15

*Construction of Culverts and Small Road Bridges Across Drains and Canals**

15.1. Introduction

Whenever a new road has to cross an existing drain or a canal, or sometimes, when a drain has to cross an existing road ; a small bridge or a culvert** is constructed at the point of crossing. The number of such culverts required in any road or canal project is generally very large and account for a large percentage of the total expenditure on the project. These small bridges or culverts should therefore, be designed safely and economically, so as to avoid unnecessary over investments. In the pages below, we shall, therefore, discuss the important considerations that are involved in their designs.

15.2. Data Collection

After a proper site of construction has been decided on some straight reach of the stream, the highest flood level at the site should be determined. The longitudinal section of the stream extending by about 200 to 1500 metres upstream and downstream should be plotted depending on the catchment area of the stream (at the point of construction) varying from 250 to 1240 hectares or more. The actual flow velocity may be observed during times of floods, which may provide a good check on the velocity calculated theoretically.

15.3. High Flood Discharge Computations

After collecting the required necessary data, the maximum discharge that is likely to reach at the site may be calculated by using any of the various methods given in article 7.9. The various empirical formulas given for this purpose should only be used by experts and a new designer should try to use the Rational formula only.

Road bridges and culverts are normally designed for 1 in 50 year frequency discharge, although important rail bridges may be designed for 1 in 100 years discharge.

After the design flood discharge has been finalised, the total span and number of spans of the bridge should be decided, as given in articles 15.4 and 15.8.

15.4. Linear Waterway of the Bridge

The linear waterway of a bridge across a purely alluvial stream is generally kept equal to its regime width (W) as given by Lacey. The Lacey's regime width of the stream is taken equal to its wetted perimeter (P), given by the equation,

$$W = P = 4.75\, Q^{1/2}$$

$$\therefore \quad L = 4.75\, Q^{1/2} \qquad ...(15.1)$$

where Q is the dominant discharge of the river

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* The discussions here are based on Indian Standards laid down by Indian Road Congress in its various publications which are hereby duely acknowledged.

** A bridge with span upto 8 m is called a culvert.

Hence, the existing width of the stream is contracted at the bridge site by upstream wing walls (or training works) so as to make it equal to W, and the bridge may then be spanned across it. The contraction of the stream up to the regime width leads to economy, as the bridge length is reduced ; but further contraction beyond regime width are generally not economical, because any contraction beyond W (Lacey's regime width) will increase the discharge intensity through the bridge spans, and thus increasing the Lacey's regime scour depth (R). Due to the increase in scour depth, the foundations of the bridge piers, abutments, and cut off walls will have to be taken deeper. The necessity of providing deeper foundations in streams contracted beyond Lacey's regime width, will increase the cost of foundations, although the length of the bridge superstructure will reduce. *Much of the saving in cost expected from decreasing the length of the bridge in such a case will, therefore, generally be eaten up by the corresponding increase in the depth of the foundations and the size of the training works.* Hence, in alluvial streams, it is generally futile to contract the waterway beyond the regime width, as no appreciable saving can be obtained.

However, in streams which are not alluvial and are either semi-alluvial (sides rigid and bed alluvial) or purely rigid, the waterway may be chosen equal to the actual surface width of the stream measured from edge to edge of water along the H.F.L., on the plotted cross-section. The waterway in purely rigid streams may be kept still less, if appreciable saving is obtained and afflux is not detrimental. For streams which overflow their banks and create very wide surface widths with shallow side sections, the bridge may span the active channel, but this waterway should not be so low as to cause excessive detrimental afflux. The waterway, in such a case has, therefore, to be chosen by judgement and intelligence of the designer with these guidelines.

Note. When the natural available width of the stream is found nowhere near the value given by Lacey's equation. (*i.e.* $W = 4.75\sqrt{Q}$), we must expect to find that the banks, even though not rocky, are not friable enough to be treated as incoherent alluvium and hence, the stream is no longer a stream in alluvium. Such streams may then be *treated as semi-alluvial streams.*

15.5. Scour Depth Computations

It was explained earlier in chapter 4, that, if a constant discharge passes through a straight stable reach of an alluvial stream for an infinite time, the boundary of its cross-section would ultimately become elliptical. This will happen when the stream has achieved a state of regime, and in such a case, the depth in the middle of the stream would be equal to the normal scour depth. This **normal depth of scour** (*i.e.* **Lacey's regime scour depth**) is generally taken as a guideline for fixing the depth of the foundations.

The *Lacey's regime scour depth* (R_r') is calculated as given below :

(*a*) For a purely alluvial stream having a span length (L) equal to W (given by $W = 4.75\sqrt{Q}$).

$$\textit{Regime scour depth} = R_r' = 0.473\left(\frac{Q}{f}\right)^{1/3} \quad ...(15.2)$$

where f is obtained by the equation

$f = 1.76\sqrt{d_{mm}}$, d_{mm} being average particle size in mm ...(15.3) (*b*) For *purely alluvial streams* which are contracted beyond W at the bridge site, the value of Lacey's Scour depth (R') may be computed by using the formula :

$$\textit{Scour depth} = R' = 1.35\left(\frac{q^2}{f}\right)^{1/3} \quad ...(15.4)$$

where q is the discharge intensity per unit width of the stream at bridge site, and equals Q/L, where L is the width of the stream at the bridge site.

R' can also be worked out by using the formula

$$R' = \left(\frac{W}{L}\right)^{0.61} \times R_r' \quad ...(15.5)$$

where W = the Lacey's regime width for alluvial streams

R_r' = the Lacey's regime scour depth given by Eq. (15.2)

Note. The above calculated values of R_r' and R' are depths below water surface elevation of the stream.

(*c*) For *quasi-alluvial streams* (*i.e.* bed is not pitched and sides are banked with stable soils or pitchings), having the bridge span length equal to the length of water line between banks, measured along HFL (called natural unobstructed width of the stream W'), the normal secoured depth (D') may be computed by using the formula :

$$D' = \frac{1.21\, Q^{0.63}}{f^{0.33}\, W'^{0.60}} \quad ...(15.6)$$

where D' = the scour depth in quasi alluvial streams having $L = W'$

W' = the natural unobstructed width of the stream, as shown in Fig. 15.1.

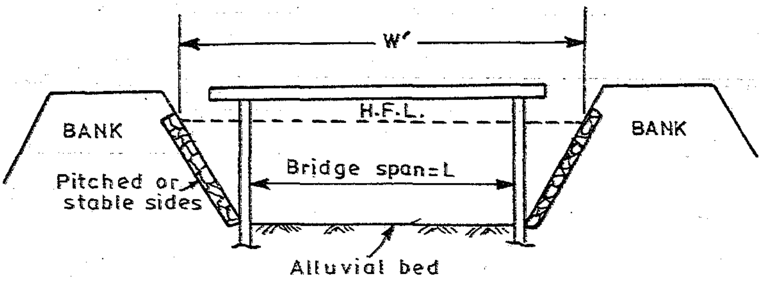

Fig. 15.1. Semi-Alluvial streams.

(*d*) For *quasi-alluvial streams which are contracted* so as to keep L lesser than W', the normal scour depth (D) is given by :

$$D = D'\left(\frac{W'}{L}\right)^{0.61} \quad ...(15.7)$$

where D' is given by Eq. (15.6).

(*e*) For *purely rigid streams* (such as lined pucca drains), however, from theoretical considerations, there will be no scouring, and the foundations of the bridge sub-structure will have to be taken below the bed, not by scour depth considerations, but only by nominal amount, so as to provide sufficient grip on them.

15.6. Maximum Scour Depth Computations

In natural streams, the scouring is not uniform along the bed width. It is not uniform even in straight reaches. Particularly at the bends and also around obstructions to the flow (*e.g.*, the piers of a bridge), there is a deeper scour than the normal scour. The maximum possible scour depth should, therefore, be fairly estimated and the foundations of the structure marginally taken below that level, so as to obtain a factor of safety on that maximum scour. The maximum scour depth is generally taken as x times the normal scour depth R. The value of x is generally taken as 1.5 for a single span structure (with no piers) on a straight reach of the stream, and is taken as 2.0 on bad sites on curves or for multispan structures.

15.7. Depth of Bridge Foundations (D_f)

The foundations of bridges on erodibe beds are taken down below the H.F.L. by an amount equal to 4/3 times the maximum scour depth, or equal to maximum scour depth plus 2 metres for arched bridges and 1.2 metres for other bridges, whichever is more.

This formula is applicable only when no bed floor is provided under the structure and the stream is free to scour as it may. The structure is then said to have deep foundations and it is always considered better to have such foundations as far as possible. But in certain cases, especially on small culverts, bed floors are generally provided. In such a case, the following in considered a safe practice on erodible beds :

"Keep the top of the floor about 0.3 metre below the bed level. Take the foundations of the abutments 1.2 metres below the top of the floor. Provide an upstream curtain wall 1 to 1.5 metres deep and down-stream curtain wall 1.5 to 2.5 metres deep from the top floor depending on the velocity of flow through the structure and erodibility of the bed materials."

These rules are based on the I.R.C. Code of Practice for Road Bridges.

15.8. Total Clear Span and Number of Spans

As a fundamental rule, the number of spans should be as less as possible, since piers obstruct flow. It is particularly important in hilly regions where torrential velocities prevail, and is always better to span from bank to bank using no peirs if possible.

By increasing the length of one span, the number of spans can be reduced. In such a case, the cost of super-structure increases and that of sub-structure decreases. *The most economical span length is one for which the cost of sub-structure is equal to the cost of super-structure.* This economy should, therefore, be worked out for large bridges and where difficult conditions are anticipated in sinking of foundations. Any deviations from the ideal economic case has to be justified strongly. When the regulators, barrages, etc. are provided with bridges, the span is generally guided by the economy as well as the availability of gate sizes.

Length of one span. The economical length of one span say (l) may be chosen as per the following guidelines :

(*a*) *For R.C.C. slab bridges*

$$l = 1.5\,H \qquad ...(15.8)$$

where H = Total height of abutment or pier from the bottom of its foundations to its top
$= D_f +$ Vertical clearance*, *i.e.* free-board
l = Clear span length.

(*b*) *For masonry arch bridges*

$$l = 2H \qquad ...(15.9)$$

where H = Total height between the bottom of the foundations to the intrados of the key stone
l = Clear span length.

Number of spans. When L is more than the economical span length (l), the number of spans required (N) is tentatively found by using

$$L = N.l \qquad ...(15.10)$$

Since N must be a whole number (preferably an odd number), l may have to be modified suitably. Varying span lengths may sometimes be adopted, so as to keep as close as possible to the requirements of economy and to cause least obstruction to the flow.

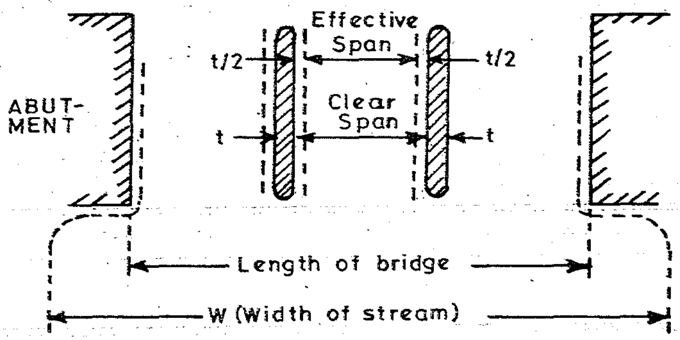

Fig. 15.2. (*a*) Plan at a Bridge site.

Effective clear span. It is assumed that each bridge pier causes contraction in the filaments of water equal to half its thickness on each side, and the obstruction at the ends due to abutments or pitched slope is ignored. Hence, the finally selected number and lengths of spans should be such that the sum of all the effective span lengths should be equal to the required linear waterway (L), and this will be equal to

$$L = \Sigma \text{ [of all the effective spans]}$$
$$= \Sigma \text{ [of all the clear span} - \text{Thickness of piers]}$$
$$L = \Sigma \text{ of all the clear spans} - \Sigma \text{ thickness of piers} \qquad ...(15.11)$$

The pier thickness above is taken as the mean value for the height of the pier from HFL to maximum scour depth.

* See Fig. 15.2 (*b*).

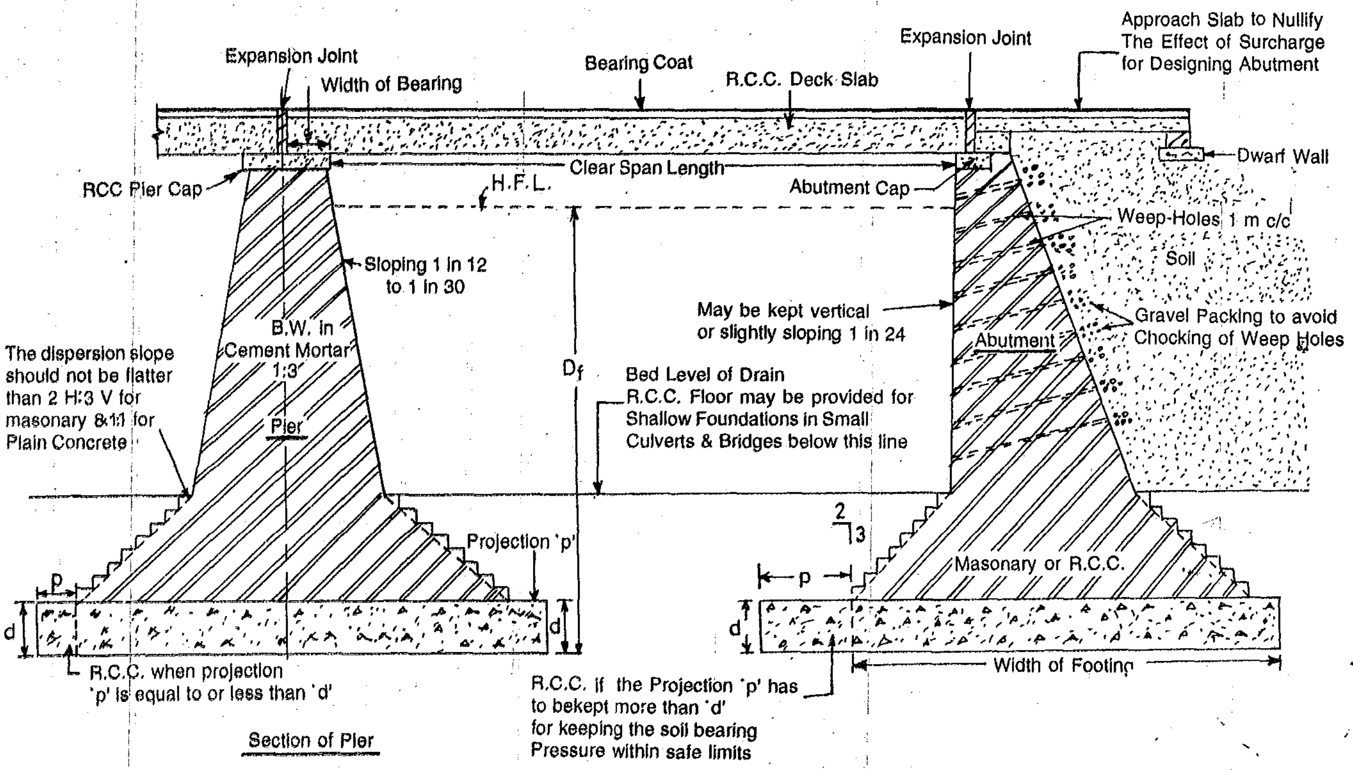

Fig. 15.2. (*b*) Section of abutment and piers for a slab bridge.

15.9. Vertical Clearance and Some Other Tentative Dimensions

They may be fixed as per I.R.C. codes for road bridges as follows :

Table 15.1. Value of Vertical Clearance

Discharge in cumecs	*Vertical clearance or Free-board in metres*
Below 0.3	0.15
0.3 to 3	0.3
3 to 30	0.6
30 to 300	0.9
300 to 3000	1.2
Above 3000	1.5

Table 15.2. Minimum Top widths of PCC or RCC Piers and Abutments

Clear span in metres	*Width of abutment in metres*	*Width of pier in metres*
3	0.38	0.45
6	0.45	0.55
9	0.55	0.65
12	0.65	0.75
15	0.75	0.88

Table 15.3. Minimum Top Widths of Masonry Piers and Abutments

Span	*3 m*	*6 m*	*12 m*	*24 m*	*40 m*	*50 m and above*
Top width of piers carrying simply supported spans (in m)	0.5	1.0	1.2	1.6	2.0	2.2
Top width of abutment and of piers carrying continuous spans (in m)	0.4	0.75	1.0	1.3	1.7	1.9

15.10. Afflux Computations

When a bridge is constructed across a contracted stream, water on the upstream will rise up. The maximum rise in water level near the bridge site is nothing but afflux. The greater is this afflux, lesser will be the available clearance for the same deck level. Since the 'clearance' is the distance between the u/s H.F.L. and the bottom of the bridge slab, it is nothing but freeboard. Hence, it is necessary to calculate afflux to examine its effects on clearance and also upon the regime of the channel u/s of the bridge, and also to determine the top levels of training walls, etc.

The afflux (h), the discharge (Q), the unobstructed stream width (W), and the provided linear waterway of the bridge (L) are all interconnected. Greater is the reduction in the linear waterway, the greater is the afflux. Since the downstream depth is not affected by the bridge, as the same is governed by the hydraulic characteristics of the d/s channel, it can be safely assumed that the u/s depth which prevailed before the bridge construction is the same as the d/s depth (y_d) that prevails even after the bridge construction.

Hence, y_d is the depth that prevailed at the bridge site before the construction of the bridge. To estimate afflux, we must know y_d. This can be found from the available gauge discharge curve of the stream, or may be calculated by the hydraulic parameters of the channel.

The discharge passing through the bridge openings can be calculated in the following different ways :

(1) **By Broad Crested Weir formula.** This formula is applicable so long as the afflux, *i.e.* [upstream depth (y_u) — downstream depth (y_d)] is not less than $\frac{1}{4} \cdot y_d$. In this formula, the discharge Q is dependent only upon y_u and is independent of y_d. The fact that the downstream depth y_d is having no effect on the discharge, nor on the upstream depth y_u, when the afflux is not less than $\frac{1}{4}y_d$, is due to the formation of the 'standing wave'.

The discharge Q is then given by

$$Q = 1.71 \cdot C_d L \left[y_u + \frac{V_a^2}{2g} \right]^{3/2} \qquad \text{...(15.12)}$$

where $\frac{V_a^2}{2g}$ = head due to velocity of approach.

y_u = upstream water depth

C_d is the coefficient of discharge and accounts for frictional losses. The various values of C_d for different types of openings are given in Table 15.4.

Table 15.4. Values of C_d for Different Types of Bridge Openings

Type of bridge Opening	*Value of C_d*
Narrow bridge opening with or without floors	0.94
Wide bridge opening with floors	0.96
Wide bridge opening with no bed floors	0.98

(2) **By the Orifice Formula.** When the downstream depth is more than 80% of the upstream depth (*i.e.* the afflux is less than $\frac{1}{4} y_d$), the weir formula is not valid, as the performance of the bridge opening gets affected by the downstream depth (y_d). In such a case, the discharge can be calculated by using the Orifice formula given by

$$Q = C_0 \sqrt{2g} \cdot L \cdot y_d \cdot \left[h + (1+e) \frac{V_a^2}{2g} \right]^{1/2} \qquad \ldots(15.13)$$

The values of coefficients e and C_0 may be taken from the curves given in Figs. 15.3 and 15.4.

COEFFICIENT C_0

FOR THE ORIFICE FORMULA

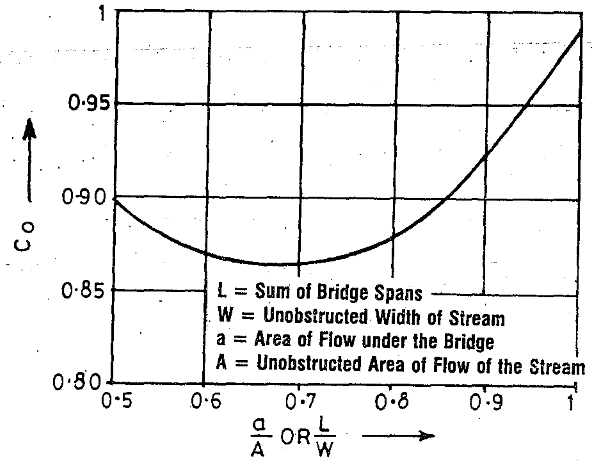

Fig 15.3

COEFFICIENT 'e'

FOR THE ORIFICE FORMULA

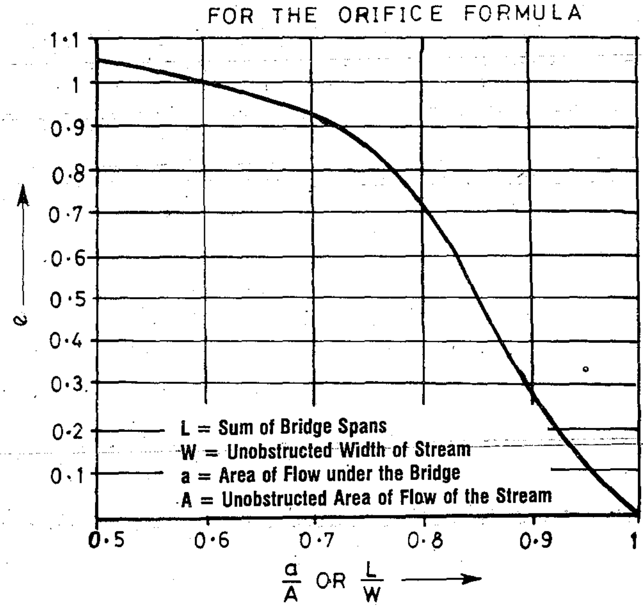

Fig 15.4

The derivations of these two formulas are given below :

Derivation of broad crested weir formula applied to bridge openings

With reference to Fig. 15.5, we have

The total energy (H) at section 1-1.

$$= y_u + \frac{V_a^2}{2g}.$$

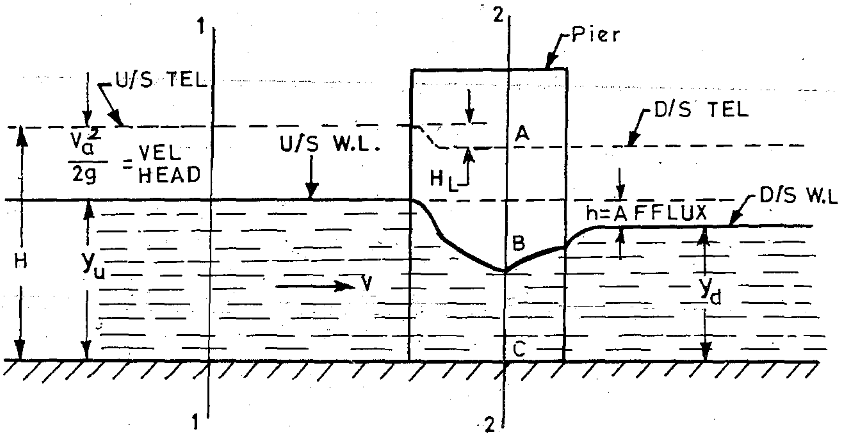

Fig. 15.5.

At section 2-2, let the velocity head AB is a certain fraction of H, say let it be $m \cdot H$.

$$\frac{V^2}{2g} = mH$$

where V is the velocity through the bridge opening.

$$\therefore \quad V = \sqrt{2g \cdot m \cdot H} \qquad ...(i)$$

Now, if the head loss due to entry and friction (*i.e.* H_L) is ignored, then by equating total energies at 1-1 and 2-2, we get

$$H = AB + BC = m \cdot H + BC$$

$$\therefore \quad BC = H - m \cdot H = H(1 - m)$$

The area of flow at section 2-2

$$= BC \times \text{Linear waterway} = BC \times L = H(1 - m) \cdot L$$

$\therefore$ Discharge through the bridge

$$(Q) = \text{Area} \times \text{Velocity} = H(1 - m)\, L \times V \qquad ...(ii)$$

But from (i) we have

$$V = \sqrt{2g \cdot mH}$$

$\therefore$ Discharge $(Q) = H(1 - m) \quad L \cdot \sqrt{2gmH}$

To account for losses at entry and friction, we introduce a coefficient C_d and get the final total discharge formula as

$$Q = C_d \cdot H(1 - m) \cdot L\sqrt{2gmH}$$

$$= C_d \cdot \sqrt{2g} \cdot L \cdot H \cdot (1 - m) \cdot \sqrt{m} \cdot \sqrt{H}$$

$$= C_d \sqrt{2g}\, \sqrt{m}\, (1 - m) \cdot \sqrt{m} \cdot L \cdot H^{3/2}$$

or

$$Q = C_d \cdot \sqrt{2g} \cdot L \cdot H^{3/2} \cdot [m^{1/2} - m^{3/2}] \qquad ...(iii)$$

The depth BC adjusts itself in such a way that the discharge passing through it, is maximum. Hence,

$$\frac{dQ}{dm}=0$$

Differentiating (*iii*), we get

$$\frac{dQ}{dm} C_d \cdot \sqrt{2g} \cdot L \cdot H^{3/2}\left[\tfrac{1}{2}m^{-1/2}-\tfrac{3}{2}m^{1/2}\right]=0$$

$$\therefore \qquad \frac{1}{2}m^{-\frac{1}{2}}-\frac{3}{2}m^{\frac{1}{2}}=0$$

or $$m^{-\frac{1}{2}}=3m^{\frac{1}{2}}$$

or $$m=\tfrac{1}{3}.$$

Hence, $$Q=C_d \cdot \sqrt{2g} \cdot L \cdot H^{3/2}\cdot\left[(\tfrac{1}{3})^{1/2}-(\tfrac{1}{3})^{3/2}\right]$$

$$=C_d \cdot \sqrt{2\times 9.81}\left[0.574-0.189\right]L \cdot H^{3/2}.$$

$$=1.71\ C_d \cdot L \cdot H^{3/2}$$

or $$Q=1.71 \cdot C_d \cdot L \cdot \left[y_u+\frac{V_a^2}{2g}\right]^{3/2} \qquad ...(15.12)$$

This is the required equation (15.12).

Derivation of orifice formula applied to bridge openings

With reference to Fig. 15.6.

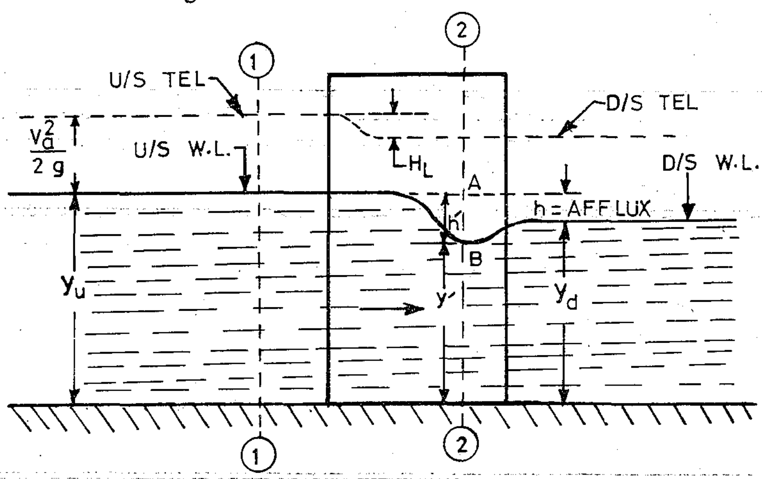

Fig 15.6

Applying Bernoulli's Equation at sections 1-1 and 2-2 and ignoring the loss (H_L) due to entry and friction, we get

$$y_u+\frac{V_a^2}{2g}=BC+\frac{V^2}{2g}=y'+\frac{V^2}{2g}$$

or $$\frac{V^2}{2g}=\left[y_u-y'+\frac{V_a^2}{2g}\right]$$

or $$V = \sqrt{2g\left(y_u - y' + \frac{V_a^2}{2g}\right)}$$

Putting $y_u - y' = h'$, we get

$$V = \sqrt{2g\left(h' + \frac{V_a^2}{2g}\right)} \qquad ...(i)$$

The discharge through section 2-2 is given by

$$Q = \text{Area} \times \text{Velocity}$$

$$= (L \cdot y') \times \sqrt{2g\left(h' + \frac{V_a^2}{2g}\right)}$$

When afflux is small $\left(i.e. < \frac{y_d}{4}\right)$, y' is nearly equal to y_d, then

$$Q = L \cdot y_d \sqrt{2g} \cdot \left[h' + \frac{V_a^2}{2g}\right]^{1/2} \qquad ...(ii)$$

Now h is less than h' because when the flow emerges from the bridge, the water surface rises due to recovery of some velocity head as depth. If $e\left(\frac{V_a^2}{2g}\right)$ is the velocity head that is converted into potential head, we have

$$h' = h + e\left(\frac{V_a^2}{2g}\right) \qquad ...(iii)$$

Substituting in (*ii*), we get

$$Q = L \cdot y_d \cdot \sqrt{2g} \cdot \left[h + e \cdot \frac{V_a^2}{2g} + \frac{V_a^2}{2g}\right]^{1/2}$$

$$Q = L \cdot y_d \cdot \sqrt{2g} \cdot \left[h + (1 + e)\frac{V_a^2}{2g}\right]^{1/2} \qquad i.e. \text{ Eq. } (15.13)$$

This is the required equation (15.13).

(3) **By Moles-worth Formula.** The afflux for non-erodible beds may be easily computed approximately by using this empirical formula, as given below :

$$h = \left[\frac{V^2}{17.88} + 0.01524\right]\left[\left(\frac{A}{a}\right)^2 - 1\right] \qquad ...(15.14)$$

where h = afflux in metres

V = velocity in unobstructed stream in m/sec.

A = unobstructed sectional area of the stream in sq. m.

a = obstructed reduced sectional area of the stream in sq. m. at the bridge site

15.11. Structural Design and Other Detailing of Slab Bridges

While proceeding to design a small bridge across a drain or a canal, the design discharge, the total linear waterway, the span length, the floor level, the invert level of the bridge deck (after making a provision of clearance above H.F.L.), the normal and maximum scour depth, the depth of bridge foundations, etc. are, first of all, decided.

The width of road bridge and number of lanes which it is going to accommodate are then decided depending upon the importance of the road.

Generally, road bridges are designed for two lanes of class A or one lane of class 70 R loading, whichever gives maximum reactions or stresses. The design procedure used for designing R.C.C. slab bridges or any other type of bridges is quite laborious and cumbersome, and the same is available in any good book of R.C.C. designs.

In actual practice, however, such detailed calculations for design of bridge decks, are usually avoided, particularly for small bridges and culverts, by making use of the standard deck designs of different spans, as published by the Ministry of Surface Transport (MOST) and Indian Road Congress (IRC).

These readymade designs not only provide the design of bridge deck, but also provide the maximum reactions that are likely to come on the piers or abutments, carrying such super structures. These reactions coupled with other forces can then be used for designing the piers as well as the abutments of such bridges.

The various forces acting on piers and abutments are taken in accordance with the provisions of I.R.C. publication No. 6–1966 titled "*Standard Specifications and Code of Practice for Road Bridges, Section II (Loads and Stresses)*.

The important forces which are taken in the design are, however, summarised below:

Forces on Piers :

(*i*) *Dead Load* (D.L.) from super-structure (available from readymade design tables).

(*ii*) *Live Load* (L.L.) from super-structure (available from readymade design tables).

(*iii*) *Impact Load** (I.L.) from super-structure (available from readymade design tables)

(*iv*) *Self weight of pier.*

(*v*) (*a*) *Longitudinal braking force*, (*P*) equal to 20% of first train load plus ten per cent of the load of the succeeding trains (in class 70 R loading, this is 70 tonnes). Thus, for ordinary two lane bridges, $P = 0.2 \times 70 = 14$ tonnes. (*b*) For multi-lane bridges, as in (*a*) above for first two lanes, plus five per cent of the loads in lanes in excess of two. Thus, for four lane bridges,

$$P = 0.3 \times 70\ t = 21 \text{ tonnes.}$$

(*vi*) The *wind load* and seismic forces (either of the two) should also be considered wherever predominant. Since the consideration of these forces permit to increase the permissible stresses in brick or concrete (materials used for piers) by 25% or so ; for ordinary designs in non-seismic or less seismic regions and where bridges are not at much height above the natural ground levels, these forces may often be neglected.

(*vii*) The *forces caused due to buoyancy of water* (*i.e.* reduction in the weight of pier material due to uplift exerted by the water surrounding the pier) may also be considered in design. Often this is also neglected when pier thickness, etc. are governed by the minimum permissible limits.

* The 50% impact load is considered to be a part of live load for calculating pressure at the bottom surface of the bed block ; and an impact load varying between 50% to zero is considered for calculating the pressure on the top 3 m of structure below the bed block. Thus, no impact is considered for examining pier or abutment sections below 3 m from bed block.

(*viii*) *Other minor forces* such as horizontal forces caused by water currents, etc.

Forces on Abutments. Besides D.L., L.L., self weight, seismic or wind forces and buoyancy if there, the important force which comes into play is the pressure exerted by the soil fill from behind the abutment. The soil may be dry or wet or saturated depending upon the site conditions. The horizontal braking force and the horizontal force caused by temperature variations will no doubt act in addition to the soil pressure. All these forces may be considered and abutment designed. No tension should be permitted in masonry piers, and the base should be wide enough so that the soil pressure does not exceed the permissible bearing capacity of the soil.

The detailed designs of bridges are beyond the scope of this book, and the students may refer to some standard book on bridge designs.

CAUSEWAYS AND BOX CULVERTS

15.12. Causeways

When a road embankment is constructed across a flat terrain, the runoff from the area having no deep and defined channels in it, may have to be disposed of by some suitable means. In such cases, the surface water will go on collecting on one side of the road embankment, unless drained out either through culvert openings inside the embankment or by allowing the water to submerge the dipped road at frequent intervals. *The second arrangement called a causeway consists in lowering or dipping the road to the ground level at frequent intervals, and thus allowing the water to flow across the road.* The provision of such a dip or causeway in the longitudinal profile of the road and letting the drainage water pass over them, may be feasible in less important towns, and may generally be impracticable because of various limitations, such as : (*i*) flow of water over numerous sections of the road makes its proper maintenance difficult and expensive; (*ii*) when the embankment is constructed high above the ground (which may be necessitated in swampy and water-logged areas), the provision of causeways will require dipping down the high road levels to the ground at frequent intervals and will produce a very undesirable road profile. Hence, under all such circumstances, it may be better to construct culverts at intervals rather than frequently dipping the road for crossing the surface water.

15.13. Pipe Culverts Flowing Full and Box Culverts

After having decided to construct a culvert across some such flat terrain, the runoff likely to be passed may be calculated. Now, the water coming towards the road in such cases is having very low velocities and cannot be passed below the road at those velocities, because in that case, the area of the opening will be very large. Hence, in such cases, the design is based on an increased velocity of flow through the culvert, and to create that velocity, the water is allowed to head up at the inlet end of the culvert. The greater is the heading up of water at the inlet end, the greater is the head causing flow ; and hence, the greater will be the flow velocity and lesser the opening. But this heading up of water can be allowed only to a safe predetermined level, and should be fixed, so that the road bank is not overtopped nor any property in the flood plain damaged.

After fixing the upstream level up to which the water will head up while passing maximum design discharge, the downstream water level is determined. This level will depend on the hydraulic conditions of the tail channel, if at all existing, and is generally taken as equal to the surface level of the natural unobstructed flow at the site before the road embankment is constructed.

The operating head, i.e. the difference of the upstream and downstream levels (say H_L) will give us the maximum head causing flow. The area of the opening should then be decided, so that it is sufficient to pass the design discharge.

If V is the velocity through the culvert opening, which runs full in such a case, then the head loss (H_L) will be equal to the sum of the entrance loss, friction loss in the barrel, and the velocity head in the barrel. The entrance loss depends on the type of entrance provided, and may be taken equal to 0.505 $\frac{V^2}{2g}$ for a square edged entrance, and equal to 0.05 $\frac{V^2}{2g}$ for a well rounded

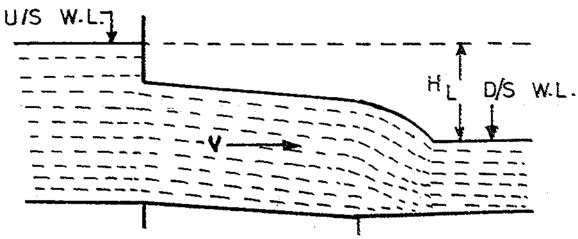

Fig. 15.7. Box or pipe culvert running full even when the outlet is not submerged.

entrance. The friction loss may be calculated by Manning's formula as $\frac{n^2 \cdot V^2 \cdot L}{R^{4/3}}$

Hence, H_L = Entrance loss + Friction loss + Velocity head in barrel

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

where $K_e = 0.505$ for square edged entrance
$= 0.05$ for bell mouthed entrance.

$$\therefore \quad H_L = K_e \cdot \frac{V^2}{2g} + \frac{V^2}{2g} + \frac{n^2 \cdot L}{R^{4/3}} \cdot 2g\left(\frac{V^2}{2g}\right)$$

$$= \frac{V^2}{2g}\left[1 + K_e + \frac{n^2 \cdot L \cdot 2g}{R^{4/3}}\right] = \frac{V^2}{2g}\left[1 + K_e + K_f\right]$$

where $K_f = \frac{n^2 \cdot L \cdot 2g}{R^{4/3}}$

or $$V = \sqrt{\frac{2g \cdot H_L}{1 + K_e + K_f}} \; ; \therefore \; A \cdot V = A \cdot \sqrt{\frac{2g\, H_L}{1 + K_e + K_f}}$$

Hence, knowing the area, the discharging capacity of the culvert, or by knowing the discharge, the required area can be easily computed. The required sized pipe (or a rectangular barrel) may then be constructed through the road enbankment.

PROBLEMS

1. Discuss briefly the various principles and guidelines which help us in fixing the hydraulic design of a bridge culvert.

2. (*a*) Distinguish between 'normal scour depth' and 'maximum scour depth' in connection with bridge design. How does it help in determining the depth of bridge foundation ?

(*b*) How will you fix up the number of spans for a bridge after computing its required linear waterway ?

(*c*) "Reducing the waterway of a bridge will reduce the cost of superstructure, but will increase the cost of sub-structure." Discuss critically the above statement.

3. Write short notes on any two of the following :

(*i*) Afflux at bridges.

(*ii*) Waterway calculations for bridges.

(*iii*) Waterway and scour depth provision in bridge design.

(*iv*) Economical span length for bridges.

(*v*) Provision of cause-ways in road construction.

(*vi*) Box culverts and their afflux computations.