.11).

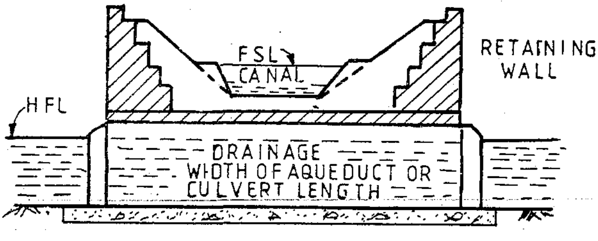


Fig. 14.11. Aqueduct (Type II)

Type III. In this type, earthen section of the canal is discontinued and the canal water is carried in a masonry or a concrete trough. The canal is generally flumed in this case, so as to effect economy in construction.

The culvert length or width of aqueduct is maximum in Type I and minimum in Type III. An intermediate value exists in Type II.

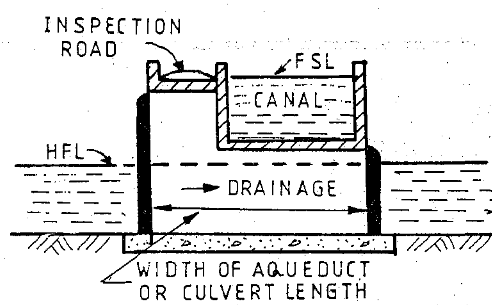


Fig. 14.12. Aqueduct (Type III)

Selection of the Suitable Type. *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 I, 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. On a very small drain, type III is most economical; while on a very wide drainage, type I is most economical. Type II is intermediate between type I and type III. The exact choice of a particular type in a particular case can be made by working out the cost of all the types and then choosing the cheapest.

14.5. 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 not much difference between them, except that the canal and the drainage are interchanged by each other.

14.5.1. 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.

14.5.2. 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

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

where P = is the watted perimeter in metres

Q = 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 the waterway required. However, no extra provision is generally made for the space occupied by piers. Hence, if the total waterway provided is equal to P, the effective or clear waterway will be less than P by as much extent as is occupied by pier widths. 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).

Size of the Barrels. After having fixed the waterway width & number of compartments (bays), the height of the drain barrels has to be fixed. In case of an aqueduct, the canal trough is carried clear above the drain HFL, and drain bed is not to be depressed. Hence, the height of bay openings is automatically fixed in aqueducts, as equal to the difference between HFL and DBL of drain.

However, in syphon-aqueducts, the required area of the drainage waterway can be obtained by dividing the drainage discharge by the permissible velocity through the barrels. This velocity through the barrels is generally limited to 2 to 3 m/sec. The waterway area is then divided by the decided waterway width of the drain openings, to compute the height of the openings, and the extent of depressed floor.

Due to the reduction in the width of the drainage, afflux is produced near the work site. The afflux will increase more and more, if the waterway is reduced more and more. The value of afflux is limited, so that there is no flooding of the country-side. The afflux may be calculated by using Unwin's formula as explained below in the following article.

14.5.3. Afflux and Head Loss through Syphon Barrels. It was stated earlier that 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*, given as :

$$h = \left[1 + f_1 + f_2 \frac{L}{R}\right] \frac{V^2}{2g} - \frac{V_a^2}{2g} \qquad \text{...(14.1)}$$

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

$f_2 \cdot \frac{L}{R} \cdot \frac{V^2}{2g}$; where f_2 is given as :

$$f_2 = a\left(1 + \frac{b}{R}\right) \qquad \text{...(14.2)}$$

where the values of a and b for different materials may be taken as given in Table 14.1.

Table 14.1

Material of the surface of barrel	a	b
Smooth iron pipe	0.00497	0.025
Encrusted pipe	0.00996	0.025
Smooth cement plaster	0.00316	0.030
Ashlar or brick work	0.00401	0.070
Rubble masonry or stone pitching	0.00507	0.250

*The total head loss consists of three losses, *i.e.*

Entry loss $= f_1 \frac{V^2}{2g}$, (*ii*) Friction loss $= \frac{f_2 L V^2}{2gR}$, (*iii*) Exit loss $= \frac{V^2}{2g}$.

After having fixed the velocity (V) through the barrels, the head (h) required to generate that much velocity can be found by using the equation (14.1).

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 premitting 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.

14.5.4. 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, where not specifically required and designed for, 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 so as to provide a smooth change from one stage to the other, so as to avoid sudden transition and the formation of eddies, etc. For this reason, the u/s or approach wings should not be steeper than $26\frac{1}{2}^\circ$ (*i.e.* 2 : 1 splay) and the d/s or departure wings should not be steeper than $18\frac{1}{2}^\circ$ (*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.

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).

(*iii*) 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 $X–X$, at a distance x from the flumed section (Fig. 14.13) is given by

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{L_f B_n - (B_n - B_f)\, x} \quad ...(14.3)$$

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.

Derivation of equation (14.3) is given below :

The above transition equation (*i.e.* equation 14.3) was derived on the basis that the rate of change of velocity per unit length of the transition remains constant throughout

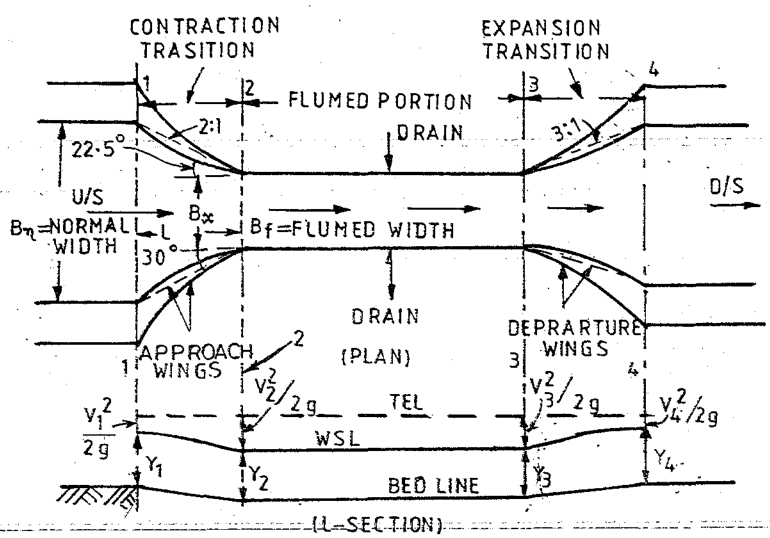


Fig. 14.13

the transition length. Thus, if V_n, V_f and V_x represent velocities at the corresponding sections, we have

$$\frac{V_f - V_x}{x} = \frac{V_f - V_n}{L_f} \quad ...(i)$$

Now, since depth y is assumed to be constant and the total discharge Q is also constant, we have

Velocity × Area = Discharge

$\therefore\ V_f \cdot B_f \cdot y = V_x \cdot B_x \cdot y = V_n \cdot B_n \cdot y = Q$ (assuming rectangular section throughout).

$$\therefore \quad V_f \cdot B_f = V_x \cdot B_x = V_n \cdot B_n$$

$$= \frac{Q}{y} = \text{constant} = K \text{ (say)}$$

or
$$V_f = \frac{K}{B_f}$$
$$V_x = \frac{K}{B_x}$$
$$V_n = \frac{K}{B_n}$$

Substituting these values in equation (*i*) we get

$$\left[\frac{\frac{K}{B_f} - \frac{K}{B_x}}{x}\right] = \left[\frac{\frac{K}{B_f} - \frac{K}{B_n}}{L_f}\right]$$

or
$$\frac{B_x - B_f}{B_f \cdot B_x \cdot x} = \frac{B_n - B_f}{L_f \cdot B_f \cdot B_n}$$

or
$$B_x \cdot L_f B_f \cdot B_n - L_f \cdot B_f^2 \cdot B_n = B_n B_f B_x \cdot x - B_f^2\, B_x \cdot x$$

or
$$B_x \cdot L_f \cdot B_f \cdot B_n - B_n \cdot B_f \cdot B_n \cdot x + B_f^2 \cdot B_x \cdot x = L_f \cdot B_f^2 \cdot B_n$$

or
$$B_x \cdot B_f \left[L_f \cdot B_n - B_n \cdot x + B_f \cdot x\right] = L_f \cdot B_f^2 \cdot B_n$$

or
$$B_x \left[L_f \cdot B_n - x\,(B_n - B_f)\right] = L_f \cdot B_f \cdot B_n$$

or
$$B_x = \left[\frac{L_f \cdot B_f \cdot B_n}{L_f \cdot B_n - x\,(B_n - B_f)}\right] \qquad \text{...(14.3)}$$

This is the required equation (14.3).

(*ii*) **Chaturvedi's Semi-Cubical Parabolic Transition when water depth remains constant.** Prof. R.S. Chaturvedi, Head of Civil Engineering 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.

$$x = \frac{L \cdot B_n^{3/2}}{B_n^{3/2} - B_f^{3/2}}\left[1 - \left(\frac{B_f}{B_x}\right)^{3/2}\right] \qquad \text{...(14.4)}$$

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

(*iii*) **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 depth in the flumed and unflumed portions are the same, or when these depths are different.

In Fig. 14.13, the contraction transition (*i.e.* the approach transition) starts at section 1-1 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.

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

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

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

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

As the trough dimensions at section 3-3 are known, V_3 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 substracting 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.*,

$$\left(Q = \frac{1}{n} A \cdot R^{2/3} \cdot S^{1/2}\right)$$

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

or
$$H_L = \frac{Q^2 \cdot n^2 \cdot L}{A^2 \cdot R^{4/3}}.$$

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 substracting 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

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

Knowing TEL at section 1-1, 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. The corners should, however, be rounded off in this case. 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.

Water surface in Transition : In the contraction transition between section 1-1 to 2-2, there will be a drop in water surface due to the drop in energy line and also due to the increased velocity head at 2-2. This drop in water surface has to be negotiated by a smooth curve tangential at both ends. This can be easily accomplished by using two parabolic curves meeting tangentially at the centre of the transition, as shown in Fig. 14.14.

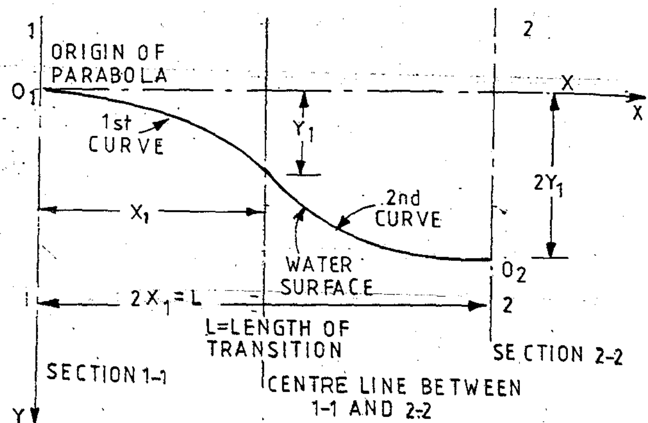


Fig. 14.14. Water Surface profile for Transition Contraction.

Let $2X_1 = L =$ The length in which fluming has been done.

$2Y_1 =$ Total difference in water levels between section 1-1 and 2-2.

The distance of the middle point of transition will be X_1 and drop in water surface will be Y_1. The equation of the first parabolic curve, with origin at water surface of section 1-1 (*i.e.* O_1) is given by

$$Y = C \cdot X^2$$

when $Y = Y_1$, and $X = X_1$

$$C = \frac{Y_1}{X_1^2}$$

Therefore, the equation of parabola becomes

$$Y = \left[\frac{Y_1}{X_1^2}\right] X^2 \qquad \text{...(14.5)}$$

Using the above equation, the first parabolic curve can be easily plotted. Similarly, the second parabolic curve can be plotted by taking the origin at O_2 on section 2-2.

The water surface in the expansion transition between sections 3-3 and 4-4 can also be plotted in a similar fashion, where there will be a rise in the water surface from section 3-3 to 4-4, as shown in Fig. 14.13.

After having plotted the water surface profile over the entire length, the velocity head say (h_v) can be found by measuring the vertical distance between TEL and water surface line at any point. The velocity head can then be converted into equivalent velocity by using $V = \sqrt{2g \cdot h_v}$. Hence, the velocity at each point can be known. The cross-sectional area required to pass the given discharge at each point can be found by dividing discharge by velocity at that point $\left(i.e.\ A = \frac{Q}{V}\right)$.

In trapezoidal channel of water depth y, the bed width B, and side slopes $s : 1$; area is given by

$$A = BD + s \cdot y^2. \qquad ...(14.6)$$

In flared wings, the side slopes are generally brought to vertical from an initial slope of $s : 1$ and, therefore, the side slopes at any point can be interpolated in proportion to the length of transition undergone. Thus at any point A, y and s are known and hence the value of B can be worked out at this pt. by using the equation (14.6). The width of the canal at various points in the transition can thus be determined. Hence, all the dimensions of the transition are fully found out.

14.5.5. 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 Fig. 14.15.

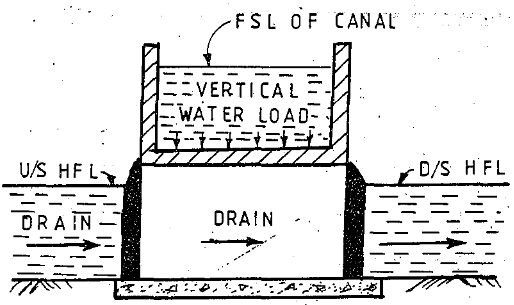


Fig. 14.15

Since in an aqueduct, there is no uplift from the underside acting on the bottom of canal bed, the canal bottom has to be designed for taking the dead weight and full water load (when canal is running full) by either providing a thickness sufficient to take this much of load merely by gravity or by providing a 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.

The amount of the uplift pressures exerted by the drain water on the roof of the culvert can be evaluated by drawing the hydraulic gradient line (H.G. line), as shown Fig. 14.16.

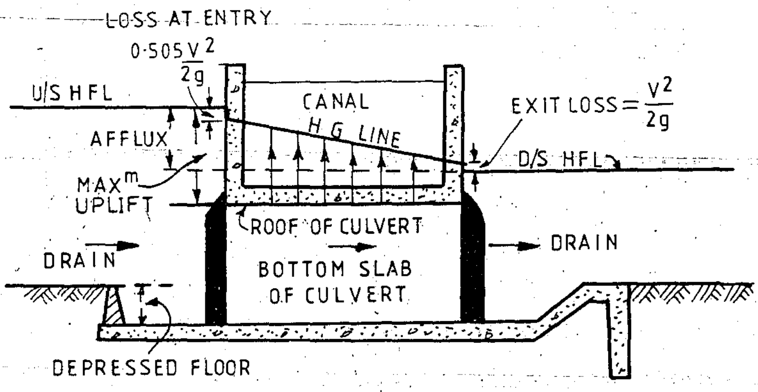


Fig. 14.16

The uplift pressure at any point under the roof of the culvert will be equal to the vertical ordinate between the hydraulic gradient line and the underside of the canal trough at that point, as shown in Fig. 14.16. From this uplift diagram of Fig. 14.16, it is very evident that the maximum uplift occurs at the upstream end point near the entry. The uplift pressure on the underside of the trough at the upstream end will be =[u/s water level – Entry loss – the level of the underside of the trough]. The slab thickness should be designed to withstand this maximum uplift.

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. Hence, in such a case, the thickness of the roof slab is generally provided from the considerations of water load and the remaining uplift is resisted by bending by providing top reinforcement in the roof slab. In such a case, the roof will have to be anchored to the bottom slab through piers by steel bolts, etc. so as to provide necessary end reactions for upward bending. A typical anchoring arrangement is shown in Fig. 14.17. *A better alternative may be to provide full fledged R.C.C. box culvert.*

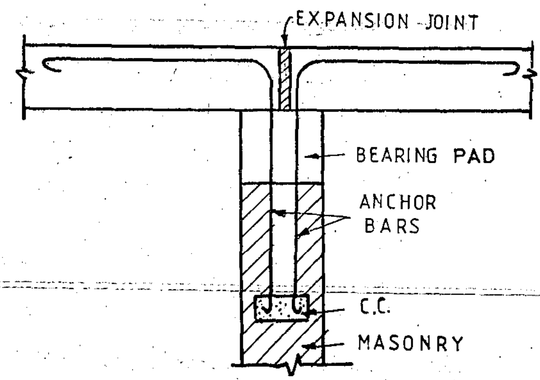


Fig. 14.17. Typical details of anchoring.

14.5.6. 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 watertable 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 Fig. 14.18.

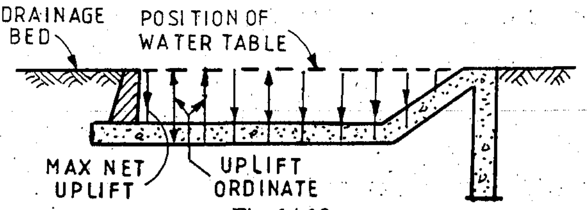


Fig. 14.18

(*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, is very complex and difficult, due to the fact that the flow takes place in three dimensional flownet. 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, for 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 :

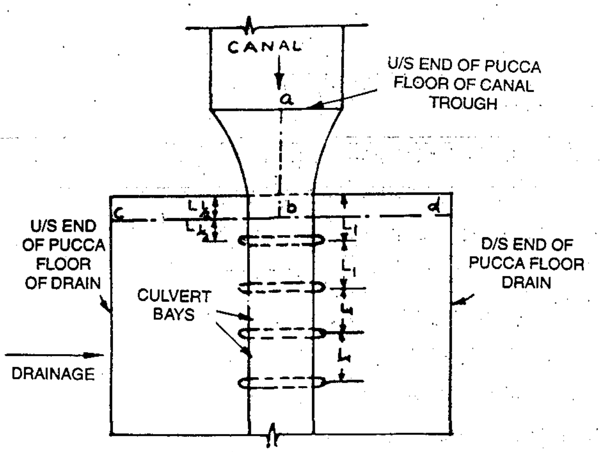


Fig. 14.19

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 Fig. 14.19. Point *b* is the point under the centre of the floor of the first culvert bay.

The seepage path from *a* to *b* and from *b* to *c* can be known. The total creep length will then be equal to $= ab + bc$. 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

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

or

$$H_b = \left[\frac{H}{ab+bc} \times bc\right] \quad ...(14.7)$$

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.

14.5.7. 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 upto 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 Examples

Example 14.1. *Design a suitable cross-drainage work, given the following data at the crossing of a canal and a drainage.*

Canal

Full supply discharge	= *32 cumecs*
Full supply level	= *R.L. 213.5*
Canal bed level	= *R.L. 212.0 m.*
Canal bed width	= *20.*

Trapezoidal canal section with $1\frac{1}{2}$ *H : 1 V slopes.*

Canal water depth	= *1.5 m.*

Drainage

High flood discharge	= *300 cumecs.*
High flood level	= *210.0 m.*
High flood depth	= *2.5 m.*
General ground level	= *212.5 m.*

Solution. Since the drainage is of a large size, work of type III will be adopted. Further, because the canal bed level (212.0 m) is much above the H.F.L. of drainage (*i.e.* 210.0 m) an **aqueduct** will be constructed. The earthen banks of the canal will be discontinued and the canal water taken in a concrete trough. For effecting economy, the canal shall be flumed.

Step 1. Design of Drainage Waterway

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

where Q = High flood discharge of drain = 300 cumecs (given)

$$P = 4.75 \cdot \sqrt{300} = 82.3 \text{ m.}$$

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 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

$$= \frac{20-10}{2} \times 2 = \mathbf{10.0\ m}$$

Providing a splay of 3 : 1 in expansion, the length of expansion transition

$$= \frac{20-10}{2} \times 3 = \mathbf{15\ m}$$

Length of the flumed rectangular portion of the canal between abutments = 82.5 m (provided).

In transitions, the side slopes of the canal section will be warped in plan from the original slope of $1\frac{1}{2}$: 1 to vertical.

Step 3. Head loss and bed levels at different sections. (Fig. 14.20).

At Section 4-4

At section 4-4, where the canal returns to its normal section, we have

Area of trapezoidal canal section

$$= (B + 1.5y)\, y$$
$$= (20 + 1.5 \times 1.5)\, 1.5 = 22.5 \times 1.5 = 33.75 \text{ m}^2$$

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

$$\text{Velocity head} = \frac{V_4^2}{2g} = \frac{(0.947)^2}{2 \times 9.81} = 0.046 \text{ m}$$

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

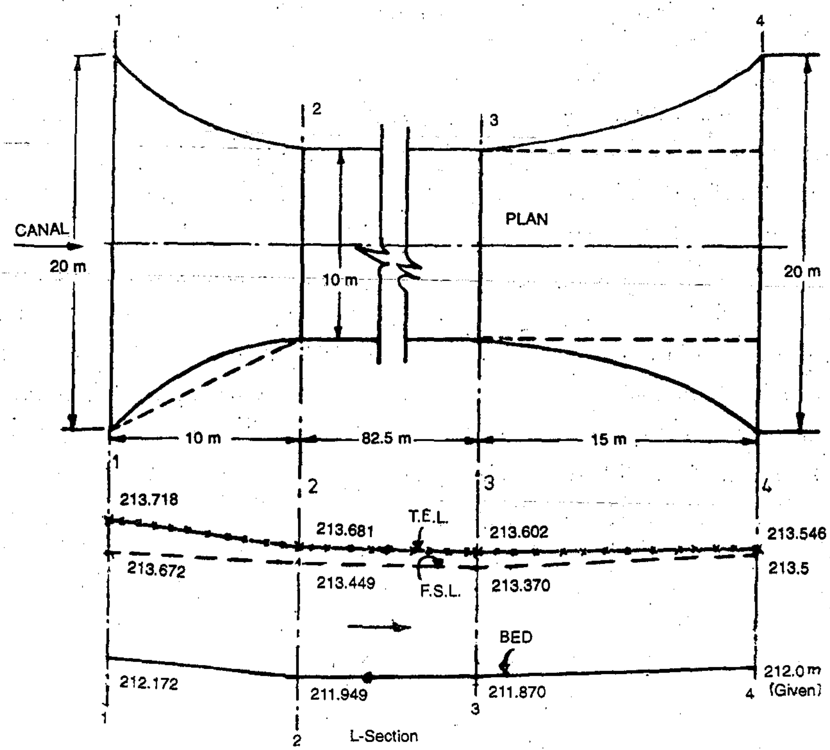


Fig. 14.20. Plan and Section of Canal Trough in Example 14.1.

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

R.L. of T.E.L. at 4.4 $= 213.5 + 0.046 = 213.546$ m

The known condition of 4-4 shall now be utilised for finding the bed levels etc. at 3.3.

At Section 3-3

Keeping the same depth of 1.5 m throughout the channel, we have at section 3.3 :

Bed width $= 10$ m

Area of channel $= 10 \times 1.5 = 15$ sq m

$$\text{Velocity} = V_3 = \frac{32}{15} = 2.13 \text{ m/sec}$$

$$\text{Velocity head} = \frac{V_3^2}{2g} = \frac{(2.13)^2}{2 \times 9.81} = 0.232 \text{ m}$$

Assuming that the loss of head in expansion from section 3-3 to section 4-4 is taken as

$$= 0.3 \left[\frac{V_3^2 - V_4^2}{2g} \right]$$

$$= 0.3\,[0.232 - 0.046]$$

$$= 0.3 \times 0.186 = 0.0558 \text{ m} \text{ ; say } \mathbf{0.056\ m}$$

R.L. of T.E.L. at section 3-3 = R.L. of T.E.L. at 4-4 + Loss in expansion

$$= 213.546 + 0.056 = 213.602 \text{ m}$$

$\therefore$ R.L. of water surface at 3-3 = R.L. of T.E.L. at 3-3 – Velocity Head

$$= 213.602 - 0.232 = \mathbf{213.370\ m}$$

R.L. of bed at 3.3

$$= 213.370 - 1.5 = \mathbf{211.87\ m}$$

At Section 2-2

From section 2-2 to 3-3, the trough section is constant. Therefore, area and velocity at 2-2 are the same as at 3-3. But from 2-2 to 3-3, there is a friction loss between 2-2 and 3-3 which may be computed by Manning's formula as equal to

$$H_L = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}}.$$

where n is rugosity coefficient whose value in concrete trough may be taken as 0.016; and L is the length of trough = 82.5 m.

Area of trough section $(A) = 10 \times 1.5 = 15$ sq m

Wetted perimeter $(P) = 10 + 2 \times 1.5 = 13$ m

Hydraulic mean depth $(R) = \frac{A}{P} = \frac{15}{13} = 1.16$ m

Velocity in trough $= \frac{Q}{A} = \frac{32}{15} = 2.13$ m/sec

$$\therefore \quad H_L = \frac{(0.016)^2 \times (2.13)^2 \times 82.5}{(1.16)^{4/3}}$$

$$= 0.0787 \text{ m; say } \mathbf{0.079\ m}$$

R.L. of T.E.L. at 2-2 = R.L. of T.E.L. at 3-3 + Friction loss in trough

$$= 213.602 + 0.079 = 213.681 \text{ m}$$

R.L. of water surface at 2-2

$$= 213.681 - 0.232 = \mathbf{213.449\ m}$$

R.L. of bed at 2-2

$$= 213.449 - 1.5 = \mathbf{211.949\ m}$$

At Section 1-1

Loss of head in contraction transition from 1-1 to 2-2

$$= 0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$$

$$= 0.2\left[\frac{(2.13)^2 - (0.947)^2}{2 \times 9.81}\right]$$

$$= 0.2\,[0.232 - 0.046] = \mathbf{0.037\ m}$$

R.L. of T.E.L. at 1-1 = R.L. of T.E.L. at 2-2 + Loss in contraction

$$= 213.681 + 0.037 = \mathbf{213.718\ m}$$

R.L. of water surface at 1-1

$$= 213.718 - 0.046 = \mathbf{213.672\ m}$$

R.L. of bed at 1-1

$$= 213.672 - 1.5 = \mathbf{212.172\ m}$$

All the bed levels, F.S.L. and T.E.L. are plotted in Fig. 14.20.

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 Hyperbolic transition equation (14.2) given as :

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

where $B_f = 10$ m

$B_n = 20$ m

$L_f = 10$ m

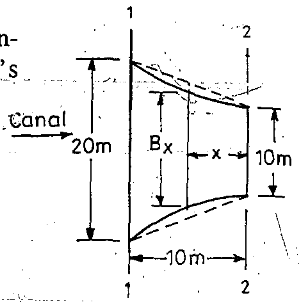


Fig. 14.21

Substituting we get

$$B_x = \frac{20 \times 10 \times 10}{10 \times 20 - x\,(20 - 10)} = \frac{2{,}000}{200 - 10x}$$

For various values of x lying between 0 to 10 m, various values of B_x are worked out, as shown below in Table 14.2. The distance x is measured from flumed section *i.e.* 2-2, as shown in Fig. 14.21.

Table 14.2

x *in metres*	0	2	4	6	8	10
$B_x = \dfrac{2{,}000}{200 - 10x}$ *in metres*	10.0	11.11	12.5	14.29	16.67	20.0

The contraction transition can be plotted with these values.

Expansion Transition. In this case $B_n = 20$ m, $B_f = 10$ m, and $L_f = 15$ m.

Using Eqn. (14.2), we get

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

$$= \frac{20 \times 10 \times 15}{15 \times 20 - x\,(20 - 10)} = \frac{3{,}000}{300 - 10x}.$$

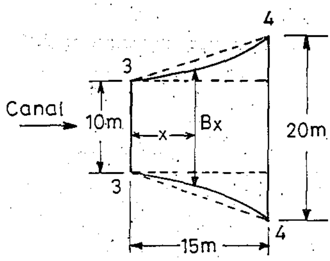


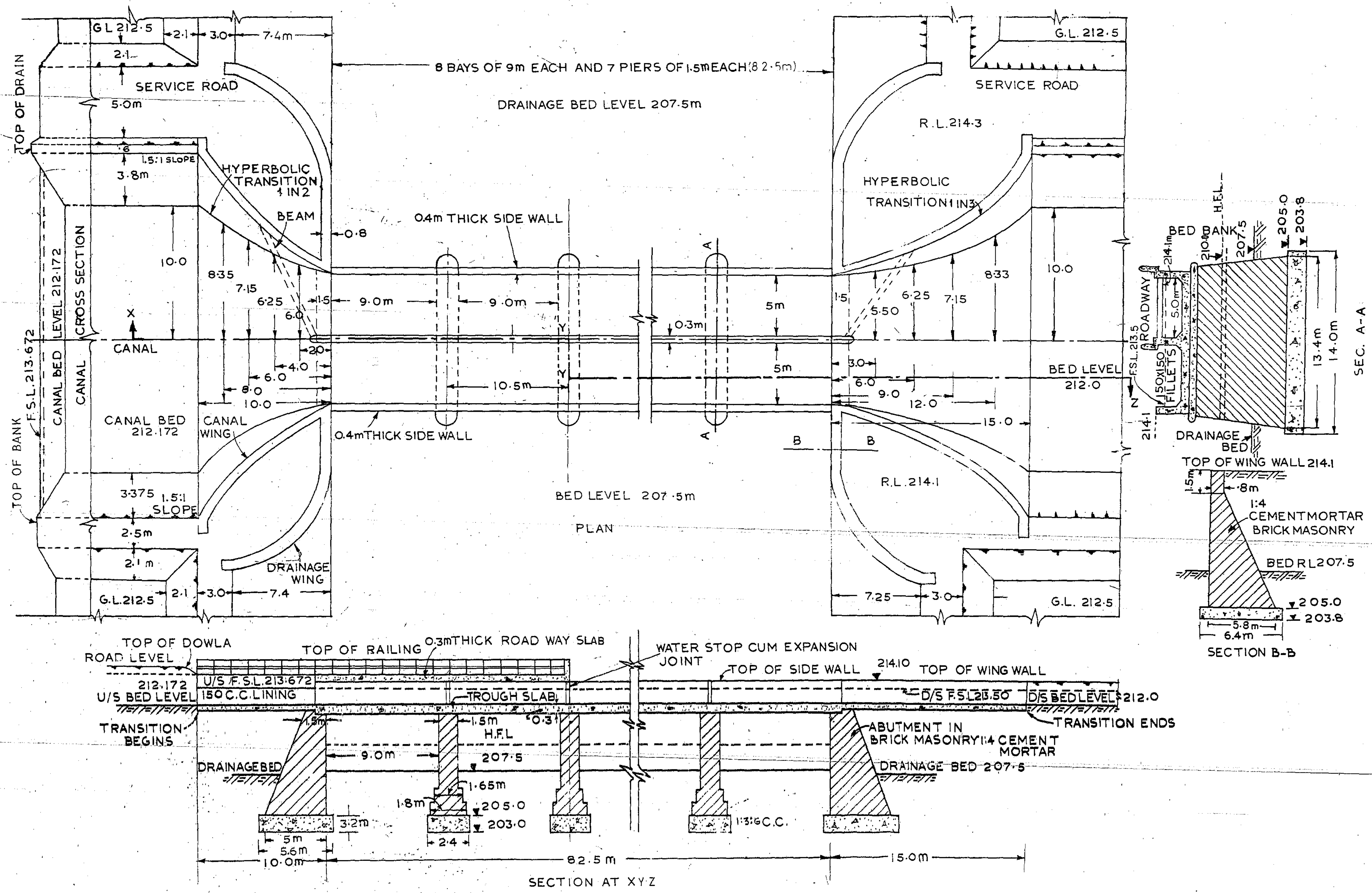
Fig. 14.22

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 shown in Table 14.3.

Table 14.3

x *in metres*	0	2	4	6	8	10	12	14	15
$B_x = \dfrac{3{,}000}{300 - 10x}$ *in metres*	10.0	10.71	11.54	12.5	13.64	15.0	16.67	18.75	20.0

The expansion transition can be easily plotted with these values.



DETAILS OF THE AQUEDUCT (FIG. 14.24.)

Step 5. Design of Trough

The trough shall be divided into two equal 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 Fig. 14.23.

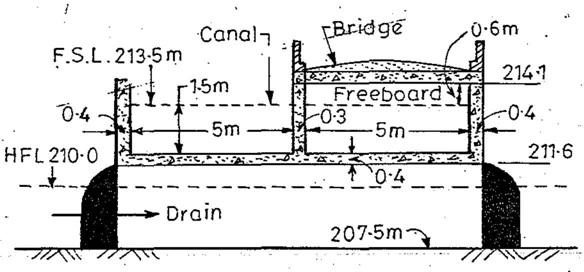


Fig. 14.23

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 the trough. The height of the trough will, therefore, be kept equal to 2.1 m. The entire trough section will be constructed in monolithic reinforced concrete and can be designed by usual structural methods. The tentative thicknesses may be used as follows :

Outer walls = 0.4 m thick

Bottom slab of trough = 0.4 m thick

The intermediate partition wall is to be extended in the transitions so as to provide the necessary clear width of 10 m. The detailed drawing of the aqueduct is illustrated in attached chart Fig. 14.24.

Example 14.2. *Design a syphon aqueduct if the following data at the crossing of a canal and a drainage are given :*

(i) *Discharge of canal*	= 40 cumecs.
(ii) *Bed width of canal*	= 30 m.
(iii) *Full supply depth of canal*	= 1.6 m.
(iv) *Bed level of canal*	= 206.4 m.
(v) *Side slopes of canal*	= $1\frac{1}{2}$ H : 1 V.
(vi) *High flood discharge of drainage*	= 450 cumecs
(vii) *High flood level of drainage*	= 207.0 m.
(viii) *Bed level of drainage*	= 204.5 m.
(ix) *General ground level*	= 206.5 m.

Solution. Since the drainage is of a large size, work of type III will be adopted. Further, because the canal bed level (206.4 m) is slightly below the drainage HFL (207.0 m) ; **a syphon aqueduct** is required and is also asked for. The earthen banks of the canal will be discontinued and the canal water taken in a concrete trough. For affecting economy, the canal shall be flumed.

Step 1. Design of Drainage Waterway

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

$$= 4.75\sqrt{450} = 100.8 \text{ m}$$

Provide 11 clear spans of 8 m each and let the width of each pier be 1.5 m.

The length occupied by 11 bays of 8 m each $= 11 \times 8 = 88$ m

The length occupied by 10 piers of 1.5 each $= 10 \times 1.5 = 15$ m

Total length of waterway $= 88 + 15 =$ **103 m.**

Let us, now, limit the velocity through syphon-barrels, to a value, say 2 m/sec.

Height of barrels required

$$= \frac{\text{Discharge}}{\text{Velocity} \times \text{clear width of waterway}} = \frac{450}{2 \times 88} \text{ m} = 2.56 \text{ m}.$$

Hence, provide 11 rectangular barrels, each 8 m wide and 2.5 m high.

Actual velocity through barrels $= \dfrac{450}{11 \times 8 \times 2.5} = 2.05$ m/sec.

Step 2. Design of Canal Waterway

Normal bed width of canal = 30 m

Let the width be reduced to 15 m.

Providing a splay of 2 : 1 in contraction, the length of contraction transition

$$= \frac{30-15}{2} \times 2 = 15 \text{ m}.$$

Providing a splay of 3 : 1 in expansion, the length of expansion transition

$$= \frac{30-15}{2} \times 3 = 22.5 \text{ m}.$$

Length of flumed rectangular portion of the canal between abutments = 103 m (provided). In transitions, the side slopes of the canal section shall be warped in plan from the original slope of $1\frac{1}{2} H : 1 V$ to vertical.

Step 3. Design of Bed Levels at Different Sections (Fig. 14.25)

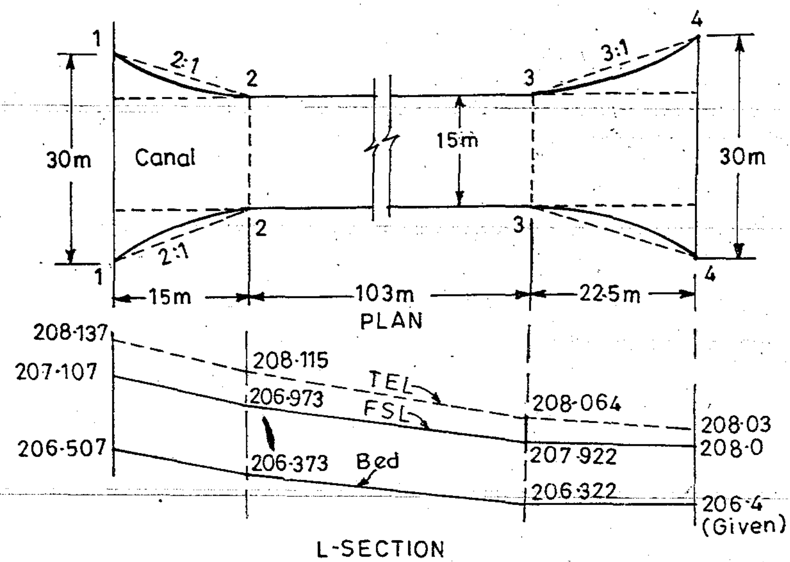


Fig. 14.25. Plan and Section of Canal Trough in Example 14.25.

At Section 4-4

When the canal returns to its normal section, we have the known conditions as follows :

Area of trapezoidal canal section

$$= (B + 1.5y)\,y$$

where B = Bed width = 30 m

y = Depth = 1.6 m

$$= [30 + 1.5 \times 1.6]\ 1.6$$

$$= 32.4 \times 1.6 = 51.84 \text{ sq. m.}$$

$$\text{Velocity of flow} = V_4 = \frac{Q}{A} = \frac{40}{51.84} = 0.77 \text{ m/sec.}$$

$$\text{Velocity head} \quad = \frac{V_4^2}{2g} = \frac{(0.77)^2}{2 \times 9.81} = 0.030 \text{ m}$$

R.L. of canal bed at 4-4 = **206.4 m** (given)

Water depth = 1.6 m (given)

R.L. of water surface at 4-4 = 206.4 + 1.6 = **208.0 m**

R.L. of T.E.L. at 4-4 = 208.0 + 0.03 = 208.03 m.

At Section 3-3

Assuming a constant depth of 1.6 m throughout the channel, we have at section 3-3, a rectangular channel, as follows :

Bed width = 15 m

Depth = 1.6 m (assumedconstant)

Area = 15 × 1.6 = 24 sq. m.

$$\text{Velocity} \quad = V_3 = \frac{40}{24} = 1.67 \text{ m/sec.}$$

$$\text{Velocity head} = \frac{V_3^2}{2g} = \frac{(1.67)^2}{2 \times 9.81} = 0.142 \text{ m}$$

Assuming that the loss of head in expansion from section 3-3 to section 4-4 is taken as

$$= 0.3 \left[\frac{V_3^2 - V_4^2}{2g}\right]$$

$$= 0.3\ [0.142 - 0.030] = 0.3 \times 0.112 = 0.0336 \text{ m; say } \mathbf{0.034\ m.}$$

R.L. of T.E.L. at 3-3 = R.L. of T.E.L. at 4-4 + Loss in expansion

$$= 208.030 + 0.034 = 208.064 \text{ m.}$$

R.L. of water surface at 3-3

$$= 208.064 - 0.142 = \mathbf{207.922\ m.}$$

R.L. of bed at 3-3 $= 207.922 - 1.6 = \mathbf{206.322\ m.}$

At Section 2-2

From section 2-2 to section 3-3, the trough section is constant. Therefore, the area and velocity at 2-2 are the same as at 3-3. There is a friction loss between 2-2 and 3-3, which may be computed by Manning's formula, as equal to

$$H_L = \frac{n^2 V^2 L}{R^{4/3}}$$

where n is rugosity coefficient, whose value in a concrete trough may be taken as 0.016 and L is the length of channel = 103 m.

Area of trough section $(A) = 15 \times 1.6 = 24$ sq. m

Wetted perimeter $= 15 + 2 \times 1.6 = 18.2$ m

Hydraulic mean depth $= R = \frac{A}{P} = \frac{24}{18.2} = 1.32$ m

Velocity in trough $= \frac{Q}{A} = \frac{40}{24} = 1.67$ m/sec

$$\text{Head loss, } H_L = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}} = \frac{(0.016)^2 \times (1.67)^2 \times 103}{(1.32)^{4/3}} = \mathbf{0.051\ m.}$$

R.L. of T.E.L. at 2-2 = R.L. of T.E.L. at 3-3 + Head loss in trough
$= 208.064 + 0.051 = 208.115$ m.

R.L. of water level at 2-2 $= 208.115 - 0.142 = \mathbf{207.973\ m.}$

R.L. of bed at 2-2 $= 207.973 - 1.6 = \mathbf{206.373\ m.}$

At Section 1.1

Loss of head in contraction transition from section 1-1 to section 2-2 may be taken as

$$= 0.2\left[\frac{V_2^2 - V_1^2}{2g}\right] = 0.2\left[\frac{(1.67)^2 - (0.77)^2}{2 \times 9.81}\right] = 0.0224\text{ m ; say } \mathbf{0.022\ m.}$$

R.L. of T.E.L. at 1-1 = R.L. of T.E.L. at 2-2 + Loss in contraction
$= 208.115 + 0.022 = 208.137$ m

R.L. of water surface at 1-1
$= 208.137 - 0.030 = \mathbf{208.107\ m.}$

R.L. of bed at 1-1 required to maintain constant depth
$= 208.107 - 1.6 = \mathbf{206.507\ m.}$

All these levels are plotted and shown in Fig. 14.25.

Step 4. Design of Transitions

(*a*) *Contraction Transition.* Since depth is kept constant, the transition shall be designed on the basis of Mitra's Hyperbolic transition equation, (14.2) given by

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

where $B_f = 15$ m
$B_n = 30$ m
$L_f = 15$ m.

Substituting, we get

$$B_x = \frac{30 \times 15 \times 15}{30 \times 15 - x\,(30 - 15)} = \frac{6750}{450 - 15x} = \frac{450}{30 - x}$$

For various values of x lying between 0 to 15 m, various values of B_x are worked out, as shown in Table 14.4. The distance x is is measured from the flumed section 2-2.

Table 14.4

x in metres	0	2	4	6	8	10	12	14	15
$B_x = \frac{450}{30 - x}$ in metres	15.0	16.04	17.27	18.72	20.42	22.5	25.0	28.1	30.0

Expansion Transition. In this case, we have

$$B_n = 30 \text{ m}$$
$$B_f = 15 \text{ m}$$
$$L_f = 22.5 \text{ m}$$

$$B_x = \frac{B_n \cdot B_n \cdot L_f}{B_n \cdot L_f - x(B_n - B_f)} \quad \textit{i.e. } \text{Eq. (14.2)}$$

$$= \frac{30 \times 15 \times 22.5}{30 \times 22.5 - x(30 - 15)} = \frac{675}{45 - x}$$

For various values of x lying between 0 to 22.5 m, corresponding values of B_x are worked out, as shown in Table 14.5. The distance x is measured from the flumed section B_f.

Table 14.5

x in metres	0	2	4	6	8	10	12	14	16	18	20	22.5
$B_x = \frac{675}{45 - x}$ in metres	15.0	15.7	16.46	17.3	18.25	19.3	20.4	21.75	23.3	25.0	27.0	30.0

Step 5. Design of Trough

The trough shall be divided into three equal compartments, each 5 m wide, separated by 0.3 m thick partition walls (2 Nos.). The inspection road (5 m wide) shall be carried in the extreme left compartment, as shown in Fig. 14.28. A free-board of 0.6 m above the normal water depth of 1.6 m is sufficient, and hence, the bottom level of bridge slab may be kept at $1.6 + 0.6 = 2.2$ m above the bed level of the trough. The height of the trough will also be kept as equal to 2.2 m. The entire trough section can be designed as monolithic reinforced concrete structure by the usual structural methods. The tentative thicknesses may be used as follows :

Outer walls $= 0.4$ m thick

Bottom slab of trough $= 0.4$ m thick.

The intermediate walls shall be extended into transitions, so as to provide the necessary clear width of 15 m.

Now, the overall outer width of trough (including walls)

$$= 15 + 2 \times 0.3 + 2 \times 0.4$$
$$= 15 + 0.6 + 0.8 = \mathbf{16.4\ m.}$$

Hence, the length of syphon barrel = **16.4 m**

Step 6. Head Loss Through the Syphon Barrels

The head loss through the syphon barrels is given by Unwin's formula as equal to (neglecting vel. of approach)

$$h = \left[1 + f_1 + f_2 \cdot \frac{L}{R}\right]\frac{V^2}{2g} \qquad \textit{i.e. } \text{Eq. (14.1)}$$

where V = velocity through barrels = 2.05 m/sec

f_1 = coefficient of loss of head at entry,

= 0.505 for unshaped mouth.

$f_2 = a\left(1 + \frac{b}{R}\right)$ where the values of a and b a

taken from table 14.1 for ceme

plastered barrels as

a = 0.00316

b = 0.030

R = Hydraulic mean depth for barrel.

$$= \frac{A}{P} = \frac{8 \times 2.5}{2\,(8 + 2.5)} = \frac{20}{21} = 0.953$$

L = Length of barrel = 16.4 m.

Substituting these values, we get

$$f_2 = 0.00316\left[1 + \frac{0.030}{0.953}\right] = \mathbf{0.00326}$$

$$\therefore \qquad h = \left[1 + 0.505 + 0.00326\left(\frac{16.4}{0.953}\right)\right]\frac{(2.05)^2}{2 \times 9.81} = \mathbf{0.333\ m}$$

High flood level of Drainage is given = 207.0 m

$\therefore$ d/s H.F.L. = 207.0 m

Afflux (h) = 0.333 m

u/s H.F.L. = d/s H.F.L. + Afflux (or loss of head)

= 207.0 + 0.333 = **207.333 m.**

Step 7. Uplift Pressure on Roof of barrels

R.L. of bottom of trough = R.L. of canal bed – Slab thickness

= 206.4 – 0.4 = 206.0 m

$$\text{Loss of head at entry of barrel} = 0.505\,\frac{V^2}{2g} = 0.505 \times \frac{(2.05)^2}{2 \times 9.81} = 0.108\ \text{m.}$$

Uplift on the roof

= u/s H.F.L. – Loss at entry – Level of underside of roof slab

= 207.333 – 0.108 – 206.0

= 1.225 m of water = 12.25 kN/m^2 (1.225 t/m^2)

(Assuming unit wt. of water = 10 kN/m^3 or 1 t/

The concrete trough slab is 0.4 m thick and will thus exert a downward load o

$0.4 \times 24 = 0.96$ kN/m^2 (assuming unit wt. of concrete = 24 kN/

∵ Strictly speaking, unit wt. of water = 9.81 kN/m^3 ; but to ease in calculations we have taken unit water = 10 kN/m^3 (1 t/m^3) and of concrete = 24 kN/m^3.

The balance of the uplift pressure *i.e.* $12.25 - 9.6 = 2.65\ \text{kN/m}^2$ has to be resisted by the reinforcement to be provided at the top in the roof slab. The roof slab has to be designed for full canal water load (1.6 m of water) plus self weight, when the drainage water is low and not exerting any uplift. Suitable reinforcement at bottom of the slab may be provided for this downward force, as worked out below :

Step 8. Design of roof of barrel

Uplift to be balanced by top reinforcement = $2.65\ \text{kN/m}^2$

Downward water load acting when there is no uplift = 1.6 m of water = $16\ \text{kN/m}^2$

Load due to self wt. of slab $= 0.4 \times 24 = 9.6\ \text{kN/m}^2$

Total downward load $= 16 + 9.6 = 25.6\ \text{kN/m}^2$

An intermediate wall of 0.3 m thickness has been provided in the trough, having clear span of 5 m, the effective span is, therefore, $= 5 + 0.3 = 5.3$ m.

Let us consider 1 m wide strip of slab.

Maximum sagging bending moment in slab due to downward loads

$$= \frac{25.6 \times (5.3)^2}{10}\ \text{kN–m} = 71.7\ \text{kN–m} = 71.7 \times 10^5\ \text{N.cm}$$

Maximum Hogging moment due to residual uplift acting from below

$$= \frac{2.65 \times (5.3)^3}{10}\ \text{kN–m} = 7.42\ \text{kN–m} = 7.42 \times 10^5\ \text{N.cm}$$

$$\text{Max. shear force} = \frac{wl}{2} = \frac{25.6 \times 5.0}{2}\ \text{kN} = 64\ \text{kN}$$

Using 1 : 2 : 4 cement concrete, we have

Effective depth (d) of slab required

$$d = \sqrt{\frac{M}{Qb}} = \sqrt{\frac{71.7 \times 10^5}{87 \times 100}} = \mathbf{28.8\ cm}$$

Provided overall thickness is 40 cm and thus provided effective depth

$$d = 37.5\ \text{cm}.$$

Steel required at the bottom of the slab

$$= \frac{71.7 \times 10^5}{12000 \times 0.87 \times 37.5}\ \text{cm}^2/\text{m length}$$

(using reduced stress in steel as $12000\ \text{N/cm}^2$)

$$= 18.3\ \text{cm}^2$$

Provide 16 mm ϕ bars @ 10 cm c/c in the bottom of the slab.

Steel required at top

$$= \frac{7.42 \times 10^5}{12000 \times 0.87 \times 37.5} = 1.9\ \text{cm}^2\ \text{(Nominal)}$$

Provide 10 mm ϕ bars @ 15 cm/cc.

Also provide 10 mm ϕ bars @ 20 cm c/c as distribution reinforcement both at top as well as at bottom, as shown in Fig. 14.26.

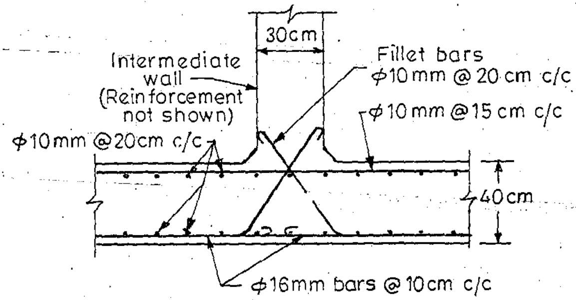


Fig. 14.26. Reinforcement in roof slab.

Step 9. Uplift on the bottom floor of barrel

(*a*) *Static Head*

R.L. of barrel floor = R.L. of trough bottom – Height of barrel

$$= 206.0 - 2.5 = 203.5 \text{ m}.$$

Let us assume that a thickness of 0.8 m is provided.

$\therefore$ R.L. of bottom of floor

$$= 203.5 - 0.8 = 202.7 \text{ m}$$

Bed level of drain = 204.5 m.

Assuming that the water-table has gone upto bed level of drain, the static uplift on the floor (refer Fig. 14.18)

$$= 204.5 - 202.7 = 1.8 \text{ m of water}$$

(*b*) *Seepage Head.* The seepage head will be maximum when the canal is running full and the drain is dry.

Thus, the total seepage head = F.S.L. of canal – bed level of drain

$$= 208.0 - 204.5 = 3.5 \text{ m}.$$

The residual seepage head at a point '*a*' in the centre of the first barrel (Fig. 14.27) has been calculated by Bligh's theory, as follows :

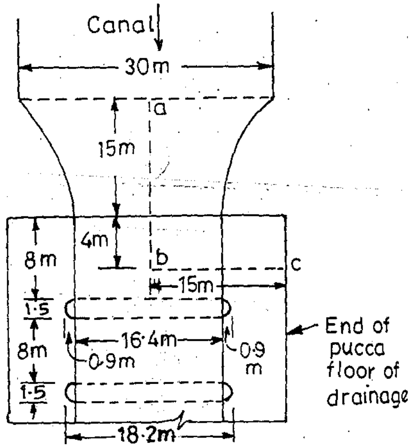


Fig. 14.27

Assuming that the total length of drainage floor = 30 m.

The seepage line *abc* will traverse creep lengths as follows :

ab = Length of u/s transition + Half the barrel span = 15 + 4 = 19 m.

bc = 15 m (Half of the total length of 30 m = assumed)

Total creep length = 19 + 15 = 34 m.

$$\text{Residual seepage head at } b = 3.5\left[1 - \frac{19}{34}\right] = 3.5 \times \frac{15}{34} = 1.55 \text{ m}.$$

Total uplift = Static head + Seepage head

$$= 1.8 + 1.55 = 3.35 \text{ m of water} = 33.5 \text{ kN/m}^2$$

The provided 0.8 m thickness of slab will resist due to its own wt., an uplift

$$= 0.8 \times 24 = 19.2 \text{ kN/m}^2.$$

$\therefore$ Balance to be resisted by reinforcement due to bending action

$$= 33.5 - 19.2 = 14.3 \text{ kN/m}^2.$$

Suitable reinforcement for this uplift (*i.e.* 14.3 kN/m^2) has to be provided at the top of the culvert floor so as to counteract the bending action.

Note. The length of the floor has been provided equal to 32 m, as shown in Fig. 14.28, on the following considerations.

(*i*) Length of floor required under barrel	= 16.4 m
(*ii*) Extra floor length required to accommodate pier noses on both sides = 2 × 1.0	= 2.0 m
(*iii*) Horizontal length of d/s ramp joining to bed level at a slope of 5 : 1 = 5 (204.5 – 203.5)	= 5.0 m
(*iv*) Width of d/s cut-off beyond ramp	= 0.6 m
(*v*) Length of extra floor provided on u/s side	= 0.6 m
	Total length = 30.0 m

Step 10. Design of cutoffs and protection works for the drainage floor

The depth of scour $(R) = 0.47\left[\frac{Q}{f}\right]^{1/3}$; assuming $f = 1.0$

$$\therefore \quad R = 0.47\left(\frac{450}{1}\right)^{1/3} = 0.47 \times 7.65 = 3.59 \text{ m}$$

Provide depth of cut-offs for scour hole of 1.5 R on both sides

Depth of u/s cut off below H.F.L.

$$= 1.5R = 1.5 \times 3.59 = 5.4 \text{ m}$$

R.L. of bottom of u/s cut-off = u/s H.F.L. – 5.4 m

$$= 207.333 - 5.4 = 201.933 \text{ m} \text{ ; say } \mathbf{201.93 \text{ m}}.$$

R.L. of bottom of d/s cut-off = d/s H.F.L. – 5.4

$$= 207.0 - 5.4 = \mathbf{201.6 \text{ m}}$$

Length of u/s protection (*i.e.* 40 cm thick brick pitching)

$$= 2 \text{ [R.L. of u/s bed – R.L. of bottom of u/s cut-off]}$$

$$= 2\,[203.50 - 201.93]$$

$$= 2 \times 1.57 = 3.14 \text{ m} \text{ ; say } \mathbf{3.2 \text{ m}}$$

Similarly, length of d/s brick pitching

$$= 2 \text{ [R.L. of d/s bed – R.L. of bottom of d/s cutoff]}$$

$$= 2\,[204.5 - 201.6] = 2 \times 2.9 = \mathbf{5.8 \text{ m}}$$

The pitchings may be supported by 0.4 m wide and 1 m deep toe walls, as shown in Fig. 14.28.

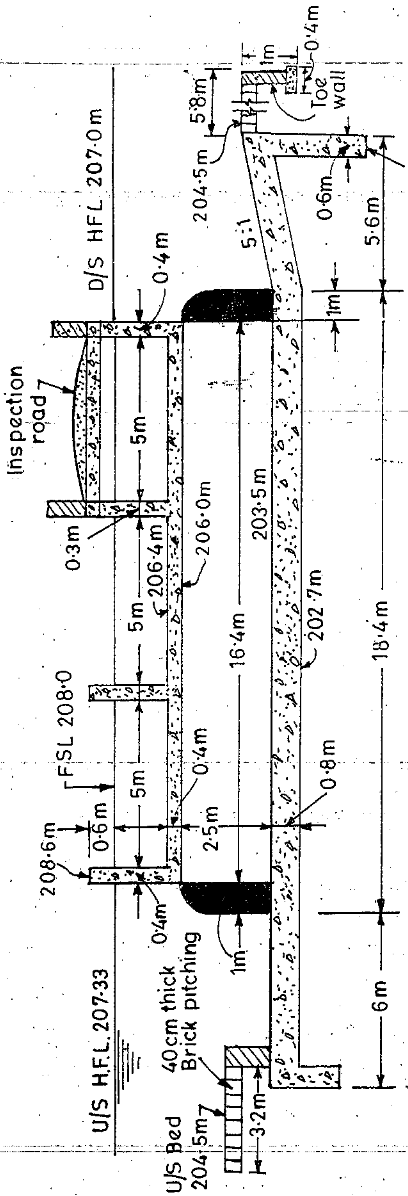


Fig. 14.28. Section of Syphon Aqueduct in Example 14.2.

Example 14.3. *Design a suitable cross drainage work, given the following data at the site of the crossing of two streams of water :*

(a) Irrigation channel :

Full supply discharge = *350 cumecs*

Full supply level = *202.5 m.*
Canal bed level = *197.8 m.*
Canal bed width = *35 m.*
Full supply depth = *4.7 m.*
Side slopes = $\frac{1}{2}H : 1V$

(b) Natural Drainage :
Drainage bed level = *203.9 m.*
High flood level = *205.2 m.*
Catchment area of drainage up to crossing = *14.3 sq. km.*

The Dicken's formula may be used for computing H.F.Q. with its coefficient as 18 (SI or MKS units).

Solution. The high flood discharge of the drainage at the point of the crossing may be obtained by using Dicken's formula, which states

$$Q = C \cdot A^{3/4} = 18 \cdot (14.3)^{3/4} = 18 \times 7.33 = \mathbf{132\,cumecs}.$$

Since the bed level of drainage (203.9 m) is much above the canal *FSL* (202.5 m), *i.e.* by about 1.4 m ; the canal water will be taken below the drainage. Hence, the cross-drainage 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 that of an aqueduct, and is given below :

Step 1. Design of Canal Water-way

The flow velocity in the canal (as it is)

$$\frac{\text{Discharge}}{\text{Area}} = \frac{Q}{(B + \frac{1}{2}y)\, y} \text{ (using } \tfrac{1}{2} H : 1V)$$

$$= \frac{350}{(35 + 0.5 \times 4.7)\, 4.7} = \frac{350}{37.35 \times 4.7} = 1.99 \text{ m/sec.}$$

This high velocity shows that the canal is already a lined canal, and much more fluming can not 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 (on u/s side), the length of contraction transition

$$= \frac{35 - 31.5}{2} \times 2 = 5 \text{ m}$$

Providing a splay of 3 : 1 in d/s expansion, the length of expansion transition

$$= \frac{35 - 31.5}{2} \times 3 = 7.5 \text{ m}$$

Length of flumed rectangular portion of canal will be equal to the width of the drainage troughs, as worked out in the next step (*i.e.* 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 II. 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 provided 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 R.C.C. compartments, each of clear width equal to 8.0 m, thus giving

Clear waterway $= 8.0 \times 6 = 48$ 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 $= 48 + 1.5 = \mathbf{49.5}$ **m**; against P of 54.5 m (which is not too less as to cause troubles)

The two side walls of the R.C.C. 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 canal will also be equal to 50.5 m.

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

Assuming 2 : 1 convergence, we have the length of contraction transition

$$= \frac{54.5 - 49.5}{2} \times 2 = 5 \text{ m}$$

Assuming 3 : 1 splay in expansion, we have the length of expansion transition

$$= \frac{54.5 - 49.5}{2} \times 3 = 7.5 \text{ m}$$

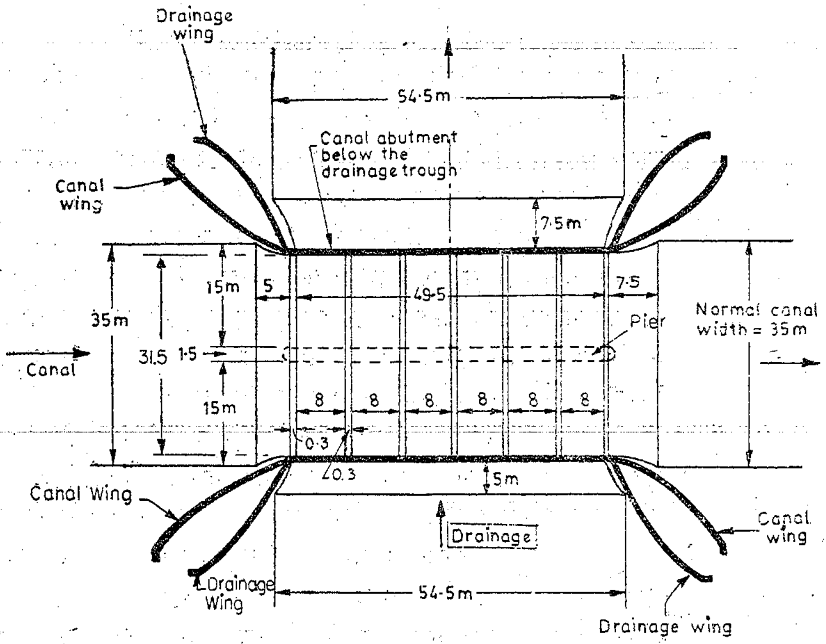


Fig. 14.29. Indicative Plan of the Super-passage Crossing in Example 14.3.

The wing walls will be constructed to reduce the drainage waterway width fron 54.5 m to 49.5 m on upstream, and return walls will be constructed to expand the drainage waterway from 49.5 m to 54.5 m on downstream. 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). A line plan of the crossing is shown in Fig 14.29.

Step 3. Head Loss and Bed Levels at different Sections along the length o drainage trough.

At section 4-4 (Refer Fig. 14.30)

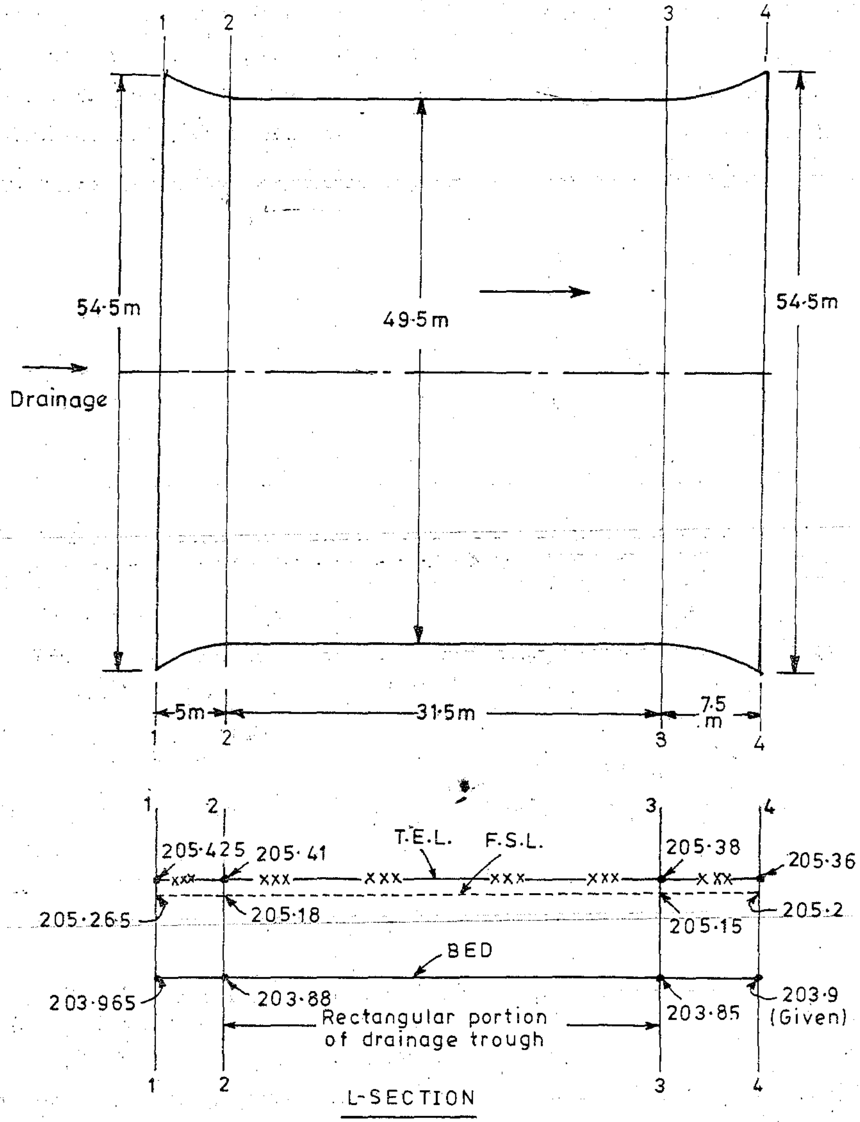


Fig. 14.30. Plan and L-section of Drainage trough carried over the canal.

At section 4-4, where the drainage returns to its normal section, we have the area of the natural drainage section

$$= \text{width } (\approx \text{perimeter}) \times \text{Depth}$$
$$= 54.5 \times (205.2 - 203.9) = 54.5 \times 1.3 = 70.85 \text{ m}^2$$

$$\text{Velocity} = V_4 = \frac{Q}{A} = \frac{132}{70.85} = 1.86 \text{ m/sec.; say } \mathbf{1.9 \text{ m/sec.}}$$

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

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

R.L. of water surface at 4-4

$$= 203.9 + 1.3 \text{ (as the normal depth in drainage} = 1.3 \text{ m)}$$
$$= 205.2 \text{ m}$$

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

At Section 3-3 (Refer Fig. 14.30)

Keeping the same depth of 1.3 m throughout the drainage, we have at section 3-3.

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

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

$$\text{Velocity} = \frac{132}{62.4} = 2.12 \text{ m/sec.}$$

$$\text{Velocity Head} = \frac{(2.12)^2}{2 \times 9.81} = 0.23 \text{ m.}$$

Assuming that the loss of head in expansion from section 3-3 to section 4-4 is taken as

$$= 0.3 \left(\frac{V_3^2 - V_4^2}{2g} \right)$$
$$= 0.3\,(0.23 - 0.16) = 0.3 \times 0.07 = 0.021 \text{ m; say } \mathbf{0.02 \text{ m}}$$

$\therefore$ R.L. of TEL at 3-3

$$= \text{R.L. of TEL at section 4-4 + Loss in expansion}$$
$$= 205.36 + 0.02 = 205.38 \text{ m}$$

$\therefore$ R.L. of water surface at 3-3

$$= \text{R.L. of TEL at 3-3} - \text{Velocity head}$$
$$= 205.38 - 0.23 = \mathbf{205.15 \text{ m}}$$

$\therefore$ R.L. of bed at 3-3 $= 205.15 - 1.3 = \mathbf{203.85 \text{ m}}$

At Section 2-2

From section 2-2 to 3-3, the trough section is constant. Therefore, area and velocity at 2-2 are the same as at 3-3. But from 2-2 to 3-3, there is a friction loss between 2-2 and 3-3, which may be computed by Mannings formula, as :

$$H_L = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}}$$

where n is the rugosity coefficient whose value in concrete trough may be taken as 0.016;

and L is the length of trough = 31.5 m.

The area of trough section $(A) = 48 \times 1.3 = 62.4$ sq m

Wetted perimeter $(P) = 48 + 2 \times 1.3 = 50.6$ m

Hydraulic mean depth $(R) = \frac{A}{P} = \frac{62.4}{50.6} = 1.23$ m

Velocity in trough $= \frac{Q}{A} = \frac{132}{62.4} = 2.12$ m/sec

$$\therefore \quad H_L = \frac{(0.016)^2 \times (2.12)^2 \times 31.5}{(1.23)^{4/3}} = 0.0275 \text{ m} ; \textbf{ say } \mathbf{0.03 \text{ m}}$$

R.L. of TEL at 2-2 = R.L. of TEL at 3-3 + Friction loss (H_L) in trough

$= 205.38 + 0.03 = 205.41$ m

R.L. of water surface at 2-2

$= 205.41 - 0.23 = 205.18$ m

$\therefore$ R.L. of bed at 2-2

$= 205.18 \text{ m} - 1.3 = \mathbf{203.88 \text{ m}}$

At Section 1-1

Loss of head in contraction transition from 1-1 to 2-2

$$= 0.2 \left(\frac{V_2^2 - V_1^2}{2g} \right) = 0.2\,[0.23 - 0.16] = \mathbf{0.014 \text{ m}; \text{ say } 0.015 \text{ m}}$$

R.L. of TEL at 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 $= 205.425 - 0.16 = 205.265$ m

R.L. of bed at 1-1 $= 205.265 - 1.3 = \mathbf{203.965 \text{ m}}$

All these bed levels, FSL and TEL are plotted in Fig. 14.30.

Step 4. Design of Transitions for the Drainage

(*a*) *Contraction Transition.* Since the depth is kept constant, the transitions can be designed on the basis of Mitra's hyperbolic transition equation given by *eqn.* (14.2) as:

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

where $B_f = 49.5$ m

$B_n = 54.5$ m

$L_f = 5$ m.

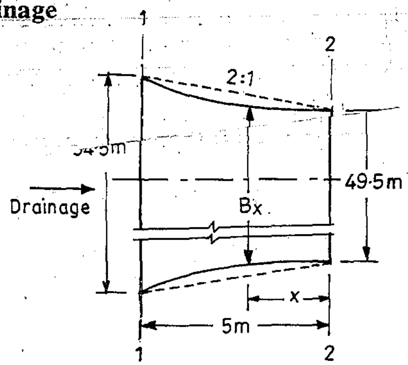


Fig. 14.31

Substituting, we get

B_x = width at any distance x from the flumed section 2–2

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

$$= \frac{2700}{54.5 - x}$$

For various values of x lying between 0 to 5 m, values of B are worked out as shown in Table 14.6.

Table 14.6

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

The contraction transition can be plotted with these values.

Expansion Transition. In this case $B_x = 54.5$ m, $B_f = 49.5$ m, $L_f = 7.5$ m.

Using Eq. (14.2), we get

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

$$= \frac{154.5 \times 49.5 \times 7.5}{7.5 \times 54.5 - x\,(54.5 - 49.5)}$$

$$= \frac{4050}{81.7 - x}$$

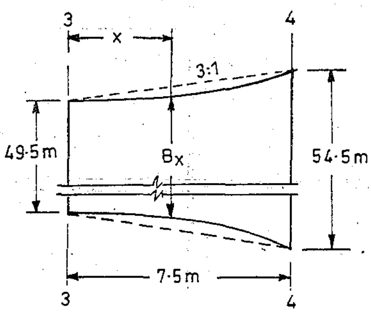


Fig. 14.32

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 shown in Table 14.7.

Table 14.7

x in metres	0	2	4	6	7.5
$B_x = \dfrac{4050}{81.7 - x}$ in metres	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, as pointed out earlier, has been divided into six compartments of 8 m clear width each, and separated by intermediate walls (5 No.) each 0.3 m thick (tentative thickness). The end walls of the trough have tentatively been kept as 0.4 m wide each, as shown in Fig. 14.33. An inspection road may be carried on the top of the end compartment as shown.

A free-board 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 may be kept $1.3 + 0.6 = 1.9$ m above the bed level of trough (Different bed levels along the length of trough are shown in Fig. 14.30.). The height of the trough walls will also be kept equal to 1.9 m. The entire trough section will be constructed in monolithic reinforced concrete, and can be designed by usual structural methods. The tentative thickness for the walls have already been assumed and should be verified. A detailed drawing for the crossing can be prepared on the same lines on which the drawing 14.24 was prepared.

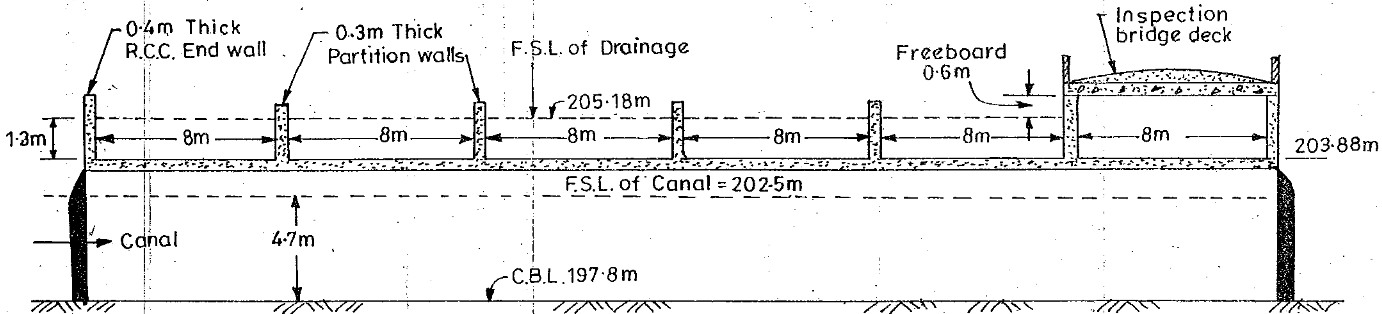


Fig. 14.33. Section of Drainage Trough passing over the Canal
(Levels of Trough bed and FSL at the start of rectangular portion of Trough are shown).

Example 14.4. *Design a suitable cross drainage work, given the following data :*

(*a*) *Irrigation channel*

F.S.Q.	= *354 cumecs.*
Bed width	= *24 m.*
FSL	= *207.60 m.*
C.B.L.	= *201.4 m.*
Side slopes	= $\frac{1}{2}H : IV.$

(*b*) *Natural drainage*

H.F.Q.	= *600 cumecs.*
Drainage bed level	= *203.6 m.*
H.F.L.	= *206.3 m.*

Solution. Since the bed of the natural drainage (203.6 m) is higher than the canal bed (201.4 m) by a sufficient margin of 2.2 m, we may think of taking the canal below the drainage. But since the FSL of canal (207.6 m), is much above the drainage bed (203.6 m), the simple super passage cannot be provided. The canal water will, therefore, have to be syphoned below the drainage. Since the discharge of the canal is low compared to that of the drainage, such a crossing will not prove uneconomical also. Hence, a **canal syphon** will be designed as below :

Step I. Design of Canal Waterway

Let the velocity through the canal barrels be limited to 3 m/sec.

$\therefore$ Area of waterway of barrels required $= \dfrac{354}{3} = 118 \text{ m}^2$

Let us provide 4 spans each having a clear waterway of 5.4 m, thus giving a clear waterway of $4 \times 5.4 = 21.6$ m. Let R.C.C. piers (three No.) each of width 0.6 m are provided ; thus occupying a width $3 \times 0.6 = 1.8$ m.

Total overall waterway provided between abutments

$= 21.6 + 1.8$

$= 23.4$ m; which is O.K., as it is less than 24 m (*i.e.* the width of canal)

Assuming 0.3 m as the width of each bearing on abutments, the total length of drainage trough $= 23.4 + 0.6 =$ **24 m.**

Thus, the canal on the u/s and d/s sides is joined to the barrels aggregate width of 23.4 m by suitable transitions and approaches.

$$\text{The height of the barrel} = \frac{\text{Discharge}}{\text{Velocity} \times \text{Waterway width}}$$

$$= \frac{354}{3 \times 21.6} = 5.47 \text{ m.}$$

So let us keep 5.5 m height of the barrels, thus maintaining an actual velocity through them $= \dfrac{354}{5.5 \times 21.6} = 2.98$ m/sec.

Step 2. Design of Drainage Waterway

Lacey's regime perimeter $= P = 4.75\sqrt{Q} = 4.75 \times \sqrt{600} = 116$ m.

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Let us assume that the rectangular R.C.C. drainage trough is provided in such width that its perimeter becomes equal to that required by Lacey's equation, thus givin width of the trough = 116 − 2 (2.7) = 110.6 m ; say 111 m.

Let us divide the drainage trough into 9 (compartments each of 12 m width (clea width) with 0.4 m thick partition walls (8 No.). Thus the overall width of the troug provided will be equal to $9 \times 12 + 8 \times 0.4 = 108.0 + 3.2 = 111.2$ m. Two end walls eac of 0.4 m thickness can be provided, thus giving the total width of drainage trough en to end = 112 m ; and this will be equal to the length of the canal barrels.

Step 3. Design of Bed Levels along the Drainage Trough

Since the drainage waterway has not been flumed, the only loss of head in TE when the water flows in the trough is due to frictional loss. Assuming that the drainag levels (given) are on the d/s of the crossing.

The bed level of drainage trough (d/s) = 203.6 m.

$$\text{Velocity in trough, } V = \frac{600}{7\,(15.5 \times 2.7)} = 2.05 \text{ m/sec.}$$

$$\text{Head loss due to friction } (H_L) = \frac{n^2 \cdot V^2 L}{R^{4/3}}$$

$$\text{where } R = \frac{A}{P} = \frac{7\,(15.5 \times 2.7)}{7\,(15.5 + 5.4)} = 2.0$$

$$\therefore \quad H_L = \frac{(0.016)^2 \times (2.05)^2 \times 24}{(2)^{4/3}} = 0.0102 \text{ m.}$$

Hence, the trough should be given a slope (S_0) [as to pass the full drainage d charge] given by

0.0102 m loss in 24 m length

$$1 \text{ m loss will occur in a length} = \frac{24}{0.0102} \text{ m} = 2340 \text{ m.}$$

Hence, the slope required to be given to trough is 1 in 2340 ; and bed level at t d/s of crossing = 203.6 m; and bed level at u/s of crossing = 203.61 m.

Step 4. Design of Transitions

As no fluming either of canal or of drainage has been done, the question designing transitions does not arise. The wing walls and return walls shall, however, being provided, as shown in the attached chart Fig. 14.36, after properly ensuring their str tural safety.

Step 5. Design of Trough

The trough shall be made of reinforced cement concrete and shall be designed by the usual structural methods. A bridge for inspection can be provided on one e end compartment, as shown in Fig. 14.36. The trough is divided into 9 compartments each of 12 m clear width as already decided. The tentative thicknesses of partition walls (0.4 m) and end walls (0.4 m) have also been made.

Step 6. Head Loss Trough Syphon Barrels

The head loss trough syphon barrels is given by Eq. (14.1) as

$$h = \left(1 + f_1 + f_2 \cdot \frac{L}{R}\right) \frac{V^2}{2g} \text{ (neglecting velocity of approach)}$$

where V is the velocity through the syphon barrels

$= 2.98$ m/sec.

f_1 = coeff. of loss of head at entry

$= 0.505$ for unshaped mouth

$f_2 = a\left[1 + \frac{b}{R}\right]$ where the values of a and b are taken from Table 14.1 for cement plastered barrels as

$a = 0.00316$

$b = 0.030$

R = Hydraulic mean depth for barrel

$$= \frac{A}{P} = \frac{5.4 \times 5.5}{2\,(5.4 + 5.5)} = \frac{5.4 \times 5.5}{21.8} = 1.36 \text{ m}$$

L = Length of barrels = 112 m.

Substituting these values, we get

$$f_2 = 0.00316\left[1 + \frac{0.030}{1.36}\right] = 0.00323$$

$$\therefore \quad h = \left[1 + 0.505 + 0.00323\left(\frac{112}{1.36}\right)\right]\frac{(2.98)^2}{2 \times 9.81} = 0.855; \text{ say } 0.86 \text{ m.}$$

FSL of canal (given) $= 207.6$ m

$\therefore$ d/s FSL $= 207.6$ m

Afflux (h) $= 0.86$ m.

$\therefore$ u/s FSL = d/s FSL + Afflux $= 207.6 + 0.86 =$ **208.46 m**

Step 7. Uplift Pressure on Roof of Barrels

R.L. of bottom of trough

= R.L. of drainage bed – Slab thickness

$= 203.6 - 0.8$ (assuming 0.8 m thick roof slab)

$= 202.8$ m

$$\text{Loss of head at entry of barrel} = 0.505\,\frac{V^2}{2g} = \frac{0.505 \times (2.98)^2}{2 \times 9.81} = 0.23 \text{ m}$$

Uplift on the roof = u/s HFL – Loss at entry – Level of under-side of roof slab

$= 208.46 - 0.23 - 202.8$

$= 5.43$ m of water head $= 54.3$ kN/m^2.

The concrete trough slab is 0.8 m thick and will thus exert a downward load of $.8 \times 24 = 19.2$ kN/m^2.

The balance of the uplift pressure, *i.e.* $54.3 - 19.2 = 35.1$ kN/m^2 has to be resisted reinforcement to be provided at the top in the roof slab. The roof slab has to be igned for full drainage water load (2.7 m of water) plus self-weight, when the canal er is low and not exerting any uplift. Suitable reinforcement, may be provided for downward force, as worked out below :

Step 8. Design of roof barrel

Uplift to be balanced by top reinforcement $= 35.1 \text{ kN/m}^2$

Downward water load acting when there is no uplift $= 2.7$ m of water $= 27 \text{ kN/m}^2$

Load due to self-weight of slab $= 0.8 \times 24 = 19.2 \text{ kN/m}^2$

Total downward load (when there is no uplift) $= 27 + 19.2 = 46.2 \text{ kN/m}^2$

Walls of 0.4 m thickness have been provided in the trough having clear span of 12 m ; the effective span between any two walls is, therefore $= 12 + 0.4 = 12.4$ m

Let us consider 1 m wide strip of slab.

Maximum sagging bending moment in slab due to downward loads (when there is no uplift)

$$= \frac{46.2\,(12.4)^2}{10} \text{ kN.m.} \left(i.e.\ \frac{wl^2}{10}\right) = 708 \text{ kN-m} = 708 \times 10^5 \text{ N.cm}$$

Maximum hogging bending moment due to residual uplift acting from below

$$= \frac{35.1\,(12.4)^2}{10} \text{ kN} \cdot \text{m} = 538 \text{ kN} \cdot \text{m} = 538 \times 10^5 \text{ N} \cdot \text{cm}$$

$$\text{Max. shear force} = \frac{wl}{2} = \frac{46.2 \times 12.4}{2} \text{ kN} = 287 \text{ kN}$$

Using 1 : 2 : 4 cement concrete, we have

Economical Effective depth (d) of slab required

$$d = \sqrt{\frac{M}{Qb}} = \sqrt{\frac{708 \times 10^5}{87 \times 100}} \text{ cm} = 90 \text{ cm.}$$

Provided overall thickness is 80 cm and may be kept the same. Thus. effective depth provided, $d = 77.5$ cm.

Steel required at the bottom of the slab

$$= \frac{708 \times 10^5}{12000 \times 0.87 \times 77.5} \text{ cm}^2\text{/m length}$$

(using reduced stress in steel as 12000 kN/cm^2)

$= 87.8 \text{ cm}^2$/m length of slab.

Provide 32 mm dia bars @ 9 cm c/c in the bottom of the slab.

Steel required at top

$$= \frac{538 \times 10^5}{1200 \times 0.87 \times 77.5} = 66.7 \text{ cm}^2\text{/m length of slab.}$$

Provide 32 mm dia bars @ 12 cm c/c.

Also provide 0.15% of gross cross-sectional area of concrete as distribution steel.

$\therefore$ Area of Dist. steel required

$$= \frac{0.15 \times 80 \times 100}{100} \text{ cm}^2 = 12 \text{ cm}^2$$

$\therefore$ Spacing of 12 mm ϕ bars required

$$= \frac{\text{Area of one bar} \times 100}{\text{Area required}}$$

$$= \frac{1.13 \times 100}{12} = 9.4 \text{ cm; say } 10 \text{ cm c/c}$$

Hence, use 12 mm ϕ bars @ 10 cm c/c as distribution steel, both at top as well as bottom, as shown in Fig. 14.34.

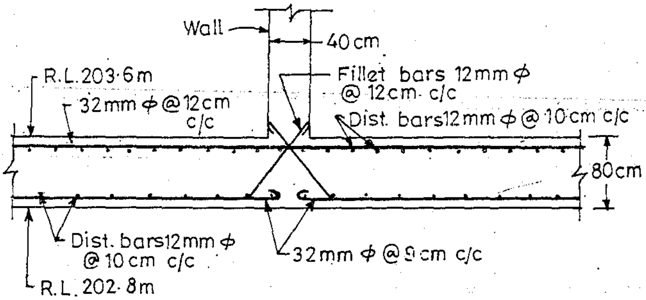


Fig. 14.34. Reinforcement in roof slab.

The partition walls will be designed as beams carrying the bending load from the roof slab.

Step 9. Uplift on the bottom floor of syphon barrels

(*a*) *Static Head*

R.L. of barrel floor = R.L. of trough bottom − Height of barrel

$$= 202.8 - 5.5 = 197.3 \text{ m}$$

Let us assume that a thickness of 0.8 m is provided.

$\therefore$ R.L. of bottom of floor = 197.3 − 0.8 = 196.5 m

Bed level of canal = 201.4 m

Assuming that the water-table has gone upto bed level of the canal, the static uplift on the floor (refer Fig. 14.18)

$$= 201.4 - 196.5 = \mathbf{4.9 \text{ m of water}}$$

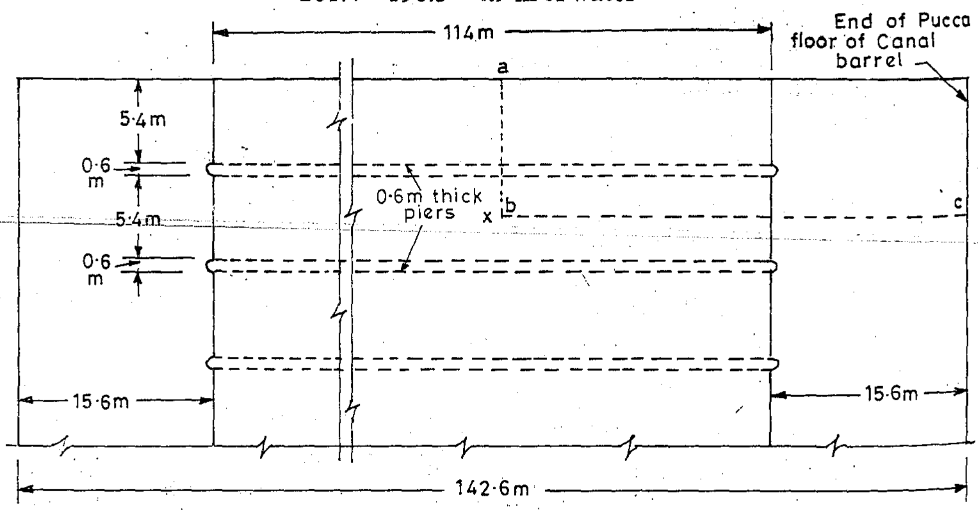


Fig. 14.35

(*b*) *Seepage Head.* The seepage head will be maximum when the drain trough is running full and the canal barrels are dry.

Thus, the total seepage head = H.F.L. of drain − Bed level of canal

$$= 206.3 - 201.4 = 4.9 \text{ m.}$$

The residual seepage head at a point '*a*' in the centre of the first barrel (Fig. 14.35) has been calculated by Bligh's theory as follows :

Assuming that the total length of drainage floor = 142.6 m

The seepage line of drain water *abc* will traverse creep lengths as follows :

$$ab = 5.4 + 0.6 + 2.7 = 8.7 \text{ m} \; ; \; bc = \frac{142.6}{2} = 71.3 \text{ m}$$

Total creep length = 8.7 + 71.3 = 80 m

Residual seepage head at $b = 4.9\left[1 - \frac{8.7}{80}\right] = 4.9 \times \frac{71.3}{80} = 4.37 \text{ m}$

Total uplift = Static head + Seepage head $= 4.9 + 4.37 = 9.27 \text{ m} = 92.7 \text{ kN/m}^2$

The provided 0.8 m thickness will resist due to its own weight, an uplift

$$= 0.8 \times 24 = 19.2 \text{ kN/m}^2$$

∴ Balance to be resisted by reinforcement due to bending action

$$= 92.7 - 19.2 = 73.5 \text{ kN/m}^2$$

Max. hogging B.M. in bottom slab

$$= \frac{73.5 \times (5.4 + 0.6)^2}{10}$$

$$= \frac{73.5 \times 6^2}{10} = 264 \text{ kN : m} = 264 \times 10^5 \text{ N–cm}$$

Steel required $= \frac{264 \times 10^5}{12000 \times 8.7 \times 77.5}$ cm²/m length = 32.7 cm²

Use 32 mm ϕ bars @ 24 cm c/c at top in the barrel floor (*i.e.* bottom floor). 12 mm ϕ bars @ 10 cm c/c both ways at bottom, and one way also at top, may be provided as distribution bars.

The hydraulic dimensions and detailings are shown in attached chart Fig. 14.36.

Step 10. Design of Cutoff's and protection works for the canal floor

Lacey's normal depth of scour $= R = 0.47\left(\frac{Q}{f}\right)^{1/3}$; assuming $f = 1$

$$R = 0.47 \times \left(\frac{354}{1}\right)^{1/3} = 7 \text{ m below FSL}$$

Provide depth of cut off's for scour holes of 1.5 *R* on both sides

or $\quad 1.5R = 1.5 \times 7 = 10.5$ m below FSL

∴ RL of bottom of u/s cutoff = 208.46 − 10.5 = **197.96 m**

R.L. of bottom of d/s cut-off = 207.6 − 10.5 = **197.1 m**

Length of u/s pitching* = 2 [R.L. of u/s bed – R.L. of bottom of u/s cut–off]
= 2 [201.4 – 197.96] = 2 × 3.44 = 6.88 m; **say 6.9 m**

Similarly, length of d/s brick pitching
= 2 [R.L. of d/s bed – R.L. of bottom of d/s cut off]
= 2 [201.4 – 197.1] = 2 × 4.3 = 8.6 m.

The pitchings may be supported by toe walls, 0.4 m wide and 1 m deep, as shown in chart Fig. 14.36.

Example 14.5. *Suggest suitable cross-drainage works at the following crossings :*

(a) Irrigation channel	*Natural Drainage*
Discharge = 350 cumecs	*HFQ = 4300 cumecs*
Bed width = 28 m	*Drainage bed level = 194.9 m*
FSD = 6.2 m	*HFL = 198.5 m*
FSL = 204.3 m	
CBL = 198.1 m	
Natural ground level = 179.3	

(b) Irrigation channel	*Natural Drainage*
Full supply discharge = 350 cumecs	*H.F.Q. = 390 cumecs*
	Drainage bed level = 207.4 m
Bed width = 24 m	*HFL = 209.3 m*
FSD = 6.2 m	*Drainage bed slope = 1 in 500*
FSL = 215.6 m	
CBL = 209.4 m	*Springing level = 207.6 m*
Canal bed slope = 1 in 10,000.	

Solution. (*a*) In this case, the canal bed level (198.1 m) is higher than the drainage bed level (194.9) by 3.2 m and; therefore, we may think of taking the canal above the drainage. Further, drainage FSL (198.5 m) is above the canal bed (198.1 m), we will have to syphon the drainage water ; and thus a *syphon aqueduct* can be constructed at the site. However, since syphoning of drainage may not prove economical, and on the other hand, the canal bed may be raised slightly so as to pass drainage water freely below the canal trough (as the difference in CBL and drainage *HFL* is quite small *i.e.* 0.3 m). In that case, the minimum canal bed level to be kept at the crossing will be equal to

= Drainage HFL + thickness of canal trough slab + Min. free-board
= 198.5 + 0.6 + 0.9 = 200 m.

Therefore, bed of canal trough will have to be raised by an amount equal to 200 – 198.1 = 1.9 m at the site of the crossing. Hence, hump is provided along the length of trough with flat u/s slope and d/s slope to join to the natural canal bed levels on both sides. The crossing will then be designed as an **Aqueduct.** The design procedure as used in example 14.1 can be adopted.

*Although the canal is evidently a pitched canal as the velocity $\left(i.e. \dfrac{354}{30.2 \times 6.2} = 1.89 \text{ m/sec.}\right)$ is high, still these pitching lengths which would be necessary to be properly maintained for the safety of the structure have been worked out. Moreover, due to pitched canal, the scouring tendencies etc. will be less, however, for safety and conservative design purposes, the various data have been worked out assuming that in a worst case, the pitching of canal may get broken.

(*b*) In this case, the canal bed level (209.4 m) is higher than the drainage bed level (207.4 m) by about 2 m ; so we may think of taking the canal above the drainage. Further, the drainage HFL (209.3 m) is below the canal bed level (209.4 m) by only 0.1 m ; which is less than the free-board and slab thickness, hence, the drainage will have to be syphoned below the canal. The other possibility of raising the canal bed is not being adopted here, as the drainage is not too large in comparison with canal discharge.

The work-will, therefore, be designed as a **syphon aqueduct**, and design procedure adopted as in example 14.2.

14.6. Provision of Joints and Water Bars in R.C.C. Ducts of Aqueducts and Super Passages

R.C.C. box troughs (or ducts) carry water in aqueducts and super-passages, as explained in the previous articles. The box trough will span across the drain or the canal, rested at suitable intervals (spans) on piers or abutments. The construction of such R.C.C. troughs will involve various types of joints, such as *construction joints, expansion joints,* etc. Water will leak through these joints, unless proper arrangements are made to prevent such leakages. *Water bars* or *water-stops* are provided to stop such leakages through the joints. The water bars may be made of metals like copper sheets or galvanised iron sheets, or of rubbers (natural as well as synthetic), or of P.V.C., etc.

14.6.1. Types of Joints in R.C.C. Constructions. The various types of joints that may be involved in R.C.C. constructions can be divided into the following types :

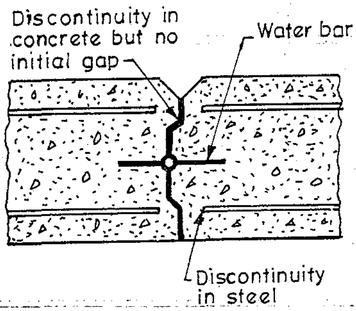
(1) *Movement joints* including the :

(*i*) *Contraction joints*, (*ii*) *Expansion joints*, and (*iii*) *Sliding joints* ; and

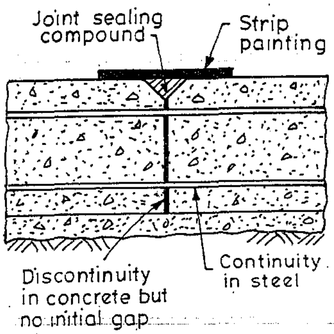
(2) *Construction joints.*

These joints are discussed below :

1 (*i*). **Contraction joints.** It is a movement joint with a deliberate discontinuity but no initial gap between the concrete on either side of the joint, the joint being intended to accommodate contraction of concrete. (See Fig. 14.37).



(*a*) Complete contraction joint



(*b*) Partial contraction joint

Fig. 14.37. Typical contraction joints.

A distinction should also be made between a *complete contraction joint* (See Fig 14.37 *a*) in which both concrete and reinforcing steel are interrupted ; and a *partial contraction* joint (See Fig. 14.37 *b*) in which only the concrete is interrupted, the reinforcing steel running through.

1(*ii*). **Expansion joints.** A movement joint with complete discontinuity in both reinforcement and concrete, and intended to accommodate either expansion or contraction of the structure (See Fig. 14.38) is known as an expansion joint.

In general, such a joint requires the provision of an initial gap between the adjoining parts of a structure, which by closing or opening, accommodates the expansion or contraction of the structure. Design of the joint as to incorporate sliding surfaces is not, however, precluded and may sometimes be advantageous.

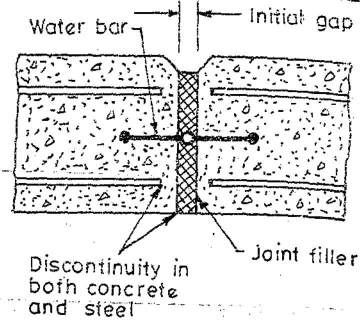


Fig. 14.38. A typical Expansion joint.

1(*iii*). **Sliding joints.** A movement joint with complete discontinuity in both reinforcement and concrete at which special provision is made to facilitate relative movement in place of the joint is known as a sliding joint. A typical application is in between a wall and a floor in same cylindrical tank designs (See Fig. 14.39)

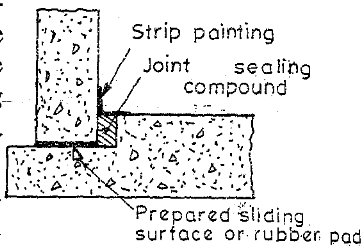


Fig. 14.39. A typical sliding joint.

(2) **Construction joints.** A joint in the concrete introduced for convenience in construction at which special measures are taken to achieve subsequent continuity without provision for further relative movement, is called a construction joint. A typical application is between successive lifts in pouring concrete in walls of a reservoir or a box trough. (See Fig. 14.40).

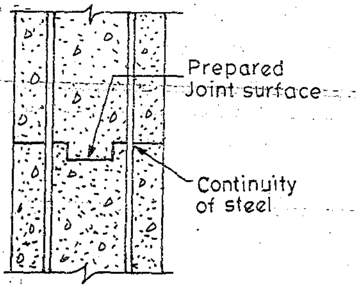


Fig. 14.40. A typical construction joint.
(Provision of water bar may also be made not shown here)

The position and arrangement of all the construction joints should be predetermined by the design engineer. *Consideration should be given to limiting the number of such joints, and to keeping them free from possibility of percolations in a similar manner for as for contraction joints.* Water bars may, therefore, be provided in the construction joints also, as is done in contraction joints (Fig. 14.37).

14.6.2. Spacing of Joints in R.C.C. Structures. Unless alternative effective means are taken to avoid cracks by allowing for the additional stresses that may be induced by temperature or shrinkage changes or by unequal settlement, movement joints should be provided at the following spacings :

(*i*) In reinforced concrete floors, movement joints should be spaced at not more than 7.5 m apart in two directions at right angles. The wall and floor joint should be in line, except where sliding joints occur at the base of the wall, in which case correspondence is not so important.

(*ii*) For floors with only nominal percentage of reinforcement (smaller than the minimum specified), the concrete floor should be cast in panels with sides not more than 4.5 m.

(*iii*) In concrete walls, the vertical movement joints should normally be placed at a maximum spacing of 7.5 m in reinforced walls and 6 m in unreinforced walls. The maximum length desirable between vertical movement joints will depend upon the tensile strength of the walls, and may be increased by suitable reinforcement. Thus when a sliding layer is placed at the foundation of a wall, the length of wall that can be kept free of cracks depends upon the capacity of wall section to resist the friction induced at the plane of sliding. Approximately, the wall has to stand the effect of a force at the plane of sliding equal to weight of half the length of wall multiplied by the coefficient of friction.

(*iv*) Amongst the movement joints in floors and walls as mentioned above, expansion joints should normally be provided at a spacing of not more than 30 m between successive expansion joints or between the end of the structure and the next expansion joint, all other joints being of the contraction type.

(*v*) When, however, the temperature changes to be accommodated are abnormal or occur more frequently than usual as in the case of storage of warm liquids or in uninsulated roof slabs, a smaller spacing than 30 m should be adopted, (that is a greater proportion of the movement joints should be of the expansion type). When the range of temperature is small, for example, in certain covered structures, or where restraint is small, for example, in certain elevated structures, none of the movement joints provided in small structures upto 45 m length need be of the expansion type. Where sliding joints are provided between the walls and either the floor or roof, the provision of movement joints in each element can be considered independently.

14.6.3. Width of Gap in Expansion Joint. An expansion joint requires the provision of an initial gap between the concrete faces on the two sides of the joint. The initial width of this gap should be sufficient to accommodate freely the maximum expansion of the structure.

In determining this initial width, consideration should be given to the requirements of the jointing materials. These will normally require the maintenance of certain minimum width of gap during maximum expansion of the structure. The joint should also be suitably treated as to maintain water-tightness during movement of the joint.

14.6.4. Jointing Materials. Jointing materials may be classified as follows :

(*i*) *joint fillers* ;

(*ii*) *Water bars* and *joint cover plates*; and

(*iii*) *Joint sealing compounds* (including primers where required)

They are discussed below :

(*i*) **Joint fillers.** Joint fillers are usually compressible sheet or strip materials used as spacers. They are fixed to the face of the first placed concrete and against which the second placed concrete is cast. With an initial gap of about 30 mm, the maximum expansion or contraction that the filler materials may allow may be of the order of 10 mm.

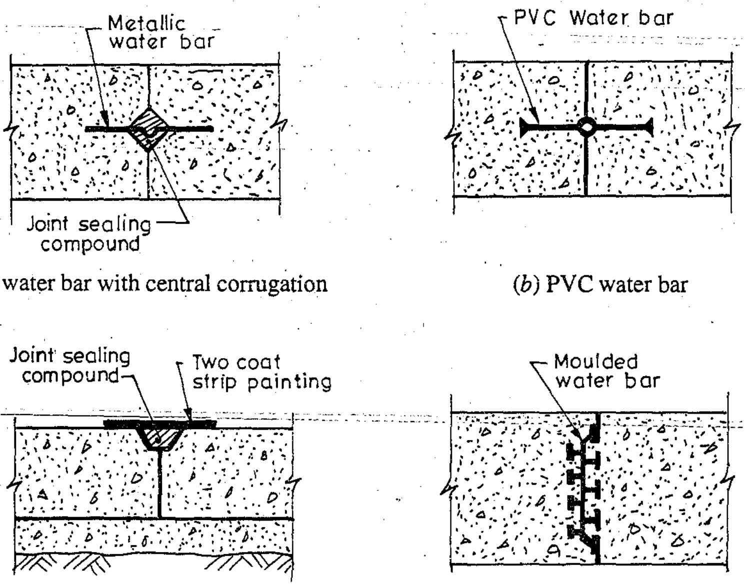
Joint fillers, as at present available, can not by themselves function as watertight expansion joints. They may be used as support for an effective joint sealing compound in floor and floor joints. But they can only be relied upon as spacers to provide the gap in an expansion joint, the gap being bridged by a water bar (See Fig. 14.42).

(*ii*) **Water bars.** Water bars or *water stops* are the preformed strips of impermeable materials which are embedded in concrete during construction, so as to span across the joint, as to provide a permanent water tight seal during the whole range of joint movement.

The water bars are usually made of *metal* sheets like copper, galvanised iron etc. ; or may be made of *rubber* (natural or synthetic) ; or of plastics like *P.V.C.* (Polyvinyl chloride). *Metal bars* can be used in water conveyance structures or dams, where the foundations are not expected to yield appreciably ; otherwise they will snap. *Annealed copper* is the most common metal used for water stops, in thickness not less than 1.5 mm; but it is likely to become brittle in course of time and may then crack. *Monel metal* is considered more durable though sometimes 20 gauge *galvanised iron* or *steel* strips are also used.

The most usual shape of metal bars is in the form of strips with central longitudinal corrugation, to obtain a V, U or M shape. The total width of the water bar may range from 150 mm to 300 mm. For box aqueducts etc., 180 to 225 mm wide water bar may suffice; while for dams, 300 mm wide bars may be preferred. *Rubber water bars* are, however, preferred to metal bars for their economy, and suitability to some what yielding foundations. Metal and rubber water bars are invariably used in ducts of aqueducts and super-passages in expansion joints; *PVC water bars*, on the other hand, are, usually adopted in contraction and construction joints.

Typical use of various types of water bars in contraction or construction joints are shown in Fig. 14.41 (*a*) to (*d*); whereas Fig. 14.42 (*a*) to (*g*) shows the use of various type of water bars in expansion joints.



(*a*) metal water bar with central corrugation

(*b*) PVC water bar

(*c*) metal water bar with two coat strip painting

(*d*) moulded water bar

Fig. 14.41. Typical details showing use of water bars surrounded with joint sealing material in Contraction Joints.

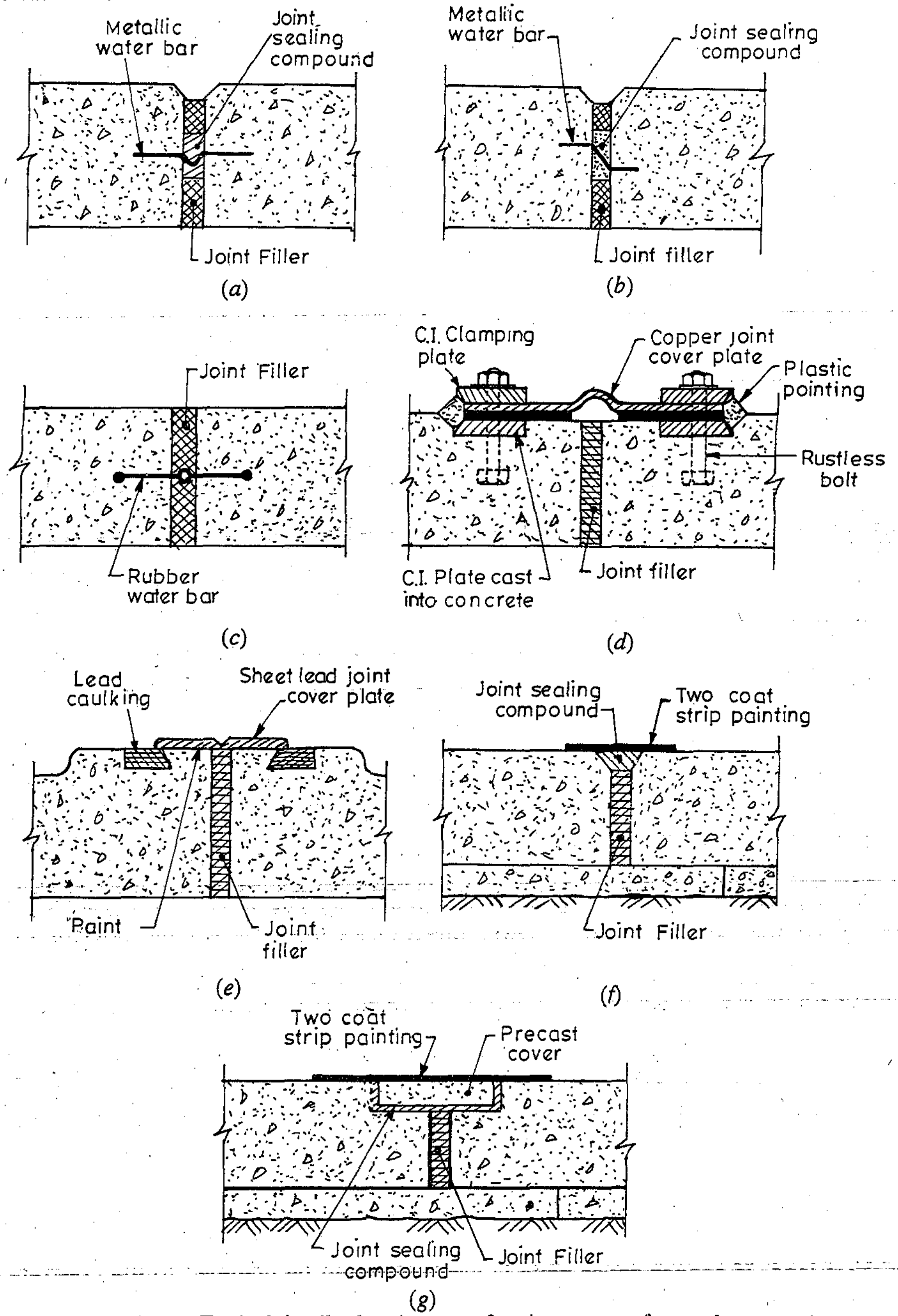
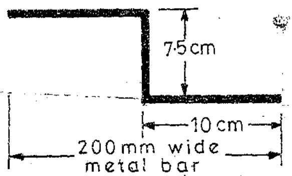
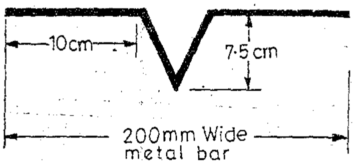


Fig. 14.42. Typical details showing use of various types of water bars and jointing materials in Expansion joints.

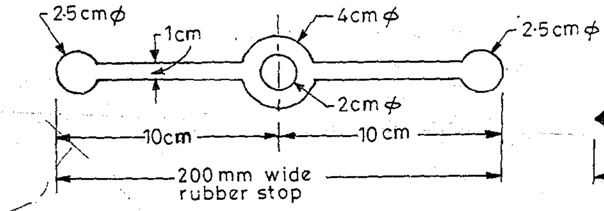
Most Commonly used dimensions & shapes of *metal water stops* (Z-shape and V-shape), *rubber water stop* (dumb well with central bulb type shape) for expansion joints; and of PVC water stop for construction joints of aqueducts and super-passages, are shown in Fig 14.43 (*a*) to (*d*).



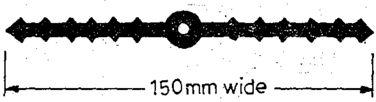
(*a*) 200 mm wide Z-shape metal water stop for expansion joints



(*b*) 200 mm wide V-shape metal water stop for expansion joints.



(*c*) 200 mm wide dumb well with central bulb type rubber water stop for expansion joints



(*d*) 150 mm wide *PVC* water stop serrated with central bulb for construction joints.

Fig. 14.43. Dimensions and shapes of commonly used water bars for ducts of Aqueducts and Super-passages.

With all water bars, it is important to ensure proper compaction of the concrete. The bar should have such shape and width that the water path through the concrete round the bar should not be unduly short.

The holes sometimes provided on the wings of copper water bars to increase bond, shorten the water path and may be disadvantageous. *The water bar should either be placed centrally in the thickness of the wall ; or its distance from either face of the wall should not be less than half the width of the bar. The full concrete cover to all reinforcement should be maintained.*

The strip water bars at present available in the newer materials need to be passed through the end shutter of the first-placed concrete. It can be appreciated, however, that the use of the newer materials make possible a variety of shapes or sections. Some of these designs, for example, those with several projections (*see* Fig. 14.41*d*), would not need to be passed through the end shutter and by occupying a bigger proportion of the thickness of the joint, would also lengthen the shortest alternative water path through the concrete.

(*ii b*) **Joint Cover Plates.** Joint cover plates are sometimes used in expansion joints to avoid the risk of a fault in an embedded water bar. The cover plate may be of copper or sheet lead. If copper cover plate is used, it should be clamped to the concrete face on each side of the joint using suitable gaskets to ensure water tightness (*see* Fig. 14.42*d*). If sheet lead is used, the edges may return into grooves formed in the concrete and be made completely watertight by lead caulking (*see* Fig. 14.42 *e*). Faces of the concrete to which sheet lead is to be fixed should be painted with bituminous or other suitable composition, and the lead sheet should be similarly coated before fixing.

(*iii*) **Joint Sealing Compounds.** Joint sealing compounds are impermeable ductile materials which are required to provide a watertight seal by adhesion to the concrete throughout the range of joint movement. The commonly used materials are based on asphalt, bitumen, or coal tar pitch with or without fillers, such as limestone or slate dust, asbestos fibre, chopped hemp, rubber or other suitable material. These are usually applied after construction or just before the reservoir is put into service by pouring in the hot or cold state, by trowelling or gunning or as performed strips ironed into position. They may also be applied during construction, such as by packing round the corrugation of a water bar. A primer is often used to assist adhesion and some local drying of the concrete surface with the help of a blow lamp is advisable. The length of the shortest water path through the concrete should be extended by suitably painting the surface of the concrete on either side of the joint.

The main difficulties experienced with this class of material are in obtaining permanent adhesion to the concrete during movement of the joint, whilst at the same time ensuring that the material does not slump or is not extruded from the joint.

In floor joints, the sealing compound is usually applied in a chase formed in the surface of the concrete along the line of the joint (*see* Fig. 14.42*c*). The actual minimum width will depend on the known characteristics of the material. In the case of an expansion joint, the lower part of the joint is occupied by a joint filler (*see* Fig. 14.42*f*). This type of joint is generally quite successful, since retention of the material is assisted by gravity and, in many cases, sealing can be delayed until just before the reservoir is put into service so that the amount of joint opening subsequently to be accommodated is quite small. The chase should not be too narrow or too deep to hinder complete filling and the length of the shortest water path through the concrete should be extended by suitably painting the surface of the concrete on either side of the joint. Here again a wider joint demands a smaller percentage distortion in the material.

An arrangement incorporating a cover slab, similar to that shown in Fig. 14.42 g, may be advantageous in reducing dependence on the adhesion of the sealing compound in direct tension.

Use of sealing compounds for vertical joints is not very successful. A stepped-joint instead of a straight through-joint with a water bar incorporated in the joint and sealing compound packed round the corrugation of the water bar would be much more successful.

Complete typical details of a 30 mm wide expansion joint adopted in the construction of an Aqueduct are shown in Fig. 14.44, where two water stops (225 mm wide rubber

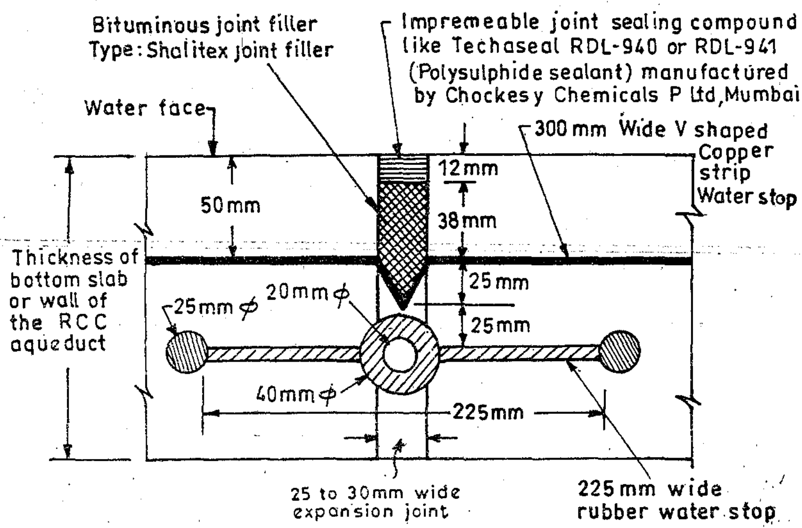


Fig. 14.44. Details of typical Expansion joint in the base slab and walls of R.C.C. duct or trough.

water stop and 300 mm wide V-shaped copper water stop) were used to ensure complete water-tightness.

PROBLEMS

1. (*a*) What is meant by a "Cross-Drainage Works" ? Explain as to why such works are not met within a ridge canal system. (*Madras University, 1973*)

(*b*) Write short notes on :

(*i*) Syphon

(*ii*) Super passage

(*iii*) Syphon aqueduct. (*Madras University, 1973*)

2. Write short notes on :

(*i*) Aqueduct

(*ii*) Syphon aqueduct

(*iii*) Canal syphon (*Madras University, 1972*)

(*iv*) Level crossing.

3. Design and give a dimensional sketch of an aqueduct to carry water of an earth canal over a drainage with the following data :

RL of bed of drainage	= 520.00 m
HFL of drainage	= 523.00 m
Bed width of drainage	= 50.00 m
Side slopes of drainage at crossing	$= \frac{1}{2} : 1$
R.L. of ground	= 525.00 m
R.L. of bed of canal	= 524.50 m
Discharge of canal	= 30 cumecs
Depth of water in canal	= 1.70 m
Bed width of canal	= 22.0 m.

4. (*a*) Name the different types of cross drainage works. Explain how you would avoid one type of cross drainage work and prefer to adopt another type by simply changing the alignment of the canal taking off from a head-work. (*Engg. Services, 1974*)

(*b*) Under what conditions of drainage and canal crossings are syphons provided ? Draw a plan and section through a typical branch canal syphon, and suggest a method for reducing uplift on the floor of the work.

5. (*a*) What are the different types of cross drainage works that are necessary on a canal alignment? State briefly the conditions under which each one is used. (*Madras University, 1974*)

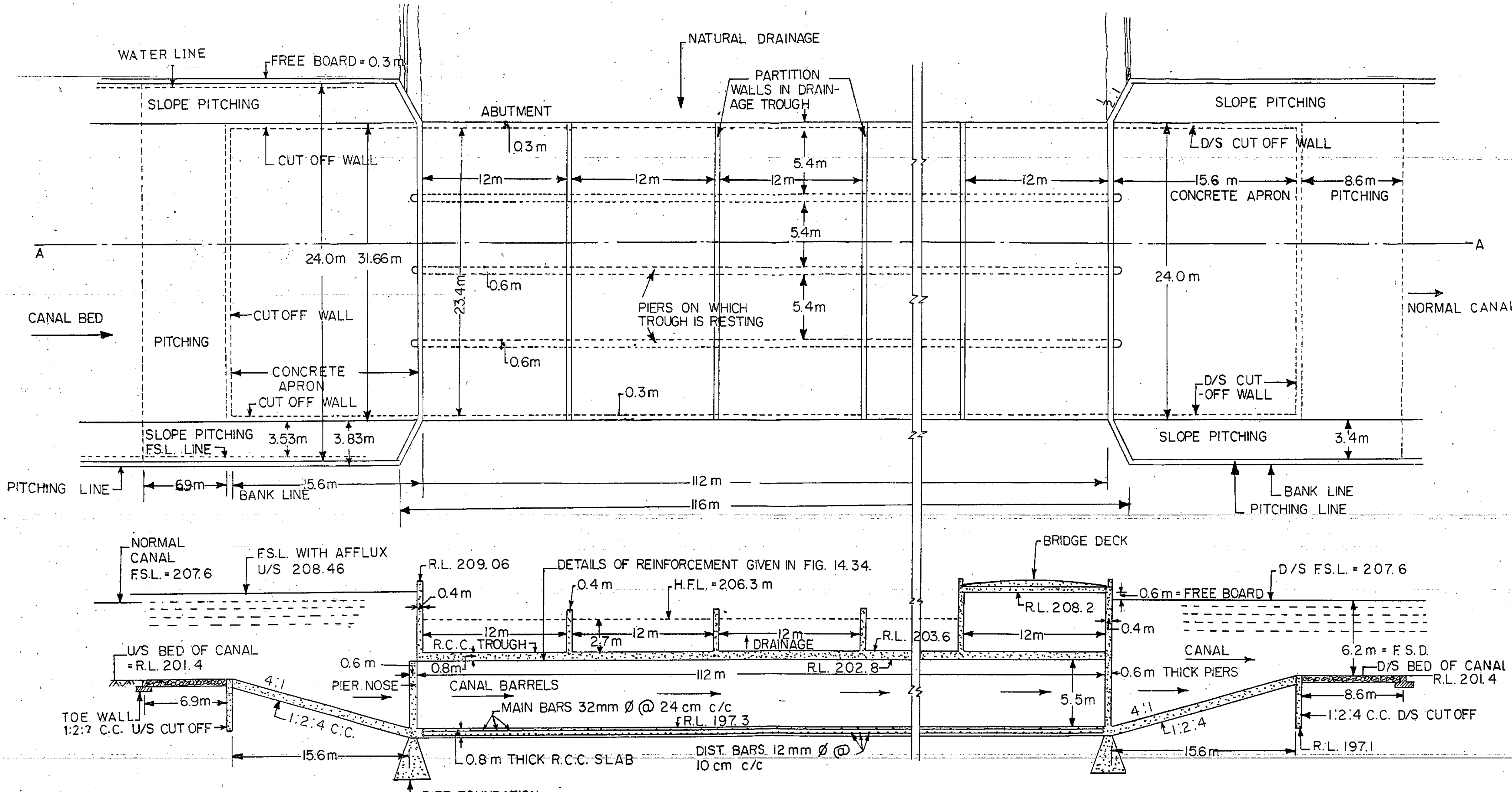
(*b*) Discuss with neat sketches, the three different types of aqueducts which can possibly be constructed depending upon the size of the drainage to be passed below the canal. Also discuss the factors governing the choice of any of these three types of aqueducts.

6. Show by rough sketches the type of cross drainage work suitable for each of the following cases:

(*a*)

	Drain	*Canal*
Discharge in cumecs	200	20
BL	25.00 m	30.00 m
FSL		31.50 m
HFL	28.00 m	

[**Ans.** Aqueduct of Fig. 14.3 is recommended]



SECTION AT A-A

Fig. 14.36

(*b*)

	Drain	*Canal*
Discharge in cumecs	2 cumecs	400 cumecs
BL	52.2 m	48.00 m
FSL		53.00 m
HFL	53.2 m	------

[**Ans.** Syphon of Fig. 14.7 is recommended]
(*Madras University, 1976*)

7. Discuss the various types of cross drainage works used in canal systems. What considerations govern the selection of the different types of works, mainly depending upon the levels of the canal and the drainage. Illustrate by drawing a neat sketch of each type of structure.

8. Give neat sketch of suitable designs of aqueducts for each of the following crossings :

(*a*) A major canal over a small drainage. [**Ans.** Aqueduct of type I, Fig. 14.10]

(*b*) A canal carrying low discharge over a large drainage. [**Ans.** Aqueduct of Type II, Fig. 14.11]

(*c*) A major canal over a large drainage. [**Ans.** Aqueduct of Type III, Fig. 14.12]

Explain in detail the method of design of fluming required in the crossing at (*c*) above.

9. (*a*) State under what circumstances you will recommend the use of the following cross drainage structures :

(*i*) Syphon
(*ii*) Inlet
(*iii*) Aqueduct

(*b*) Following data were collected from a syphon aqueduct :

Diameter of the barrel (single)	= 3 m.
Length of the barrel	= 90 m
Discharge through the barrel	= 25 cumecs
Friction factor (in Darcy-Weisbach formula)	= 0.013
Coefficient of bend loss (2 bends)	= 0.10
Coefficient of head loss in expansion at outlet	= 0.20
Coefficient of head loss in contraction at inlet	= 0.10.

Determine afflux. Neglect velocity head in drainage channel. (*AMIE, 1970*)

[Solution :

$$\text{Total head loss} = h = \underbrace{0.10\left(\frac{V^2}{2g}\right)}_{\text{entry loss}} + \underbrace{2\times 0.10\left(\frac{V^2}{2g}\right)}_{\text{bend loss}} + \underbrace{0.2\left(\frac{V^2}{2g}\right)}_{\text{outlet expansion loss}} + \underbrace{\frac{f' L V^2}{2gd}}_{\text{friction loss in pipe}}$$

$$= 0.5\frac{V^2}{2g} + \frac{0.013\times 90}{3}\frac{V^2}{2g} = 0.89\frac{V^2}{2g}$$

$$\text{Velocity in the barrel} = \frac{Q}{A} = \frac{25}{\frac{\pi}{4}\times(3)^2}\ \text{m/sec.} = 3.52\ \text{m/sec.}$$

$$\therefore \qquad h = 0.89\times\frac{(3.52)^2}{2\times 9.81} = 0.57\ \text{m.}\ \textbf{Ans.]}$$

10. (*a*) What are cross-drainage works ? What is the necessity of such a work in a canal project, and how does this necessity is fulfilled by such works ?

(*b*) Enumerate the different methods which may be used for designing the canal transitions for a flumed canal, and the conditions under which each can be used. Describe in details a method of designing canal transitions when water depth may or may not remain constant.

11. (*a*) What is meant by cross drainage works and what is their importance in a canal project ?

(*b*) Describe briefly the step by step procedure that you will adopt for designing an unflumed syphon aqueduct.

12. Design a syphon aqueduct with the following data :

For Canal

Discharge	= 56 cumecs
Bed width	= 32 m
F.S. depth	= 1.98 m
R.L. of bed	= 267.00 m

For Drainage

High flood discharge	= 425 cumecs
HFL	= 268.20 m
General bed level of low water cross-section	= 265.50 m
General ground level	= 267.20 m

13. Design a suitable cross drainage work, if the crossing in problem 12 above is shifted upstream along the drainage so that the HFL is 269.55 m, the bed level of the drainage is 267.00 m, and general ground level is 269.05 m. The canal data remaining the same.